



# **BERGMEKANIKDAG 1996**

**Föredrag**

STIFTELSEN SVENSK BERGTEKNISK FORSKNING  
SWEDISH ROCK ENGINEERING RESEARCH



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SWEDISH ROCK ENGINEERING RESEARCH

**Föredrag vid  
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Redigerad av Per Andersson



## FÖRORD

Föredragen till årets bergmekanikdag har en mycket stark förankring i den praktiska verklighet som de flesta av våra åhörare arbetar i idag. Det räcker att nämna några exempel som Ringen-projektet i Stockholm, Lundbytunneln i Göteborg, Hallandsåsen, Arlandabanan, djupförvar för kärnavfall, storskaligt skivras i Kiruna. Vi har stora och svåra uppgifter. Det kan vara beställare som har att formulera funktionskrav och specifikationer för tekniskt komplicerade arbeten eller entreprenörer som tillsammans med uppdragsgivaren har uppgiften att åstadkomma den produkt som var tänkt. Kvalitetssäkring kallas det idag. Att man levererar det som ställts i utsikt eller att man får vad man köpt.

Konsulter som upprättar handlingar eller ger råd åt olika parter i speciella frågor har en viktig och ofta svår uppgift - vi arbetar som bekant i ett material som till en början inte är fullständigt känt, ändå förväntas vi kunna ställa prognoser och ange tillförlitlighet i bedömningen av livslängder på åttio eller hundra år... Bästa teknik får inte vara detsamma som den mest exklusiva och därmed dyraste. Då får vi snart inte bygga i berg längre. Rimlighetsbedömningar måste alltså göras och grunderna för det kan bara vara god kompetens i nivå med den "bergmekaniska kunskapsfronten" och en i möjligaste mån stringent metodik för att bedöma de osäkerheter som av naturen ligger inbyggda i vårt teknikområde.

Förhoppningen är att dagens föredrag och diskussioner ska bidra till att sprida en del av det bästa av vårt kunnande för tillämpning i den praktiska verkligheten som må gälla dimensionering, brytningsteknik eller förstärknings- och tätningsarbeten i bra berg eller riktigt dåligt, i små tunnlar eller öppna skärningar eller i gruvorter på stora djup.

Vi har glädjen att ha representanter från våra grannland i öst och väst som ytterligare betoning av att vi har mycket gemensamt och värdefulla erfarenheter att utbyta. Vi har också förmånen att få höra Professor Sakurai från Universitetet i Kobe, som är nybliven president i International Society for Rock Mechanics, och som också har erfarenheter att berätta om från effekterna på tunnlar i samband med jordbävningsskatastrofen i Kobe för något år sedan. Vi ser fram emot att få ta del av Professor Sakurais syn på ISRM som ett forum för att främja internationellt bergmekaniskt erfarenhetsutbyte och därefter en intressant och stimulerande Bergmekanikdag 1996.

Stockholm i februari 1996

Tomas Franzén



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**FÖRUNTERSÖKNING  
OCH  
BERGMEKANISK KLASSIFICERING**

**Pre-Investigation  
and  
Rock Mass Classification**

Introduktion: Håkan Stille, KTH



# LESSONS LEARNED FROM FIELD MEASUREMENTS IN ROCK ENGINEERING

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## 1. INTRODUCTION

Numerical analyses such as finite element method, boundary element method, distinct element method and so on have become very popular in the rock mechanics field. These numerical approaches facilitate the consideration of complex geological and geomechanical states of rock masses in the analysis of assessing the stability of rock structures such as tunnels, large caverns, foundations of dams, slopes, etc., and make it possible to achieve the rational design of structures.

However, it is well known that the actual behaviour of structures quite often differs from that predicted by numerical analyses. This difference is mainly due to the fact that many uncertain factors are involved in modelling geological and geomechanical characteristics of rock masses. Geological structures, geomechanical characteristics of jointed rock masses, underground water conditions and initial state of stress of the ground cannot be easily explored, although various kinds of field exploration techniques are now available.

In order to overcome this difficulty, field measurements are carried out during the construction period in order to monitor the stability of structures. The design parameters of geological and geomechanical characteristics used in the original design can then be reevaluated and, if necessary, the original design and construction method can be modified.

This design/construction method is called the *observational procedure*.<sup>1)</sup> In this method, however, a question may arise concerning how to interpret the results of field measurements for

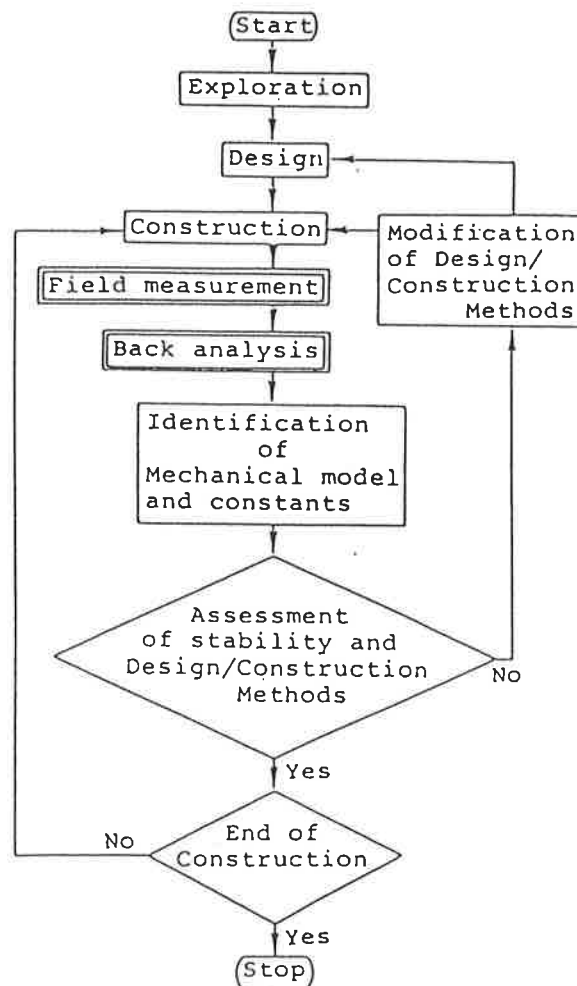


Fig. 1 Procedure of assessing the design and construction methods

monitoring the stability of structures as well as assessing the original design and the construction method of structures.

The stability of structures can be assessed by comparing the measurement results with the allowable values, and it is obvious that the structures are safe if all the measurement values remain smaller than the allowable values. These allowable values are often called the hazard warning levels. This approach may be suitable for monitoring the stability of structures during or after their construction period. However, this approach can hardly be used for assessing the adequacy of the design of structures, because the design parameters cannot be reevaluated directly from the measurement values.

In order to assess the design parameters, the field measurement results must be analyzed to identify the geological and geomechanical characteristics of rock masses. This procedure is called "back analysis" because, in this procedure, the input data are the measured values such as displacements, strains, and stresses, while the output results are material constants identifying mechanical characteristics of rocks. This is exactly the reverse procedure in comparison with "ordinary analysis", where the input data are material constants, while the output results are stress, strain, and displacements.

It is obvious that back analysis is essential in the observational procedure to achieve a rational design and construction method. As far as engineering practice is concerned, it may be worthwhile to mention that back analysis for reevaluating the design parameters must be carried out immediately after taking the measurements. And, if necessary, the original design and construction method must be modified without serious delay during construction.

The observational procedure for assessing the adequacy of the design and construction methods is shown as a flow chart in Fig. 1.

## 2. HAZARD WARNING LEVEL

It is recommended that a hazard warning level should be determined for each measurement items at the design stage, so that during the construction the stability of the structures can be assessed immediately after taking the measurements comparing the measured values with the hazard warning level. When the measured values still remain smaller than the hazard warning level, the stability of the structures is confirmed. However, even if the measurement values are still smaller than the hazard warning level, engineers should always pay attention to what happens after the lapse of a certain period of time (see Fig. 2). If the measured values are predicted to become greater than the hazard warning level after a certain period of time, then the engineers must take some action in modifying the original design.

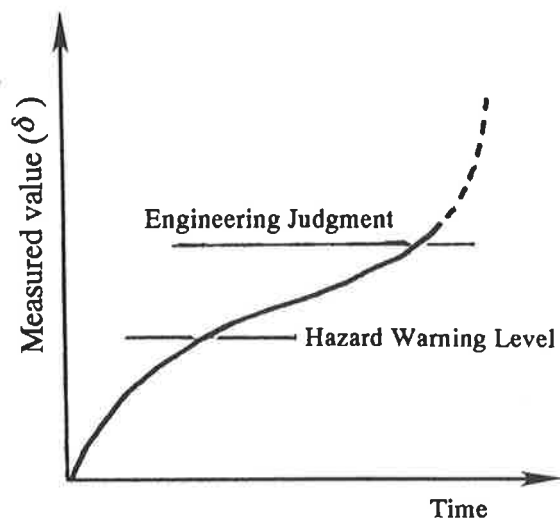


Fig. 2 Schematic diagram for measured value in relation to hazard warning level

The author has previously proposed a hazard warning level for strain and displacement that can be used in the monitoring of the stability of tunnels. The hazard warning level is classified into three stages in relation to the factor of safety as shown in Fig. 3. The hazard warning level of settlement at the crown can then be determined from the corresponding level of strain by using Eq.(2). As an example, the hazard warning levels for a tunnel with a radius of 5m are also shown in Fig. 3.

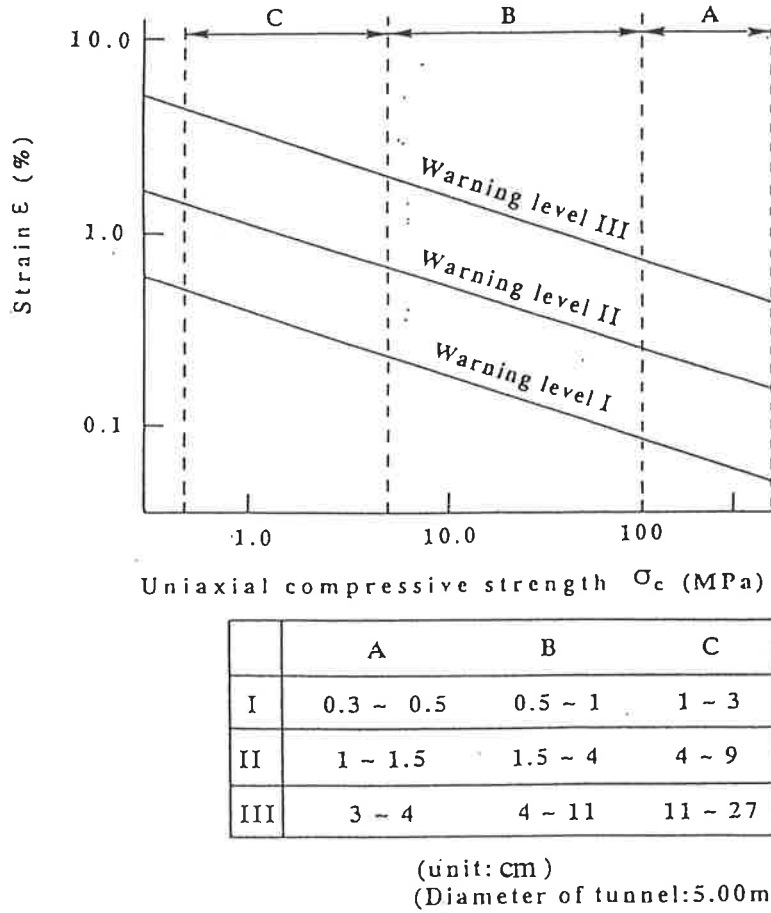


Fig. 3 Hazard warning levels for assessing the stability of tunnels

The hazard warning levels given in Fig. 3 are based on the critical strain of rocks and soils, and levels I and III correspond to the lower and upper bounds of critical strain.

The critical strain is defined as

$$\varepsilon_o = \frac{\sigma_c}{E} \quad (1)$$

where  $\sigma_c$  is uniaxial compressive strength and E is Young's modulus.

It should be noted that the critical strain is always smaller than strain at failure. Various rocks and soils were tested in the laboratory to determine the critical strain. The results are shown elsewhere.<sup>2)</sup>

It is worth mentioning that the critical strain is not much influenced by environmental factors such as confining pressure, water content, temperature, etc.<sup>3)</sup> The critical strain of rock masses is also of the same order of magnitude as that of intact rocks.<sup>4)</sup> This is simply due to the fact that the uniaxial compressive strength is influenced by the existing joints in rock masses, but Young's modulus is also influenced by the joints. Therefore, the influence is cancelled out when taking their ratio, which is the definition of critical strain as given in Eq. (1).

In order to verify the applicability of the critical strain for assessing the stability of tunnels, strains occurring around tunnels are plotted in relation to the uniaxial strength of rocks and/or soils as shown in Fig. 4. The strains shown in Fig. 4 are obtained from various tunnels, and determined from the settlement at the tunnel crown by the following equation:

$$\varepsilon = \delta_c / a \quad (2)$$

where  $\delta_c$  is the crown settlement, and "a" is the tunnel radius.

The two dotted lines correspond to the hazard warning levels I and III, given in Fig. 3. The numbers in the data indicate the kinds of problems encountered during the construction of tunnels, while the data with no numbers are those for tunnels constructed without any serious problems. The kinds of problems classified here are as follows; 1) Difficulty in maintaining tunnel face, 2) failure or cracking in shotcrete, 3) buckling of steel ribs, 4) breakage of rock bolts, 5) fall-in of roof, 6) swelling at invert, and 7) miscellaneous.

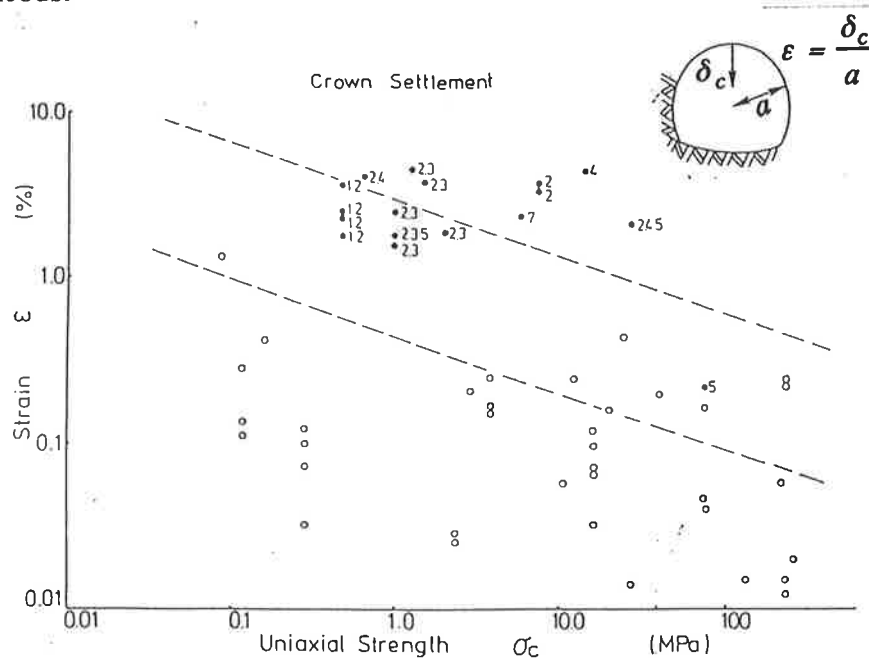


Fig. 4 Relationship between the measured strain and hazard warning levels

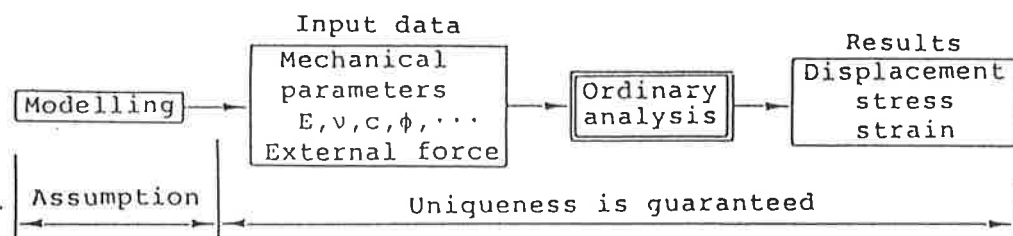
### 3. BACK ANALYSIS AND MODELLING

Back analysis is generally defined as a technique for identifying the controlling parameters of a system by analyzing its output data. In geotechnical engineering problems, mechanical constants, such as Young's modulus, Poisson's ratio, cohesion, internal friction angle, initial state of stress, etc., can usually be identified from the data of the displacements, strains and pressures measured during/after construction of the structures.

It is important, however, that a back analysis in engineering practice be capable of identifying not only the mechanical constants, but also the mechanical model itself. In ordinary analysis, a mechanical model is usually assumed or given, such that the ground is represented by a certain model such as elastic, elasto-plastic, visco-plastic, discrete block models, etc. Then the values of the mechanical constants of the models can be determined. Once all the mechanical constants are known, we can calculate displacements, strains and stresses. These results give exact values so that the uniqueness of the solution is guaranteed, at least for an assumed/given mechanical model.

In back analysis, however, we first obtain displacements, strains or pressures as a result of field measurements and the mechanical constants are then back calculated by assuming a mechanical model. It is no wonder that the back calculated values of mechanical constants depend entirely on which model we assume in back analysis. For instance, if we assume an elastic model, Young's modulus cannot be obtained, although the identical values of measured results are used. This means that the results of back analysis are basically a matter of assumption of the mechanical models representing the behaviour of soil/rocks. In other words, the uniqueness of the solution may not be guaranteed in back analysis (see Fig. 5). Therefore, in back analysis the assumption for the mechanical models is extremely important and thus more consideration and attention should be given to the modelling of soil/rocks than in ordinary analysis.

#### 1) Ordinary analysis



#### 2) Back analysis

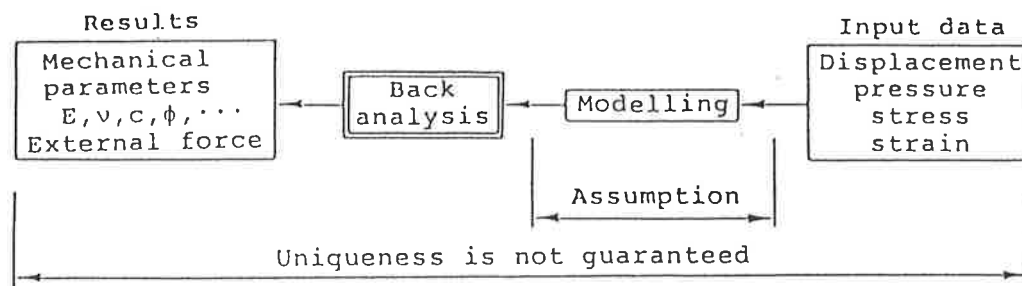


Fig. 5 Comparison between the procedures of ordinary analysis and back analysis

Considering these circumstances, we can conclude that in back analysis the mechanical model should not be assumed, but should be determined from the data of the field measurement results. In other words, a back analysis method used in geotechnical engineering practice should be one which can provide a mechanical model as well as the values of the mechanical constants involved in the model.

#### 4. PARADOX IN MODELLING IN BACK ANALYSIS

In the previous section it was emphasized that the mechanical model should not be assumed, but should be determined by back analysis of field measurement results in order to guarantee the uniqueness of solution. However, as far as engineering practice is concerned, it is not always necessary to identify the model by back analysis. Instead, the mechanical model used in back analysis for analyzing field measurement results should be the same as the one used in the ordinary analysis carried out at the design stage, no matter how the model can simulate real behaviour of the structures. This is certainly a paradox from the conclusion described in the previous section.

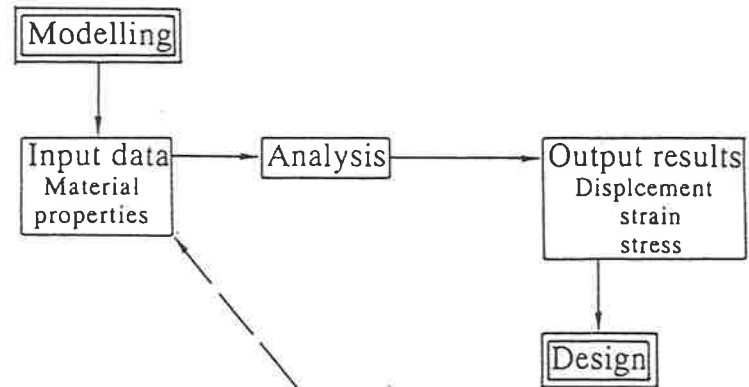
One example to demonstrate this paradox is as follows:

Let us consider the design of cut slopes. Limit equilibrium method is commonly used to assess their stability, where the cohesion and internal friction angle of rock masses as well as along the rock joints are important design parameters. When evaluating these design parameters, the factor of safety of cut slopes can be easily calculated. Field measurements are usually performed for monitoring the stability of slopes, particularly high cut slopes. Displacement measurements are commonly carried out using extensometers and inclinometers. These can provide information about the deformational behaviour of slopes. Unfortunately, however, it is extremely difficult to evaluate the warning level for the measured displacements. In addition, it is hard to assess the adequacy of the original design, because of the fact that only strength parameters like cohesion and internal friction angle are used in the design, while the field measurements provide only deformational behaviours. The displacement measurements in general provide information on the deformational characteristics of rocks. Therefore strength parameters like cohesion and internal friction angle cannot be determined directly from the results of displacement measurements. Of course it may be possible to identify the strength parameters from the measured displacements by means of back analysis. It is, however, not so easy to carry out this sort of back analysis.

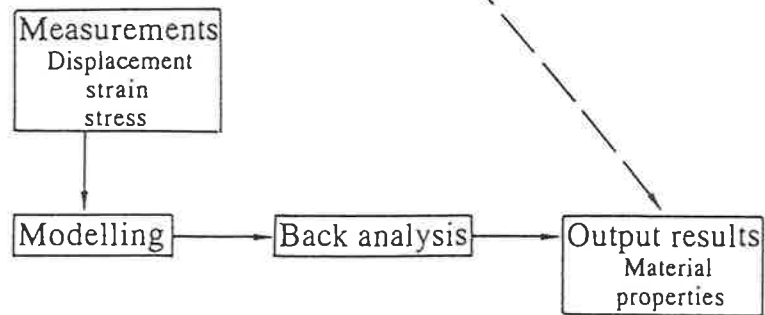
Based on the above-mentioned discussion, it is recommended that the same mechanical model should be used both in the ordinary analysis adopted in design and in the back analysis used for interpreting field measurement results. In other words, we should use the identical mechanical model in both the design and monitoring stages, though the real behaviour of slopes can be better simulated by a different model from the one adopted (see Fig. 6).

## Design vs. Monitoring

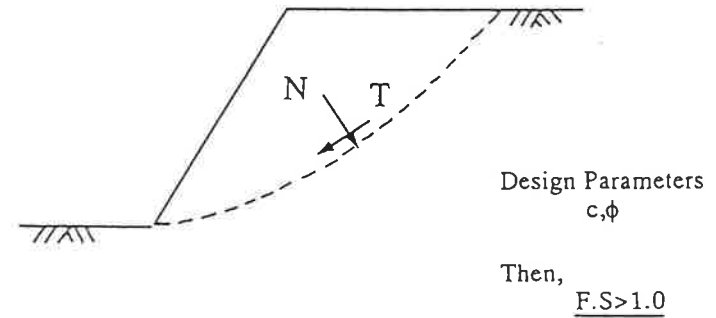
### (1) Design



### (2) Monitoring



### Design



### Monitoring

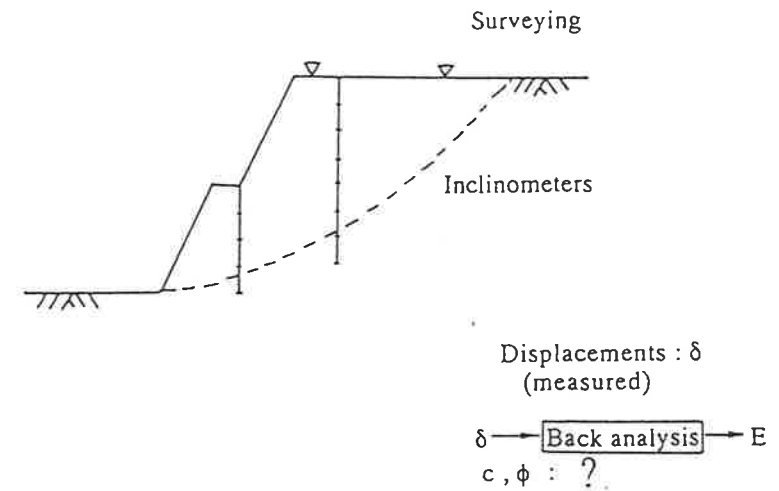


Fig. 6 Relationship between design and monitoring procedures in cut slopes

## 5. CONCLUSIONS

- 1) The observational method is a very promising way to achieve the rational design of rock structures. In this method, however, a crucial problem is how to interpret the results of observation and field measurements.
- 2) For monitoring the stability of structures, the hazard warning levels are extremely important, and they must be determined prior to construction, so that the measured values can be assessed immediately after taking the measurements.
- 3) In order to reevaluate the design parameters, back analysis is a very powerful tool. In back analysis, however, a uniqueness of solution is usually not guaranteed. To overcome this problem it is possible to identify the mechanical model as well as material constants by back analysis of the measurement results. However, as far as engineering practice is concerned, it is not always recommended to back analyze the mechanical model. Instead, it is recommended that we should use the identical mechanical model both in the ordinary analysis carried out at the design stage and in back analysis used in interpreting the field measurement results during the construction period, though a uniqueness is not guaranteed.

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# INVESTIGATION OF THE ROCK MASS STRUCTURE AND CHARACTERISTICS IN FRONT OF THE FACE USING TSP, TUNNEL SEISMIC PREDICTION, AT HALLANDSÅSEN PROJECT

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## INTRODUCTION

The highspeed railway tunnel through the Hallandsåsen is part of the extension of the double track system between Gothenburg and Malmö which is under construction by Banverket. The tunnel tracks through the crystalline formation of the Hallandsåsen will be constructed as two separate tunnels with a single track in each. The distance between the two parallel tunnels varies between 20 and 30 m. The total length of each tunnel is approximately 8600 m with a main orientation of approximate N20°E. The rock cover is in general more than 100 m.

Hallandsåsen is a Precambrian basement horst, which rises approximately 150 m above the surrounding sedimentary lower landscape. The horst is oriented in NW-SE direction and dominated by Precambrian Gneisses with lenses or bodies of amphibolite. These rock types are intruded by younger dolerite dykes with a main orientation NW-SE. The horst is limited to the north and south by fault zones. Faults are also expected to cross the tunnel line at several locations with a main orientation NW-SE and NE-SW, generally dipping steeply.

The geological pre-investigation has indicated several weakness zones and areas of rock with poorer quality. During the excavation performed by the contractor constructional problems have occurred. One of the reasons has been bad rock quality. In order to better investigate weakness zones in front of the face, a test was made at the southern portal region of the Hallandsåsen tunnel in July 1995, taking advantage of a new method, the TSP (Tunnel Seismic Prediction). The objective of the nondestructive seismic investigations was

- to investigate the capability of the TSP method at local conditions
- to provide a prediction of the rock conditions ahead of the current tunnel face

The primary target of one of the TSP surveys was to locate a zone of weak rock, which was known from surface investigations to be in the order of 40 m ahead of the face. The rock types excavated were gneiss and amphibolite and other hard intrusive rock types, locally strongly tectonized.

A total of 4 TSP surveys were performed, two at face locations and two at an intermediate position along the west tunnel (Fig.1).

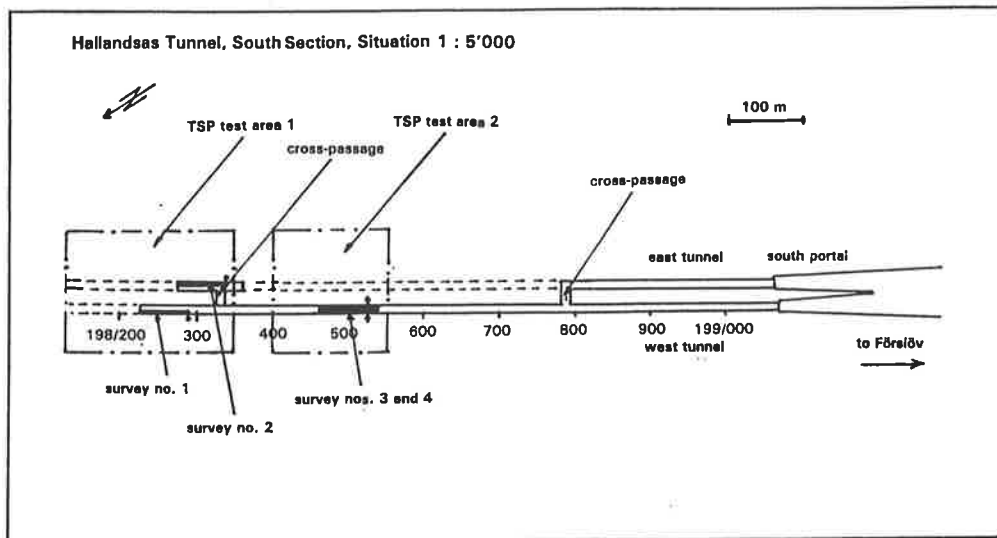


Fig.1: Situation with location of TSP surveys at Hallandsås tunnel, south portal section

#### TSP-METHOD

Tunnel Seismic Prediction, TSP® is a novel, very fast, robust and non-destructive reflection seismic technique. It is custom-built to assist construction planning and advance in tunnelling operations, underground mines, caverns and underground repositories. Its aim is rapid forward geological and geotechnical prediction in 3-D space ahead of and around drive.

In common with all reflection seismic methods, TSP employs the principle of echo sounding. Seismic (acoustic) waves are generated by small explosions at specific shotpoint locations and picked up by receivers. When a seismic wave hits a boundary marking a significant change in rock strength (e.g. changes in physical properties, rock type, fault zones, fracture zones), part of the incoming wave energy is reflected back at diffraction points (Fig. 2, top). The travel time of the reflection signal is proportional to the distance to the boundary and so provides a direct measure of this. Normally, the measurement layout consists of approximately 24 shotpoints lined up along the right or left tunnel wall, depending upon closest proximity to expected geological boundaries, and a receiver in each tunnel wall (Fig. 2). The shotpoint line-up serves mainly to determine the geometries of the boundaries relative to the tunnel axis. Receivers in either wall are used to investigate the space sectors to the right and left of the tunnel axis, henceforth referred to as TA (Fig. 2, left centre).

Each receiver system consists of groups of sensors (accelerometers) with components oriented along and perpendicular to the TA. The components serve to discriminate between reflections arriving from boundaries inclined steeply to the TA, and those arriving from boundaries inclined parallel, or at low angles to the TA (Fig. 2, top). Combining all orientations of the receiver systems in the left and right wall separates a total of four sectors for the TSP 202 investigation ahead - 1A left top, 1B left bottom, 2A right top and 2B right bottom (Fig. 2, left centre).

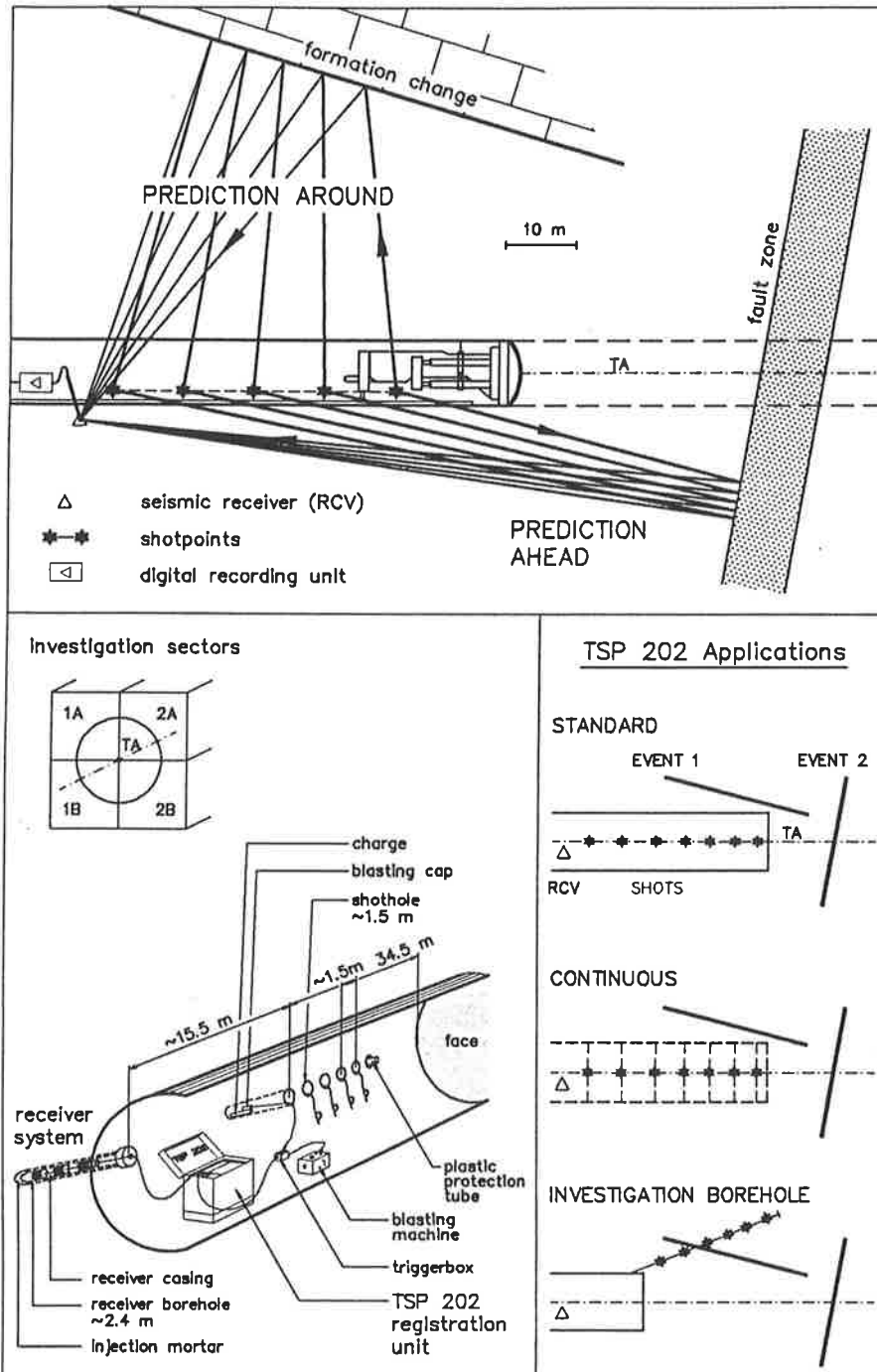


Fig.2: Principle of TSP investigation

TSP's versatility allows for different applications (Fig. 2 bottom right). Apart from periodic surveys at tunnel face positions (standard application), shooting in investigation boreholes increases the investigation range and permits precise definition of geometries of boundaries intersected by the boreholes. However, TSP's greatest value derives from continuous use accompanying TBM drive or conventional drive. Survey costs are sufficiently competitive to make TSP feasible for such continuous monitoring.

To economise on survey time, all preparatory work for a TSP measurement (drilling small diameter boreholes, cementing two metal receiver casings) can be integrated into normal tunnelling operations. Setting up for measurement includes insertion of the receiver system, charging of shot holes and a functional test. This work can accompany normal tunnelling operation without undue interference and takes approximately 1 hour. Seismic registration involves firing and recording each shot individually and takes approximately 45 minutes. Including equipment clear-up, the entire measurement cycle takes approximately 2 hours.

Data processing aims to image the reflecting boundaries and to locate them ahead of and around the face area. It is done using the specially developed TSP 202 software. This is an interactive programme supplied with every TSP 202 system and runs on standard PCs. Designed with ease of use in mind, the software is highly automated and requires minimal training, such that it can be quickly mastered by staff at construction sites.

Processing follows a systematic sequence of steps. After input of all survey data into the database, the appropriate investigation sector is selected for evaluation. Wavefield processing serves to separate the direct and reflected wavefields and to transform the energy of the reflection signal onto an evaluation plane using the diffraction stack migration method. This plane, the diffraction stack plane (DFS), is defined by the axis of the shotpoints parallel to the TA and the diagonal through the relevant investigation sector.

The diffraction stack process itself migrates the reflection signals back to their source points, i.e. onto the seismically imaged portion of a boundary plane ahead of or around the recording environment (e.g. measurement setup). Conversion of travel times to distances is normally done using the average seismic velocity, as derived from travel times of the direct wave (and assumed to be valid for the area ahead). For special cases the average velocity can be fine-tuned using focus criteria on the diffraction stack result.

The diffraction stack method itself is based upon Huyghen's Theory, whereby every single point within the rock formation that is 'hit' by a seismic wave is, in turn, capable of re-emitting a seismic wave. As shown by the diffraction stack diagrams of Fig. 3 for instance, the strength of an echo signal is represented by points resp. small circles whose radii are proportional to the energy of the echo signal. This, in turn, is a function of the size of the contrast between the rock mechanic parameters at the boundary plane. For example, a boundary representing a clear transition from 'softer' to 'harder' rock is displayed as a linear alignment of grey marked circles.

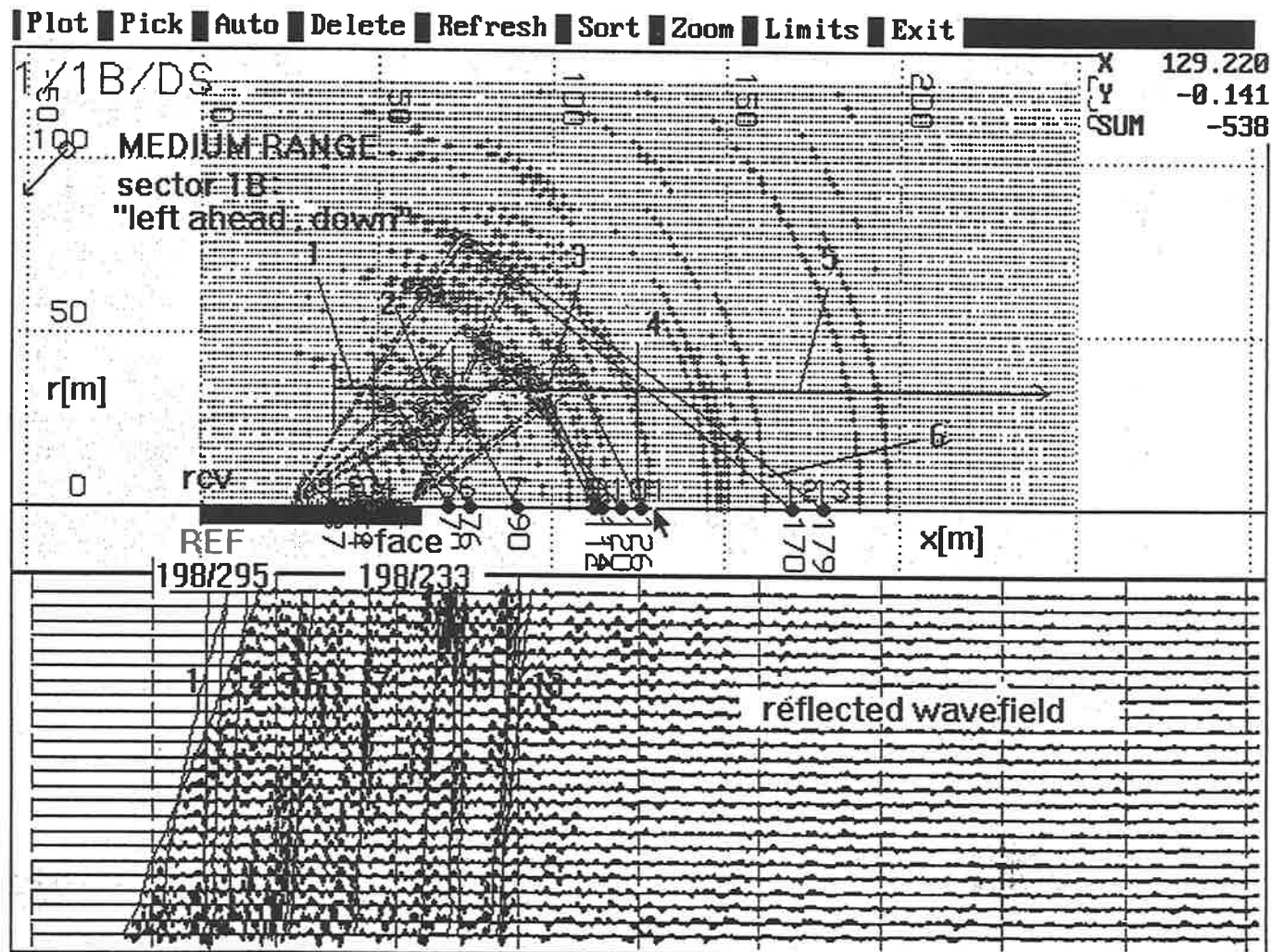


Fig. 3: Diffraction stack representation (DFS- image) of survey 1 in the western tunnel

For each of the four exploration sectors (1A, 1B, 2A, and 2B) such a diffraction stack representation is generated and interpreted as described below. In each case, a diffraction stack plane, i.e. view plane is disposed almost diagonally within the given exploration sector, bearing in mind that both for representation purposes and for simplification of the interpretation, reflection points from within the entire exploration sector are projected onto this plane.

During diffraction stack representation of the exploration sector under consideration, boundary planes ("events") in the rock show up as linear clusters of reflection points with significantly higher reflection energies than surrounding points (Fig. 3). The interpreter selects points with stronger echo signal energy interactively and a line is fitted through them automatically. This line intersects the tunnel axis at a so called space angle  $\gamma$  and at a certain distance from the recording array. This defines the point of intersection between the boundary plane and the tunnel axis.

For transfer of boundary layers determined interactively from the diffraction stack representation to a vertical and horizontal section, the point of intersection with the tunnel axis is used as starting point. Then, however, the dip angle  $\alpha$  and the strike angle  $\beta$  (relative to tunnel axis) must be calculated from the general space angle  $\gamma$ , having in advance specified a dominant direction of strike relative to the tunnel axis as a starting parameter. However the critical factor in the prognosis derived from seismic investigation ahead by TSP is the distance along the tunnel at which a boundary layer can be expected to intersect. This value is determined directly from the interactive diffraction stack evaluation and is independent of the specified direction of strike.

Assuming the reflection energy is strongest in the sector containing the plotted boundary plane, simultaneous consideration of the diffraction stack interpretations of all four sectors serves to improve determination of the geometry of the boundary layer in space.

For prediction ahead, a full interactive interpretation is usually possible separately both for the primary, compressional P-wave and for the secondary S-wave (shear wave), whose velocity constitutes a valuable additional investigation parameter. Such dual interpretation yields improved overall interpretation confidence and accuracy from the additional redundancy of the S-wave data and has the potential to improve discrimination of water-bearing fault/fracture zones and of structural variation in rock masses, such as cleavage and fracturing intensity.

Following the interactive interpretation, the selected boundaries can be presented in predictive vertical and horizontal sections with the tunnel geometry, recalculated to the tunnel survey grid (Fig.4). These sections summarize the anticipated entry and exit points of reflecting boundaries (changes in rock strength) in the advancing tunnel. This result can be complemented by processing other desirable investigation sectors in the same way .

The final product is the engineer's summary as shown for example in Fig. 5. This is a synthesized spatial interpretation of the results of all of the investigation sectors considered (Fig. 2, bottom left). Furthermore, for a known, uniform rock mass sampled by

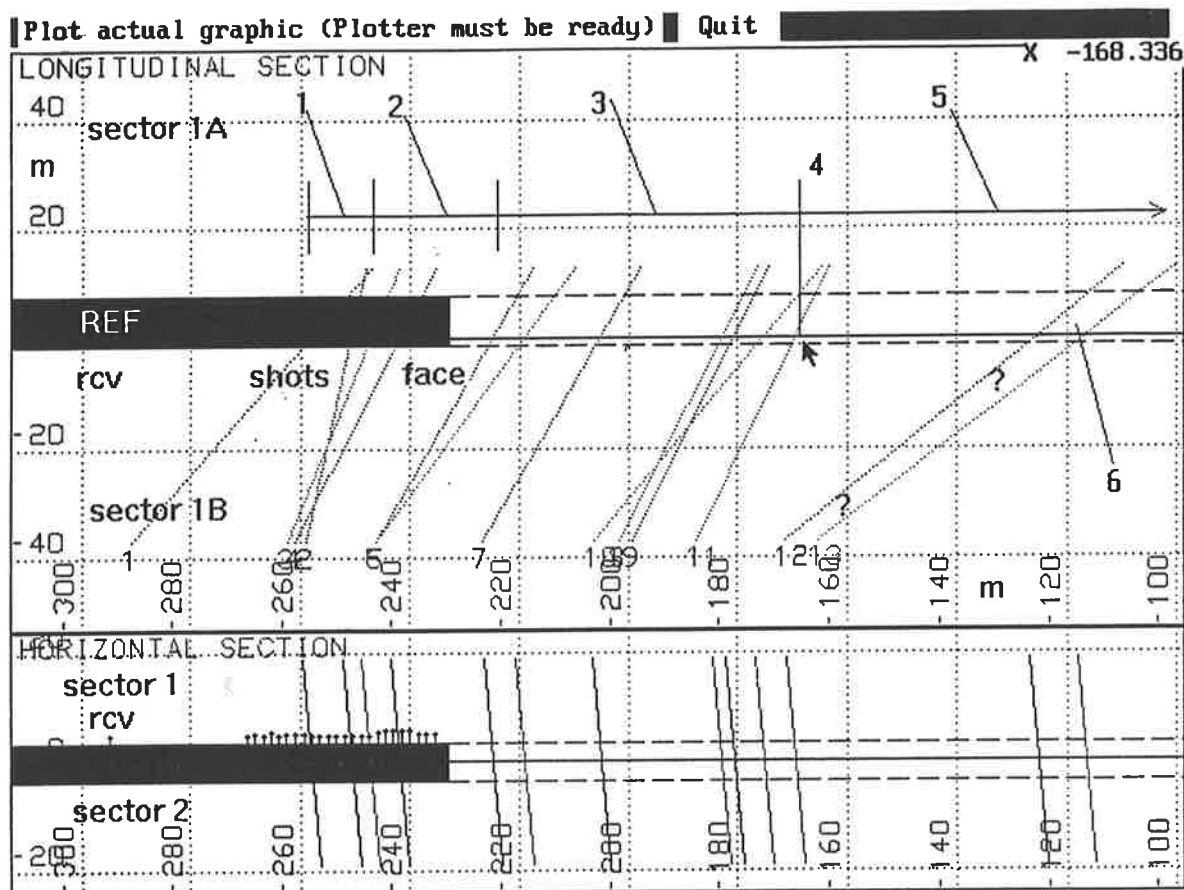


**Fig.4:** TSP result as longitudinal and horizontal section showing selected events with their orientation in respect to the tunnel axis

TSP, velocities of primary and secondary seismic waves estimated from seismic processing permit determination of important rock mechanical parameters, such as Young's Modulus and the Poisson's ratio. These assist the tunnel engineer in planning construction measures and represent a valuable tool for monitoring changes in rock mechanical properties in continuous TSP profiling (cf. Fig. 2, bottom right).

The TSP 202 software allows a comparatively rapid, straight-forward interpretation on site. Typically, a quick look interpretation is available after 2-4 hours. A comprehensive interpretation and report can be generated in 4-8 hours.

## RESULTS

### General

The acquisition of the seismic data for the 4 surveys at Hallandsåsen was successful. Thanks to the good organization on site, all, preparation, the 4 TSP measurements and a quick look evaluation was performed within 2 days.

The TSP investigations at the faces of both tunnels (survey nos. 1 and 2, location see Fig.1) yielded very interesting results and are further discussed below. The TSP investigations at the intermediate tunnel position (survey nos. 3 and 4) provided quite a detailed image of the complex geological conditions of that area. Fracture zones could be well located and correlated with the geological profile resp. the engineers log. Also the gneiss-amphibolite boundary in the roof area running almost parallel to the tunnel axis produced a significant image in the seismic section and could be monitored quite well by the TSP.

The rock mechanic parameters (Young's modulus and Poisson's ratio) as derived from the direct P- and S-wave arrivals indicated that the quality of the crystalline rock is generally relatively good. However the value of the Poisson's ratio is significantly higher than normal so one can expect a reduced shear strength probably due to local fracturing of the rock mass.

### TSP Prediction from surveys 1 and 2

As seen in Fig.1, survey no. 1 was performed with the face position located at station 198 / 233 and the reference position at 198 / 295 in the west tunnel. The rock was amphibolite and the seismic P-wave velocity determined by the direct wave was around 6340 m/s. Because most structures close to the face are dipping steeply, mainly towards the portal, only the investigation sector 1B ("left ahead, down") is discussed here.

The seismic velocity as determined in the measurement section is fairly high according to the amphibolite rock type excavated. The interpretation of the near range sector fits the velocity of 6340 m/s, but for the medium and far range the average velocity has to be reduced to values of 5'700 m/s and 5'500 m/s respectively (average velocity of all surveys approx. 5'670 m/s).

For interpretation purposes the zones as discussed in the following could be discriminated using the diffraction stack presentation (Fig.3 resp. 4) of the reflected wavefield .

Zone 1 as seen in the diffraction stack display starts with the boundary 1 (marked black) at about station 258 and ends with boundary 3 (marked grey). Boundary 1 corresponds to the beginning of a fractured zone (no.1) with probably higher fracture intensity close to the boundary 2 (marked black) at about station 251. Boundary 2 shows a good reflectivity and seems to correlate well with a decrease of fracturing at about station 247.

Zone 2 can be characterized according to the seismic result as a zone with slightly

better rock quality, although intersected probably by small fracture zones (see below). Ahead of boundary 3 (marked grey) the rock quality seems to improve slightly and it can be deduced from the diffraction stack result (Fig.3) that this situation would extend to boundary 5 (marked black) at about station 225. However the corresponding area marked as zone 2 is also intersected by a small fracture zone at about station 242, well imaged in the diffraction stack display as boundary 4 (marked black).

Zone 3 can be characterized as a more inhomogeneous area than zone 2. Ahead of boundary 5 (marked black) an area with difficult boundary character is investigated reaching up to boundary 11. It is concluded that the rock in this zone will be relatively heterogeneous, i.e. with several discontinuities of different structural geometry. The rock quality is assumed to be locally poorer than for instance ahead of boundary 7 (marked black) and boundary 9 (marked black). At the end of zone 3, an accumulation of boundaries (marked grey, nos. 8, 10, 11) indicate a gradual transition to harder rock. This can be interpreted as the end of an area with generally poorer rock quality at about station 169 and is marked with a small vertical line and the number "4".

Line 4 corresponds to the end of zone 3 (s. above)

Zone 5 is characterized as a more homogeneous section following zone 3. Ahead of the boundary 11 (marked grey) at about station 169 the diffraction stack display is different, showing only diffraction points with lower reflection energy. Considering the high reflection intensity in front of mark "4" (i.e. zone 3), one can conclude that a zone of improved rock quality should follow.

Line 6 points to the boundaries 12 (marked black) and 13 (marked grey) that may cross zone 5 as a smaller fracture zone at about stations 125 - 116. Due to the low inclination the boundaries are imaged about 100 m back from their intersection point and accordingly the intersection figures represent only an estimate.

The result of the second survey agrees well with the general prognosis based on surface investigations. There, the end of the zone with poorer rock quality (line marked with "4") is approx. at station 170, about 1 m in front of the intersection of the seismic prognosis at about station 169.

### **Comparison between TSP and geological situation**

The correlation of the TSP measurements, geology as mapped and rock quality according to Q-index is generally good.

During the autumn 1995 both the western and the eastern tunnels have been excavated through the area covered by the TSP measurements. According to the contractor; Mr. Ö. Wolff at SIAB, zone no. 3 caused excavation problems. A rock fall occurred in the roof of the western tunnel at about station 198/206 where the weak zone starts in Fig 5. The Q-index in the zone has been determined as low as  $Q = 0.25$ . The real location of the zone showed a discrepancy of about 3 metres compared with the prognosis of the TSP. The zone orientation however is in agreement with the result of the seismic result.

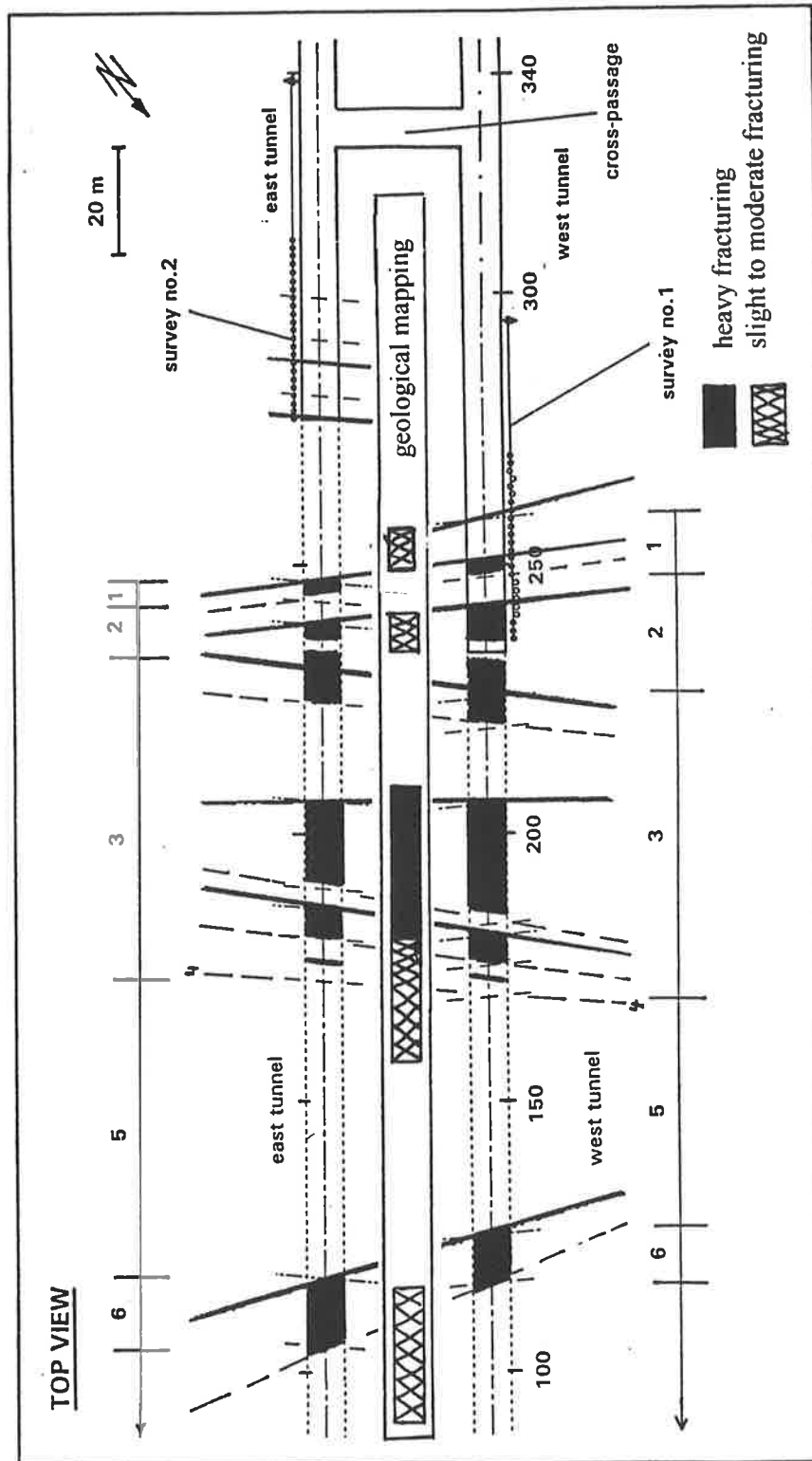


**Fig. 5:** Combined evaluation of TSP survey nos 1 and 2 with planar boundaries and interpreted rock zones in comparison with geological mapping after excavation

The geological mapping also showed that the zone correlates with a contact between amphibolite and gneiss while the boundary area is filled with clay. Its dip is about 30 degrees in driving direction and extends from station 198/205 to 198/180. From 198/180 to about station 156, the rock is fractured and blocky, but with decreasing fracture intensity in driving direction. The rock gets gradually better from about station 198/175 ( $Q=0.89$ ) and reaches a  $Q$ -value of 5 at station 198/152. For the area discussed above, the geology investigated during excavation showed a generally good correlation with the TSP prediction.

The zones no 1 and 2 did not cause any larger driving problem. However, the section including stations 198/248-256 is fractured with regular spaced cross cutting joints in 1-2 meter intervals while the section of stations 198/236 - 242 has similar fractures but with uneven spacing. In this case the TSP prediction was a bit too pessimistic.

The zone no. 6 did not cause excavation problems at all. Fractured rock with joints of mixed orientation was encountered in the section of stations 198/090 - 115. Essentially three sets of different orientation were found, an orthogonal one, a second with mainly shallow dip towards the TA and a third running almost parallel to the tunnel axis as found in section 198/090 - 080. One of the larger fractures crossing contained clay. The TSP predicted the fractured area about 5 m earlier than actually encountered, but this is a relatively small tolerance which must be taken into account when a complex geological structural situation is present.

## CONCLUSION

The results show that TSP investigations are a valuable tool for acquiring to get additional information about the conditions of the rock mass to be excavated. Data acquisition does not delay or effect the tunneling operation itself and can be performed continuously or as needed. The software provides very fast data processing and the result is available to the interpreter already a 1 or 2 hours after the measurement as demonstrated on site.

The interpretation of the boundaries imaged by the TSP method is straight forward as long as the structural conditions are not too complicated. In crystalline rock, however, the geometries of discontinuities (folds, faults, fractures etc.) can become very complex. In this case the interpretation of the TSP results requires a very good understanding of local geological conditions and in which manner the features are imaged by the seismic investigation.

For an appropriate and efficient tunnel driving predictive information gained by TSP investigation can be used to guide appropriate measures in tunnel construction. In order to provide the required basic information we recommend the use of the TSP at the site as an additional predictive tool for regular investigation.

As with every geophysical method, the success of the TSP interpretation is strongly dependent on multidisciplinary cooperations between all technical departments on site.

## SAMMANFATTNING

Tunneln genom Hallandsåsen är en del av Banverkets utbyggnad för höghastighetståg från Göteborg till Malmö. Utbyggnaden görs med dubbelspår. Genom Hallandsåsen går järnvägen i två parallella enkelspårstunnlar. Dessa har en längd av ca 8 600 m vardera och ligger 20-30 m separerade från varandra. Tunnelorienteringen är ca N20ö. Bergtäckningen varierar men är i huvudsak minst 100 m.

I förundersökningen lokaliserades ett antal svaghetszoner i bergmassan. Dessa har skapat bergtekniska problem i samband med pågående tunneldrift. För att bättre lokalisera och beskriva dessa avsnitt framför tunnelfronten har en ny metod, TSP 202, Tunnel Seismic Prediction, testats i södra tunnelpåslaget.

Syftet med testningen var att undersöka om en definierad svaghetszon kunde lokaliseras framför fronten. Zonen var förutsagd att ligga ca 40 m framför fronten i västra tunneln och ca 80 m i den östra. Zonens bergkvalitet var tolkad att ligga mellan  $Q = 0.1-1$ .

Undersökningen omfattar fyra mätsektioner placerade enligt Figur 1, varav två utfördes framför tunnefronten en i vardera tunneln och två ytterligare i västra tunneln.

I artikeln presenteras mätningen i östra tunnelfronten.

TSP 202 är en snabb, icke förstörande reflektionsseismisk teknik, som syftar till att prognostisera bergmassans struktur och egenskaper 3-dimensionellt framför tunnelfronten. Utrustningen består av 12 superkänsliga accelerometrar sammansatta till bi-axiella sensorelement samt en registreringsenhet.

Vid mätning installeras accelerometerna i speciella borrhål ca 30 m bakom sista skuthålet enligt Figur 2. I normaluppställningen görs succesiva skjutningar i ena tunnelväggen i 24 skuthål.

Databearbetning efter fältmätningen omfattar ett antal systematiska steg som leder fram till ett "diffraction stack diagram", DFS enligt Figur 3. En gränssyta som gett ett seismiskt eko projiceras på DFS-ytan som en punkt (+). En kluster av punkter bildas när gränssnittsytan tex en svaghetszon, ger flera ekon. En linje anpassas till klustern. Linjen kommer att skära tunnels längdaxel. Skärningspunkten mellan denna linje och tunneln anger avståndet i meter till läget för accelerometerna.

Därefter bearbetas data för korrekt orientering av gränssnittsytan i förhållande till tunnelaxeln enligt Figur 4.

Resultaten av mätningarna har utvärderats i tre områden nämligen:

- Närområdet, dvs närmast skjutområdet
- 150-200 m framför tunnelfronten
- 150-600 m framför tunnelfronten

Analyserat område 150-200 m framför tunnelfronten visas i Figur 5. Tre avsnitt med förändring i bergkvalitet har registrerats, nämligen zon 1-2, 3 och 6.

Den geologiska uppföljningen efter bergguttaget har gett följande resultat:

#### Zon 1-2.

Avsnittet 198/236-242 samt 198/248-256 har ökad sprickighet jämfört med omgivande bergmassa.

#### Zon 3.

Avsnittet har låg bergkvalitet motsvarande  $Q=0.25$ . Zonen är komplext uppbyggd med omväxlande vertikala och flacka sprickor. Zonområdet omfattar avsnittet 198/180-205 medan avsnittet 198/156-180 visar en omfattande uppsprickning.

Bergmassans kvalitet ökar i tunneldrivningsriktningen och motsvarar vid 198/152  $Q=5$ .

#### Zon 6.

Berggrunden är uppsprucken i tre sprickgrupper mellan 198/090-115, men några svaghetszoner har ej observerats. En lerfylld spricka skär tvärs tunneln. I väggarna finns ett antal sub-horisontella slag.

Enligt Ö Wolff, SIAB, vållade zon 3 stora drivningstekniska problem vid passagen. Ett utfall i taket inträffade i början av zonen i västra tunneln. Zonens läge predikterades med ca 3 m noggrannhet med TSP.

Zon 1-2 och 6 medförde inga drivningstekniska problem.



# UPPDATERINGSBARA BERGPROGNOSER - TILLÄMPNING AV EN NY GEOSTATISTISK METOD PÅ TBM-TUNNELN I ÄSPÖLABORATORIET

## Updating of rock predictions - Application of a new geostatistical method on the TBM-tunnel in the Äspö Hard Rock Laboratory

*Lars Rosén, CTH*

*Gunnar Gustafson, CTH*

### 1 INTRODUCTION

Sweden has decided to dispose its high-level nuclear waste between 400-700 meters below ground surface in crystalline bedrock, in accordance with the so called KBS-3 concept (SKBF-3, 1983). The Swedish Nuclear Fuel and Waste Management Company (SKB) is commissioned to find a suitable site for the repository. The issue of siting and establishing a nuclear waste repository with respect to geoscientific factors is to a great extent a hydrogeological problem, since the primary process through which the waste can be brought back to the biosphere is ground-water transport. The purpose of the siting process is to find a repository site where a required amount of fuel canisters can be safely disposed of, which requires specific hydrogeological conditions. It is therefore appropriate to direct the hydrogeological evaluations at the issue of canister positioning during all stages of the evaluation process. This requires evaluations to be made with respect to compliance levels for canister positioning.

The maximum number of canisters on a specific length of a nuclear waste repository tunnel is given with respect to practical conditions, such as tunnel-dimensions and drilling equipment. An important issue is how well this maximum capacity can be used, with respect to hydrogeology, stability, and geology. A measure of the capacity relative to the maximum capacity may be referred to as a Positioning Index (PI). The PI is defined for a given rock volume and a given purpose as the probability that the maximum number of positions can be used, given the approved repository design.

A Bayesian Markov Geostatistical Model (BayMar) was developed and applied for PI-calculations with respect to lithology, hydraulic conductivity, and RQD along a stretch of a full face boring (TBM) tunnel in the Äspö Hard Rock Laboratory (HRL) in southeastern Sweden (Figure 1). Traditionally, the term 'geostatistics' is associated with the kriging methods based on regionalized variable theory by Matheron (1963). In the BayMar methodology it is used in the more general sense described by Henley (1980). The methodology differs from conventional geostatistical methods in (1) being non-parametric to be able to handle information with complex statistical distributions, which is often the case for classified data, e.g. lithology, and (2) using Bayesian statistics for

updating of prior estimates with respect to new information. Data are generally very sparse at early stages, becoming more and more abundant as the investigations get more detailed. Professional judgments thus play an important role for prior estimations at early stages and Bayesian statistics provides a formal way for integrating these with hard data.

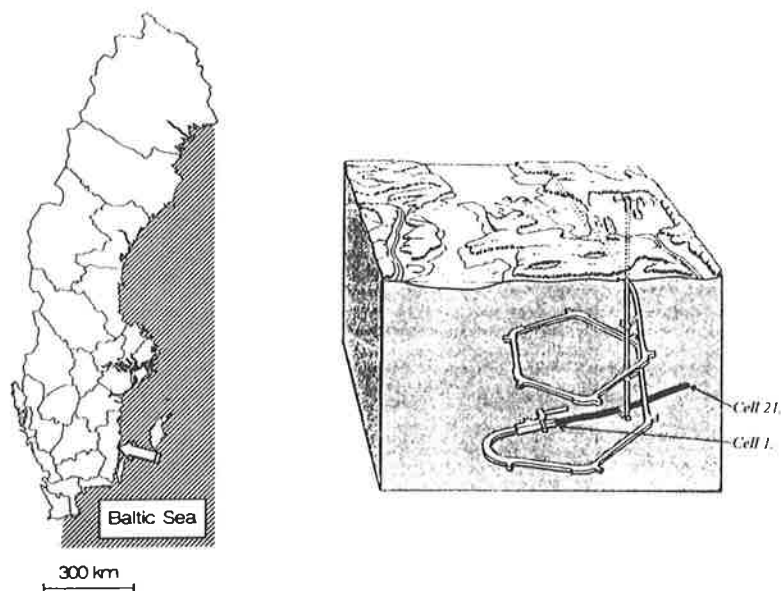


**Figure 1.** Overview of the Äspö Hard Rock Laboratory facility (after Äspö HRL, 1994). The TBM-tunnel is marked in black.

## 2 POSITIONING INDEX (PI)

The PI is defined for a given rock volume,  $V [L^3]$ , and a given purpose,  $U$ , as the probability that the maximum number of positions,  $J$ , can be used, given the approved repository design:

$$PI(V, U) = P(J | V, U) \quad (1)$$

The PI-value is equal to the expected value for an indicator,  $E(I)$ , that the maximum number of positions in a volume can be used ( $I=1$ ). The PI-value is equal to reliability in terms of engineering design. The PI-approach is non-parametric because it does not require the indicator to have any specific statistical distribution. For fuel canisters in a nuclear waste repository, the maximum number of positions,  $N_{max}$ , along the repository tunnel is known and determined by factors such as tunnel dimensions and tunnel layout. The maximum number of canisters,  $N$ , that can be placed in the tunnel with respect to the PI is then:

$$N = N_{\max} PI \quad (2)$$

The PI-estimations have to be made with respect to the scale of the specific problem at hand. In the early stages of an investigation program for a nuclear waste repository, estimations of rock volumes the size of entire units of the repository may be relevant, whereas in the last stage of the investigation program the rock volumes need to be roughly the size of a fuel canister. Since the philosophy behind the PI calculations is that they should be based on existing data and knowledge at each level of investigation, it is suitable to perform the probability estimations in a Bayesian context. Bayesian statistics differs from classical statistics by not requiring that the probability assessments be based on measured data. The Bayesian statistics can be described as:

$$P[A_i | B] = P[A_i] \{ \text{additional knowledge} \} \quad (3)$$

which indicates how the prior probabilities estimated before an experiment,  $A_i (i=1, \dots, n)$ , should be modified by the evidence of a new outcome,  $B$ , to produce posterior probabilities.

The prior estimation of  $A_i$  can be based on professional judgment from past experiences only, or on experience and existing data. As new information becomes available the prior probability estimate is updated to a posterior estimate, with a higher degree of certainty. When data are sparse, the Bayesian statistical estimates might be quite different from classical statistical estimates of the same dataset, but as additional data become available the two different types of estimates converge. The subjective input from the evaluator thus affects the results until large numbers of measurements have been taken. Bayesian statistics allows the evaluator to make a probability assessment, and thereby also a decision, whatever the level of uncertainty. It also fits nicely into geological and hydrogeological problems, since subjective interpretations and experience plays such an important role here. The possibilities and limitations of Bayesian statistics were described by Lee (1989) and Freeze *et al* (1990).

In accordance with Bayesian statistics the prior estimates of the PI are updated each time new information becomes available. When the tunnel has been completed and a choice whether to place a canister or not has to be made, the PI for this specific position is either 0 or 1.

The following set of key parameters for PI calculations with respect to canister positioning have primarily been suggested within the Group for Nearfield Design at SKB: (1) lithology, (2) groundwater inflow to the deposition hole, (3) construction stability, (4) long term stability, and (5) thermal capacity of the rock. A compliance level is assigned to each parameter that must not be exceeded for any of the parameters in order to make a specific canister position acceptable.

A Bayesian Markov Geostatistical Model (BayMar) was developed within the SKB siting program as a tool for decision support and described by Rosén and Gustafson (in print). The model was developed to comply with the approach for geoscientific classification in the siting process suggested by Rosén and Gustafson (1993). The development was partly based on the work on non-parametric geostatistics by Henley (1980). The purpose of the model is to generate probabilities for geological properties at points where no samples have been taken. The basic requirements is that the model should be able to (1) handle non-numerical classified data, e.g. lithology, (2) consider professional judgments based on past experiences of the evaluator, and (3) measure the uncertainties of the results.

The Bayesian Markov model uses transitional Markov-chain analysis for describing the spatial relationships of data. Consider the situation described in Figure 2, where the objective is to estimate the probabilities for finding states a, b, and c at the unknown point A. At points labeled B the distributions of the states are known.

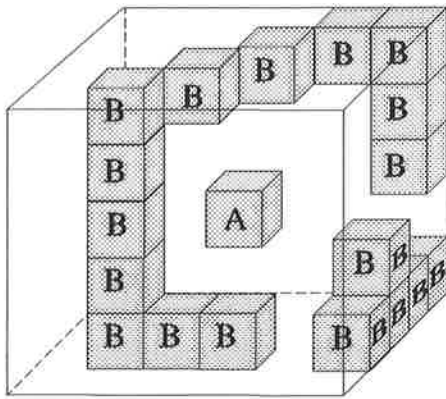


**Figure 2.** An unknown point, A, to be estimated from surrounding known points B.

A transition probability matrix,  $\bar{M}_1$ , is constructed based on the transition frequencies between mutually exclusive states a, b, and c, e.g. lithologies, when observations are taken along equally spaced intervals over the region of interest:

$$\bar{M}_1 = \begin{bmatrix} p_{aa} & p_{ab} & p_{ac} \\ p_{ba} & p_{bb} & p_{bc} \\ p_{ca} & p_{cb} & p_{cc} \end{bmatrix} \quad (4)$$

where, for example,  $p_{ab}$  is the conditional probability of finding state b if the starting point is in state a and moving one unit distance. Every time  $\bar{M}_1$  is powered, the diagonal terms describe the probabilities to remain in a specific state when moving one step. The diagonal terms of the transition probability matrix therefore describe the state influence lengths for the mutually exclusive states. The state influence length is defined as the maximum distance for statistical dependence of the state, i.e. a correlation length, with respect to a given probability level. In accordance with the Bayesian approach, the transition probability matrix can be developed from measurements and/or professional judgments. If the parameter of interest is anisotropic,  $\bar{M}_1$  will exhibit directional dependence.

In addition to the spatial dependence characteristics, the transition probability matrix holds information regarding the size, shape, and orientation of each specific state. For a specific probability level, C, the body influence length,  $\lambda_i$ , can be calculated directly from the diagonal term,  $p_{ii}$  ( $i=1, \dots, n$  states), of  $\bar{M}_1$  as:

$$\lambda_i = \frac{\log C}{\log p_{ii}} \Delta \quad (5)$$

where  $\Delta$  is the step length, or cell side length [L] in a model grid. The body influence length is defined as the radius of a body of a specific state, with respect to a given probability level. For example, the influence lengths for the 50% probability level describe a median body. The body influence lengths can be plotted in a polar plot for different probability levels in each direction, describing size, shape, and orientation of the state (Figure 2).

To construct the transition probability matrices, some assumptions about the geologic processes and the geometry of the properties need to be made. The following questions are addressed: (1) what is, in a statistical sense, the size, shape and orientation of the various states?; (2) what transitions are not possible?; and (3) what are the approximate probabilities for possible transitions?

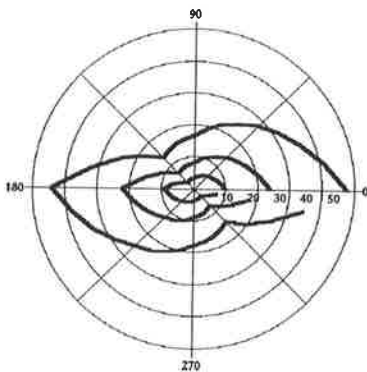


**Figure 2.** Polar plot showing size, shape, and orientation of a state as calculated from the diagonal elements of the transition probability index for different probability levels, 25% (outer), 50% (middle) and 75% (inner). The example is fine-grained granite in the Äspö area. Length scale in meters.

Each cell in the model is assigned a probability vector,  $\bar{P}$ , with elements  $p_i$  describing the probabilities for each state to occur in the particular cell:

$$\bar{P} = \begin{bmatrix} p_a \\ p_b \\ p_c \end{bmatrix} \quad (6)$$

At an unknown cell, A,  $\bar{P}(A)$  is a prior estimation of the probability vector based on professional judgments and existing data. At a known cell, B, where a measurement has been taken, the probability vector is  $\bar{P}(B)$  and describes the measured distribution of states in the cell. Using the information given by a known cell, B, and the statistical dependence between states,  $\bar{P}(A)$  is updated through a two-step procedure: (1) calculation of an updating probability vector  $\bar{P}(B|A)$ , given the information in point B at some distance  $r$  from point A and conditioned on A:

$$\bar{P}(B|A) = \bar{M}_1^{(r/\Delta)} \bar{P}(B) \quad (7)$$

where  $r$  [L] is the distance between A and B; and (2) Bayesian updating producing a posterior probability vector,  $\bar{P}(A|B)$ , for point A, conditioned on B, using a modified Bayes' formula:

$$p_i(A|B) = \frac{p_i^{1-w}(A) p_i^w(B|A)}{\sum_{i=1}^n p_i^{1-w}(A) p_i^w(B|A)} \quad (8)$$

where  $w$  is a weight function based on the spatial dependence between the known and unknown point. It has the form:

$$w(r) = \frac{e^{1-(r/\Delta)}}{r/\Delta} \quad (9)$$

Bayesian updating is performed for all unknown cells within the influence length from each known cell. When new information becomes available, e.g. from drilling, the posterior estimate of the unknown cell becomes a new prior which is updated using the new information.

A measure of the state of knowledge at a specific point can be obtained by analyzing the probability vector  $\bar{P}$ . If no information about what is the most probable state at a specific point has been gathered,  $\bar{P}$  would have a uniform distribution so that  $P_i = 1/n$  for

all  $n$  states, thus representing the most uncertain case. If there is maximum certainty about the state at the specific point the probability is 1 for that state and 0 for all other states, thus representing the most certain case. A measure ranging from  $1/n$ , i.e. the most uncertain case, to 1, i.e. the most certain case, is referred to as Bayesian convincement,  $B$ :

$$B = \sum_{i=1}^n p_i^2 = \bar{P} * \bar{P} \quad (10)$$

The Bayesian convincement uncertainty estimate has a practical implication in that it can be used for directing further investigations to areas where information is best needed.

As mentioned earlier, the PI-value is equal to the expected value for an indicator,  $E(I)$ , that a canister position in the specific volume can be used ( $I=1$ ). The variance estimate for that indicator is:

$$Var(I) = E(I^2) - [E(I)]^2 = PI(1 - PI) \quad (11)$$

#### 4 RESULTS

PI-calculations using BayMar were made with respect to lithology, Rock Designation Quality (RQD) index, and hydraulic conductivity along a 420 meter stretch of tunnel. The tunnel was drilled with a full face tunnel boring machine (TBM) at repository depth at approximately 450 meters depth below sea level at the Äspö Hard Rock Laboratory (HRL). The lithology affects a number of factors, such as the fracturing and heat properties of the rock. The RQD value is of importance to the stability of the construction, and the hydraulic conductivity is connected to the inflow of ground water to the repository.

The model was run with a cell size of 20 x 20 x 20 meters, resulting in 21 target cells along the tunnel. Compliance levels were set so that (1) fine-grained granite is not allowed, (2) the RQD value must exceed 75%, and (3) the hydraulic conductivity must be lower than  $10^{-9}$  m/s in order to make a canister position acceptable. The existing information consisted of the geological map in 1:10 000 scale, 13 boreholes from the ground surface on the Äspö island, and 11 boreholes from the access tunnel leading down to the TBM-tunnel. Data were obtained from SKB's GEOTAB database. The prior estimates for the 21 target cells along the tunnel used for the BayMar predictions based on this information are shown in Table 1.

**Table 1.** States and prior probabilities (P) for lithology, hydraulic conductivity, and RQD.  
(ND = no data)

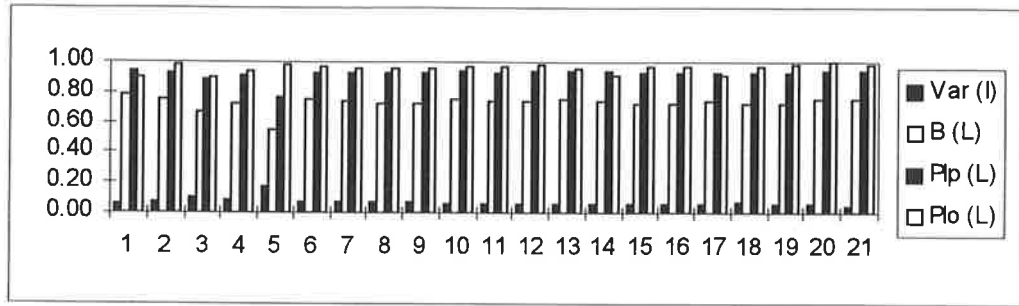
| State | Lithology            |      | Hydraulic Conductivity     |      | Rock Quality Designation (RQD) |      |
|-------|----------------------|------|----------------------------|------|--------------------------------|------|
|       | Material             | P    | Value                      | P    | RQD                            | P    |
| 1     | Soils                | 0.0  | $> 10^{-7}$ m/s            | 0.36 | 0-25                           | 0.06 |
| 2     | Greenstone           | 0.08 | $10^{-7}$ - $10^{-8}$ m/s  | 0.08 | 25-50                          | 0.04 |
| 3     | Fine-grained Granite | 0.06 | $10^{-8}$ - $10^{-9}$ m/s  | 0.12 | 50-75                          | 0.09 |
| 4     | Äspö Diorite         | 0.85 | $10^{-9}$ - $10^{-10}$ m/s | 0.07 | 75-100                         | 0.81 |
| 5     | Mylonite             | 0.0  | $< 10^{-10}$ m/s           | 0.37 | ND                             | ND   |

The predicted and observed PI-values with respect to each key parameter are given in Figure 3 for each cell along the tunnel, together with the Bayesian convincement and variance. The Bayesian convincement parameter, B, reflects the variability of the entire probability vector including all states, e.g. lithological states, in each cell, whereas the variance, V(I), reflects the uncertainty of the PI indicator value.

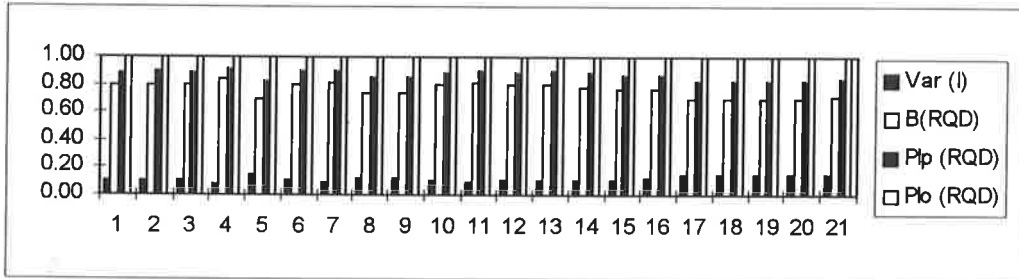
The TBM-tunnel is situated in an environment of high variability with respect to lithology, structures, and hydraulic properties, as described by Gustafson et al (1991), Wikberg et al (1991), and Wikman and Kornfält (1995), among others. This is also reflected in the boreholes used for the PI-calculations. However, locally along the TBM-tunnel the lithological and RQD conditions exhibit relatively little variability.

The BayMar model gave predictions which are in agreement with observations for lithology and RQD along the tunnel. The results show high PI-values for all cells, i.e. good conditions for placing canisters, with respect to lithology and RQD. For hydraulic conductivity, no measurements have been made in the tunnel, but the predictions indicate low PI-values, i.e. poor conditions for canister positioning along most of the tunnel. This is in agreement with the results from hydraulic tests in a borehole (KA3191F) parallel to the TBM-tunnel (see below).

(a)



(b)



(c)

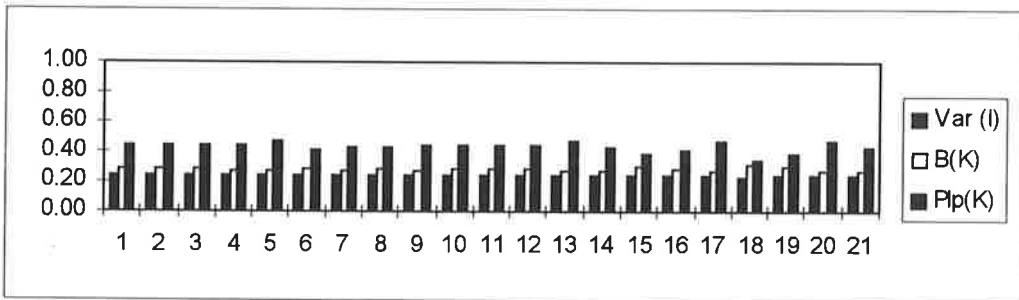


**Figure 3.** PI-values for cells 1-21 along the TBM-tunnel at the Äspö Hard Rock Laboratory (HRL). (a) and (b): Predicted (p) and observed (o) PI-values, Bayesian convincement (B) and variance for the PI-indicator (Var(l)), for lithology (L) and RQD; (c). Predicted PI-values, Bayesian convincement and variance for the PI-indicator for Hydraulic Conductivity (K). Piles in the diagrams appear in the same order from left to right as from top to bottom in the legend boxes.

A comparison between predicted and observed percentages of the four lithological states of greenstone, fine-grained granite, diorite, and mylonite along the TBM-tunnel is shown in Figure 4. The figures on the observed percentages were taken from the GEOTAB database and was not based on a continuous mapping of the tunnel, but mapping of 4 meter long blocks at irregularly spaced tunnel intervals. BayMar gave reasonable results regarding the percentages of different lithologies. The lithologies exhibit little variation along the tunnel for both predictions and observations, given the applied cell size. The large slab of greenstone in cells 10 and 11 could not be predicted.

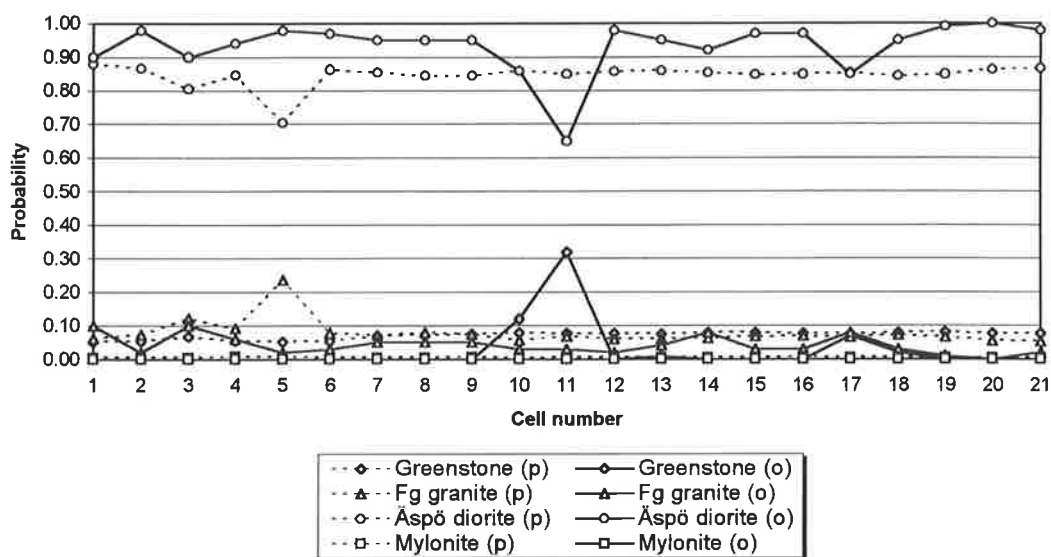


**Figure 4.** Predicted (p) and observed (o) probabilities of lithologies for 21 cells along the TBM-tunnel at the Äspö HRL using the BayMar model.

The percentages of the four lithological states based on a continuous mapping of the first 200 meters of the tunnel were given by Rhén et al (1995). A comparison between these results and the predictions by the BayMar model is shown in Figure 5.

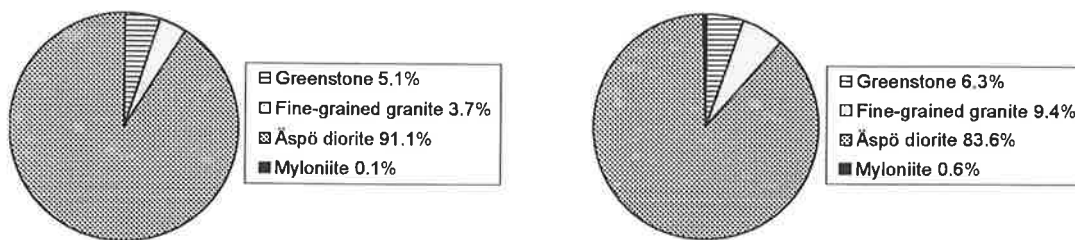


**Figure 5.** Comparison between observed (left) and predicted (right) percentages of lithologies along the first 200 meters of the TBM-tunnel.

For RQD no variation was observed along the TBM-tunnel, because of the very coarse classification into only four states; all cells along the tunnel were found to be of state 4, thus having a PI of 1.

For hydraulic conductivity, the BayMar model was not capable to predict variations along the 200 meters measured in the KA3191F borehole parallel to the TBM-tunnel. However, the model predicted the magnitude of the conductivity values reasonably well, given both surface boreholes and access tunnel boreholes. Conductivity values exceeding  $10^{-7}$  m/s were predicted as most likely along most of the tunnel, which is in agreement with measurements in KA3191F (Figure 6). The KA3191F borehole corresponds to cells 1-11 in the predictions.

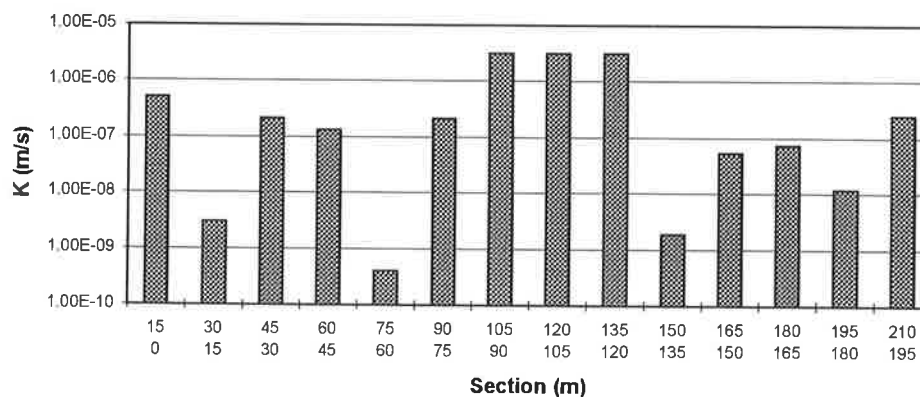


**Figure 6.** Hydraulic conductivity measurements along the KA3191F borehole.

Figure 7 shows a compilation of the PI predictions compared to observed conditions, treating the entire TBM-tunnel as one unit. This is from a practical point of view the most interesting figure, since the key issue in the design of the repository will be to decide if a drilling of a tunnel in a specific direction should be initiated or not (Rosén and Gustafson, 1994).

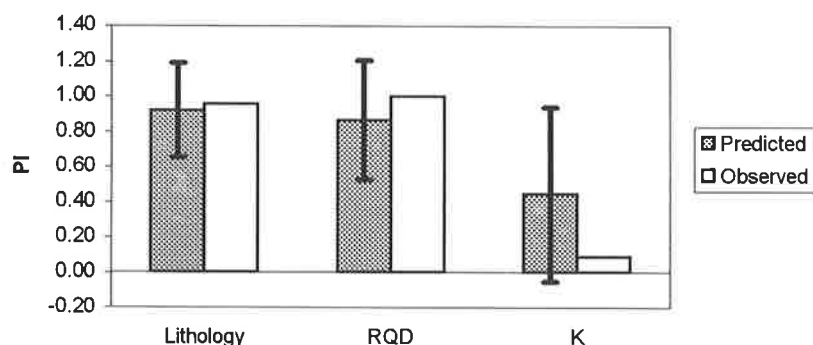


**Figure 7.** Summary statistics of predicted and observed PI-values for lithology, RQD and hydraulic conductivity (K) along the TBM-tunnel. Uncertainties of each predicted PI indicator are represented by piles showing  $\pm$  one standard deviation.

The results given in Figure 7 show that 92% of the positions could be used with respect to lithology, i.e. 19 of the 21 possible positions. For RQD the predicted PI was 87%, which is equal to 18 acceptable positions. For hydraulic conductivity the predicted PI was 44%, which equals 9 acceptable positions.

The predicted PI-values for lithology and RQD are high with relatively low standard deviations, indicating a low uncertainty about positioning conditions given assigned compliance levels. For hydraulic conductivity predicted PI-values are low with a high standard deviation, indicating a high uncertainty about the possibilities for placing canisters in the tunnel given assigned compliance levels. The general pattern was thus predicted correctly with good conditions with respect to lithology and RQD, and poor conditions with respect to hydraulic conductivity. The results indicate that a reasonable decision basis would have been obtained by using the BayMar model in the design of the Äspö HRL.

## 5 CONCLUSIONS

The BayMar model was developed for predicting hydrogeological and geological conditions in discrete cells. The results can be used for estimating the suitability for a specific purpose within each cell of a model, e.g. positioning of fuel canisters, as shown in this paper. The results can also be used as inputs in stochastic modeling, e.g. ground-water flow and transport models. The BayMar methodology provides a tool for performing the probability estimations using both measured, hard data and professional judgments. We consider the major weaknesses of the methodology to be that (1) it is less powerful than parametric methods based on regionalized variable theory in handling data that meet the requirements of additivity and normality and (2) there is a discrepancy among statisticians regarding Bayesian statistics, and it should be emphasized that its use is controversial. It can easily be misused if the evaluator is not experienced and skillful, and it is recommended that consensus among several people is achieved when prior information is sparse.

We consider the major strengths of the methodology to be that (1) the non-parametric approach allows handling of classified data without any model fitting to standard statistical distributions, i.e. there are no requirements on additivity and normality as with parametric geostatistical methods based on regionalized variable theory; (2) the use of Bayesian statistics facilitates incorporation of professional judgments and updating as new data become available in a formal way, making the methodology directly applicable to risk and decision analysis; (3) the use of Markov-chain analysis is multiattribute and the methodology is capable of handling several discrete states of information simultaneously; and (4) the approach of using sizes, shapes, and orientations of states for prior estimates of the transition probability matrices is in line with common practice in geological and hydrogeological work.

The application of the BayMar model at the Äspö HRL was made in a very heterogeneous environment, which was reflected in the transitional probability matrices, which showed statistical dependence over short distances for most of the states. The Markov-chain analysis part of the model focuses on the transitions between states, which makes it possible to generate predictions that correspond reasonably well with the natural variability, even in a complex and very heterogeneous environment, such as at Äspö.

The results of this study showed that the BayMar methodology would have given a reasonable decision basis for the TBM-tunnel with respect to lithology, RQD, and hydraulic conductivity. The approach of making optimal use of existing information and updating the results as new data become available allows for logical and cost-effective construction of a repository.

**Acknowledgements:** This work was funded by research grants to Lars Rosén and Gunnar Gustafson from the Swedish Nuclear Fuel and Waste Management Company (SKB). Thanks to Lars Ericsson, Göran Bäckblom, and Christer Svemar at SKB. Thanks to Tomas Hallbeck for helping out in programming of the BayMar code.

**Keywords:** Groundwater, Bayesian Markov, Geostatistics, Risk, Nuclear waste.

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## SAMMANFATTNING

En geostatistisk metodik benämnd *A Bayesian Markov Geostatistical Model (BayMar)* har utvecklats för sannolikhetsbedömningar av hydrogeologiska och geologiska egenskaper i samband med etablering av ett slutförvar för utbränt kärnavfall. Sannolikhetsbedömningarna har praktisk användning som beslutsunderlag vid lokalisering och utformning av ett säkert slutförvar.

Det maximala antalet bränslebehållare som kan deponeras i en förvarstunnel är beroende av ett antal praktiska faktorer, såsom deponeringstunnlarnas dimension och inbördes placering. En viktig fråga är hur väl denna maximala kapacitet kan utnyttjas med hänsyn till hydrogeologiska, geologiska och strukturella egenskaper hos berget. Ett mått på denna maximala kapacitet benämns i denna artikel *Positioning Index (PI)*. PI definieras för en given bergvolym och ett givet syfte som sannolikheten att det maximala antalet positioner kan användas, givet en viss förvarsdesign.

BayMar är en icke-parametrisk metod och använder Markov-kedjeanalys för att beskriva den statistiska korrelationen mellan punkter i plan eller volym. Det icke-parametriska angreppet medger hantering av information som inte följer standardmässiga statistiska fördelningar, exempelvis bergartsklasser. Data behöver därmed inte uppfylla de krav på additivitet och normalitet som ställs av kriging. BayMar har ett formaliserat sätt att hantera erfarenhetsmässiga bedömningar genom det Bayesianska angreppssättet, vilket medger en konditionerad uppdatering av a priori bedömningar till posteriori bedömningar så snart ny information blir tillgänglig.

BayMar har använts för PI-beräkningar med hänsyn till kapseldeponering längs den 420 meter långa TBM-tunneln i Äspölaboratoriet, belägen c:a 450 meter under markytan. Beräkningarna gjordes med hänsyn till litologi, Rock Quality Designation Index (RQD) och hydraulisk konduktivitet. Som underlagsmaterial för beräkningarna användes geologisk karta över Äspö tillsammans med observationer borrhål från markytan och från nedfartstunneln till Äspölaboratoriet.

Resultaten indikerade goda förhållanden för kapselpositionering med hänsyn till litologi och RQD och antagna gränser för acceptabla förhållanden. Detta är i god överensstämmelse med de förhållanden som observerats i TBM-tunneln. För hydraulisk konduktivitet har inga mätningar gjorts i tunneln, men beräkningarna visar på låga sannolikheter att finna acceptabla positioner med hänsyn till antagna gränsvärden. Detta stämmer väl med de mätningar som gjorts i borrhål parallellt med TBM-tunneln.

## RMi - A NEW PRACTICAL CHARACTERIZATION SYSTEM FOR USE IN ROCK ENGINEERING

RMi - et nytt system for å karakterisere bergmasser for bruk i praktisk bergteknikk

*Arild Palmström, Berdal Strømme a.s., Sandvika, Norge*

### 1. INTRODUCTION

*"Provision of reliable input data for engineering design of structures in rock is one of the most difficult tasks facing engineering geologists and design engineers."*

Z.T. Bieniawski, 1984

In addition to Bieniawski (1984), several other authors like Hoek and Brown (1980) and Nieto (1983) and have indicated a need for a *strength characterization* of rock masses. The Rock Mass index (RMi) has been worked out to satisfy this need and for improving the methods of rock mass descriptions, including better practical guidelines for obtaining numerical observational data.

Rock masses are composed of rocks penetrated by discontinuities. With great diversity both in the composition of the intact rock and in the nature and extent of the discontinuities, rock masses exhibit an enormous variation range in structure as well as composition. This creates a great challenge when characterizing such complex materials. In addition, as reliable tests of the strength of rock masses are impossible or so difficult to carry out with today's technology, rock engineering is currently based mainly on qualitative, descriptive data found from observations. These descriptive data have to be converted into numerical values to make calculations in rock engineering possible.

### 2. THE ROCK MASS INDEX (RMi)

Construction materials, such as steel and concrete, commonly used in civil engineering and mining are mostly characterized by their strength properties. This basic property of the material is used in the engineering and design. In rock engineering, no such specific strength characterization of the rock mass is in common use. The Rock Mass index is introduced to characterize the strength of the rock mass to be suitable for application in rock engineering and other types of calculations associated with construction in rock. An important issue has been to use parameters in the RMi which have the greatest significance in engineering. This is described in this section.

## 2.1 On the selection of the parameters used in the RMI

Figure 1 shows the main variables constituting a rock mass. Hoek et al. (1992), are of the opinion that the strength characteristics for jointed rock<sup>1)</sup> masses are controlled by the block<sup>2)</sup> shape and size as well as the surface characteristics of the block determined by the intersecting joints. They recommend that these parameters be selected to represent the average condition of the rock mass. Also, Tsoutrelis et al. (1990), Matula and Holzer (1978), Patching and Coates(1968) and Milne et al. (1992) have set forth similar ideas. This does not imply that the properties of the intact rock material should be disregarded in the characterization. If joints are widely spaced or if the intact rock is weak, the properties of the intact rock may strongly influence the overall behaviour of the rock mass. The intact rock properties are also important if the joints are discontinuous.<sup>3)</sup>

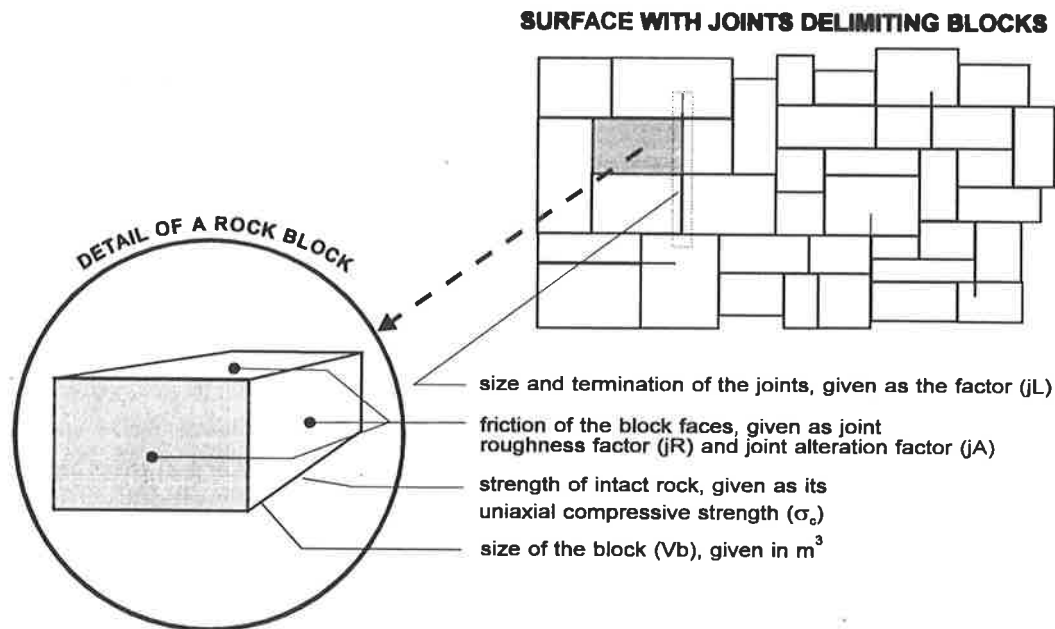


Figure 1 Idealized structure of a typical rock mass and the main parameters which are applied in the RMI. (from Palmström, 1995).

Although rock mass properties in many cases are governed by joints, rocks properties have been a major factor in the formation and development of the actual joints. In this respect petrological data can give useful information on the properties jointing

<sup>1)</sup> The term 'joint' has been used for most natural discontinuities which have thickness smaller than approx. 0.1 m. Thus, joints cover fissures, partings, fractures, natural cracks, as well as many shears and seams.

<sup>2)</sup> Joints and other types of discontinuities divide the rocks into blocks.

<sup>3)</sup> Discontinuous joints end in massive rock.

(Franklin, 1970; Piteau, 1970). A concise rock description accompanied by jointing observations will, in addition to geology and the type of material at the site, inform the reader of the probable behaviour of the ground.

As indicated in Figure 1, the RMI makes use of the following input parameters:

- compressive strength of intact rock;
- block volume; and
- joint characteristics, as given by its roughness, alteration, and size.

The combination of these parameters, included in the RMI, is shown in Figure 2.

All these are *intrinsic parameters* of the rock mass. The need to use such parameters in characterizing the properties of rock masses has earlier been stressed by Deere et al. (1969) and Patching and Coates (1968).

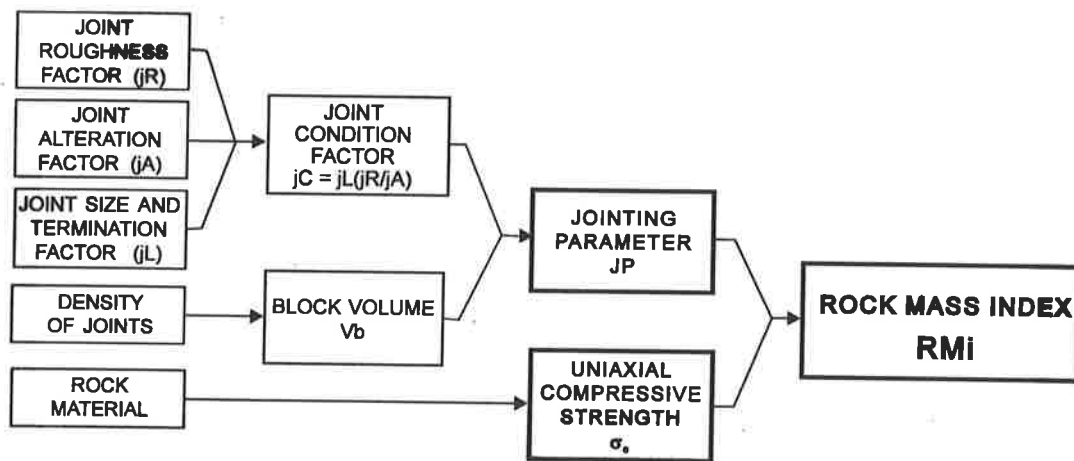


Figure 2 The input parameters to the RMI and their combination (from Palmström, 1995).

Principally, the RMI is based on the reduction in strength of a rock caused by jointing and is expressed as:

$$RMI = \sigma_c \cdot JP \quad \text{eq. (1)}$$

where  $\sigma_c$  = the uniaxial compressive strength of intact rock measured on 50 mm samples;  
 $JP$  = the jointing parameter which is a reduction factor representing the block size and the condition of its faces as represented by their friction properties. In addition, a scale factor for the size of the joints have been included, as shown in Figures 1 and 2.

The influence of  $JP$  has been found by using calibrations from test results. Because of problems of obtaining compression test results on rock masses at a scale similar to that of typical rock works, it was possible to find appropriate data from only eight large scale tests and one back analysis. Three of these are from Sweden, provided with kind help from Norbert Krauland, Boliden Mines and Bengt Leijon, Conterra AB. These test results have been used to arrive at the following mathematical expression:

$$JP = 0.2\sqrt{jC} \cdot Vb^D \quad \text{eq. (2)}$$

where  $Vb$  = the block volume given in  $m^3$ ,

$jC$  = the joint condition factor is expressed as  $jC = jL (jR/jA)$  in which  $jL$  is the joint length and continuity factor,  $jR$  is the joint wall roughness factor<sup>4)</sup>, and  $jA$  is the joint surface alteration factor<sup>2)</sup>. Their ratings are shown in Tables 1 to 3,

$D = 0.37 jC^{-0.2}$  has the following values:

|            |       |       |       |       |      |       |       |       |       |      |       |       |       |       |
|------------|-------|-------|-------|-------|------|-------|-------|-------|-------|------|-------|-------|-------|-------|
| for $jC =$ | 0.1   | 0.25  | 0.5   | 0.75  | 1    | 1.5   | 2     | 2.5   | 3     | 4    | 6     | 12    | 16    | 30    |
| $D =$      | 0.586 | 0.488 | 0.425 | 0.392 | 0.37 | 0.341 | 0.322 | 0.308 | 0.297 | 0.28 | 0.259 | 0.225 | 0.213 | 0.187 |

The value of  $JP$  varies from near 0 for crushed rocks to 1 for intact rock. The exponential form of eq. (2) fits well with the general experience that joint spacings have an exponential statistical distribution as shown by Merritt and Baecher (1981). Most commonly, the joint condition factor  $jC = 1$  to 2; thus, the jointing parameter will vary between  $JP = 0.2 Vb^{0.37}$  and  $JP = 0.28 Vb^{0.32}$ . For  $jC = 1.75$  the jointing parameter and the Rock Mass index can simply be expressed as

$$JP = 0.25 \sqrt[3]{Vb} \quad \text{and} \quad RMI = 0.25 \sigma_c \sqrt[3]{Vb} \quad \text{eq. (3a) and eq. (3b)}$$

Significant *scale effects* are generally involved when the tested rock volume is enlarged from laboratory size to field size. From the calibration described above, the  $RMI$  is tied to large samples where the scale effect has been included in  $JP$ . For massive rock masses, however, the scale effect for the uniaxial compressive strength ( $\sigma_c$ ) has not been accounted for, as  $\sigma_c$  is related to 50 mm sample size. Barton (1990) suggests from data presented by Hoek and Brown (1980) and Wagner (1987), that the actual compressive strength for large, massive 'field samples' may be determined from

$$\sigma_{cm} = \sigma_{c50} (0.05/Db)^{0.2} \quad \text{eq. (4)}$$

where  $\sigma_{c50}$  = the uniaxial compressive strength for 50 mm sample size

$Db$  = block diameter measured in metre, which may be found from  $Db = \sqrt[3]{Vb}$  or, in cases where a pronounced joint set occurs, from  $Db = S$ , where  $S$  is the spacing of this set. If the block shape factor ( $\beta$ ) is known (see Appendix<sup>5)</sup>, Section A6) the equivalent block diameter is

$$Db = \frac{\beta_0}{\beta} \sqrt[3]{Vb} = \frac{27}{\beta} \sqrt[3]{Vb} \quad \text{eq. (5)}$$

The expression  $(0.05/Db)^{0.2} = f_\sigma$  in eq. (4) is the scale factor for compressive strength.

<sup>4)</sup> The factors  $jR$  and  $jA$  are similar to the joint roughness number ( $J_r$ ) and the joint alteration number ( $J_a$ ) in the Q-system (Barton et al., 1974). The symbols  $J_r$  and  $J_a$  have been changed into  $jR$  and  $jA$  because some minor modifications have been made in their definitions.

<sup>5)</sup> In addition to the block shape factor mentioned above, the Appendix describes various types of measurements which can be used to estimate the block volume.

Eq. (4) is valid for sample diameters up to some metres, and may, therefore, be applied for massive rock masses.

**TABLE 1 THE RATINGS OF THE JOINT ROUGHNESS FACTOR ( $jR$ ) FOUND FROM SMOOTHNESS AND WAVINESS (From Palmström, 1995)**  
(The ratings of  $jR$  are similar to  $J_r$  in the  $Q$ -system)

| small scale<br>smoothness of<br>joint surface                                      | large scale waviness of joint plane |                        |                        |         |                               |
|------------------------------------------------------------------------------------|-------------------------------------|------------------------|------------------------|---------|-------------------------------|
|                                                                                    | planar                              | slightly<br>undulating | strongly<br>undulating | stepped | interlocking<br>(large scale) |
| very rough                                                                         | 3                                   | 4                      | 6                      | 7.5     | 9                             |
| rough                                                                              | 2                                   | 3                      | 4                      | 5       | 6                             |
| slightly rough                                                                     | 1.5                                 | 2                      | 3                      | 4       | 4.5                           |
| smooth                                                                             | 1                                   | 1.5                    | 2                      | 2.5     | 3                             |
| polished                                                                           | 0.75                                | 1                      | 1.5                    | 2       | 2.5                           |
| slickensided <sup>*)</sup>                                                         | 0.6 - 1.5                           | 1 - 2                  | 1.5 - 3                | 2 - 4   | 2.5 - 5                       |
| For filled joints: $jR = 1$ For irregular joints a rating of $jR = 5$ is suggested |                                     |                        |                        |         |                               |

<sup>\*)</sup> For slickensided joints the value of  $jR$  depends on the presence and appearance of the striations; the highest value is used for marked striations.

**TABLE 2 CHARACTERIZATION AND RATING OF THE JOINT ALTERATION FACTOR ( $jA$ )**  
(from Palmström (1995))  
( $jA$  is similar to  $J_a$  in the  $Q$ -system, except for the grade of alteration)

| A. CONTACT BETWEEN THE TWO ROCK WALL SURFACES                              |                                                                        |                                                                        |                                                      |
|----------------------------------------------------------------------------|------------------------------------------------------------------------|------------------------------------------------------------------------|------------------------------------------------------|
| TERM                                                                       | DESCRIPTION                                                            | $jA$                                                                   |                                                      |
| <b>Clean joints</b>                                                        |                                                                        |                                                                        |                                                      |
| -Healed or "welded" joints                                                 | - Softening, impermeable filling (quartz, epidote etc.)                | 0.75                                                                   |                                                      |
| -Fresh rock walls                                                          | - No coating or filling on joint surface, except for staining          | 1                                                                      |                                                      |
| -Alteration of joint wall:                                                 |                                                                        |                                                                        |                                                      |
| · 1 grade more altered                                                     | - The joint surface exhibits one class higher alteration than the rock | 2                                                                      |                                                      |
| · 2 grades more altered                                                    | - The joint surface shows two classes higher alteration than the rock  | 4                                                                      |                                                      |
| <b>Coating or thin filling</b>                                             |                                                                        |                                                                        |                                                      |
| -Sand, silt, calcite etc.                                                  | - Coating of friction materials without clay                           | 3                                                                      |                                                      |
| -Clay, chlorite, talc etc.                                                 | - Coating of softening and cohesive minerals                           | 4                                                                      |                                                      |
| B. FILLED JOINTS WITH PARTIAL OR NO CONTACT BETWEEN THE ROCK WALL SURFACES |                                                                        |                                                                        |                                                      |
| TYPE OF FILLING MATERIAL                                                   | DESCRIPTION                                                            | Partial wall contact<br>thin fillings<br>( $< 5 \text{ mm}^*)$<br>$jA$ | No wall contact<br>thick filling<br>or gouge<br>$jA$ |
| -Sand, silt, calcite etc.                                                  | - Filling of friction materials without clay                           | 4                                                                      | 8                                                    |
| -Compacted clay materials                                                  | - "Hard" filling of softening and cohesive materials                   | 6                                                                      | 10                                                   |
| -Soft clay materials                                                       | - Medium to low over-consolidation of filling                          | 8                                                                      | 12                                                   |
| -Swelling clay materials                                                   | - Filling material exhibits clear swelling properties                  | 8 - 12                                                                 | 12 - 20                                              |

<sup>\*)</sup> Based on joint thickness division in the RMR system (Bieniawski, 1973)

TABLE 3 THE JOINT SIZE AND CONTINUITY FACTOR ( $jL$ ) (from Palmström, 1995).

| JOINT LENGTH | TERM            | TYPE                                                                   | $jL$              |                                    |
|--------------|-----------------|------------------------------------------------------------------------|-------------------|------------------------------------|
|              |                 |                                                                        | continuous joints | discontinuous joints <sup>*)</sup> |
| < 0.5 m      | very short      | bedding/foliation partings                                             | 3                 | 6                                  |
| 0.1 - 1.0 m  | short/small     | joint                                                                  | 2                 | 4                                  |
| 1 - 10 m     | medium          | joint                                                                  | 1                 | 2                                  |
| 10 - 30 m    | long/large      | joint                                                                  | 0.75              | 1.5                                |
| > 30 m       | very long/large | filled joint <sup>*)</sup> , seam <sup>*)</sup> or shear <sup>*)</sup> | 0.5               | 1                                  |

<sup>\*)</sup> Often occurs as a single discontinuity, and should in these cases be treated separately. <sup>\*\*)</sup> Discontinuous joints end in massive rock

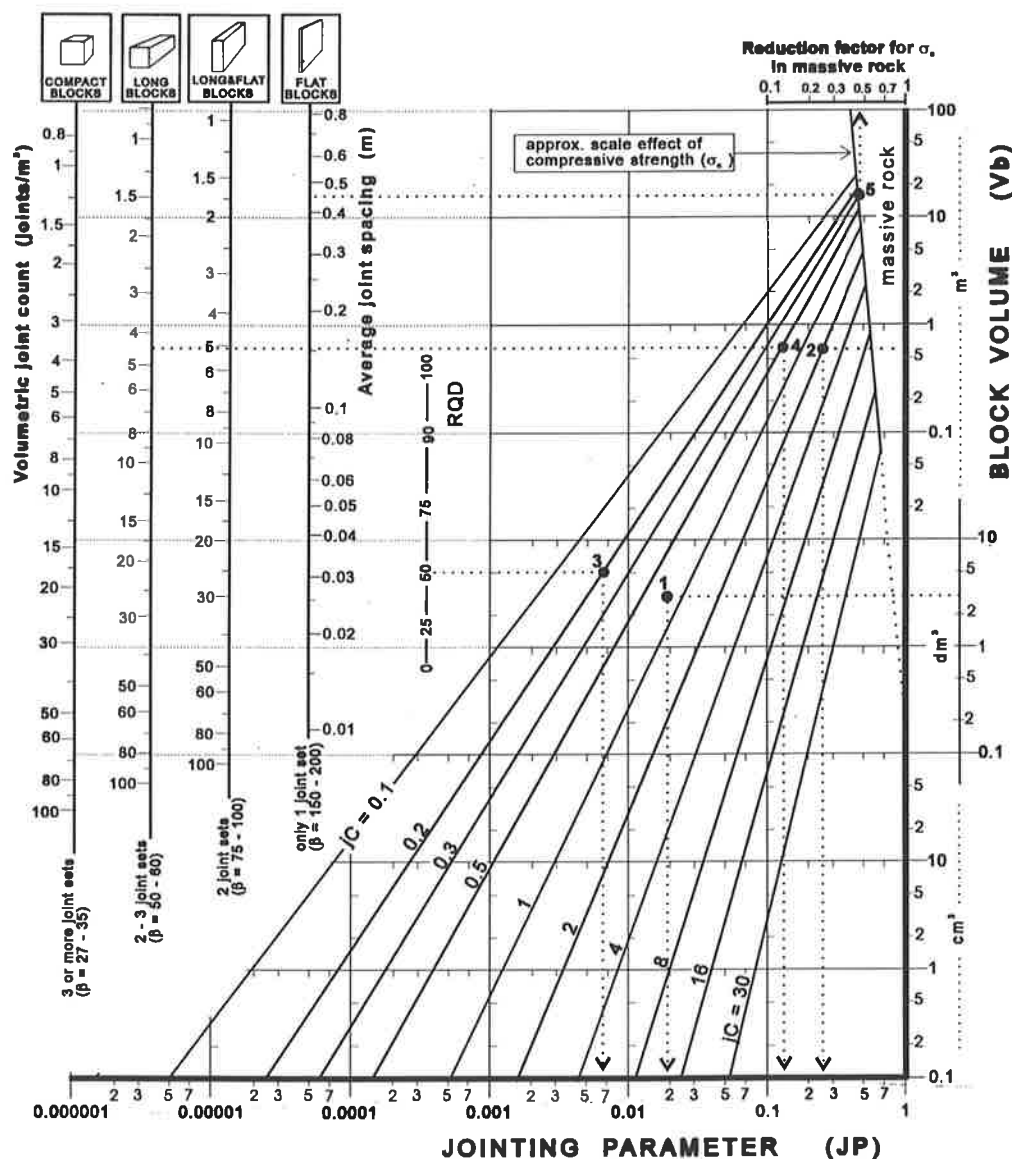


Figure 3 The jointing parameter (JP) found from the joint condition factor ( $jC$ ) and various measurements of jointing intensity ( $V_b$ ,  $J_v$ , RQD). The determination of JP from  $V_b$  (or RQD or  $J_v$ ) in the examples shown in Section 3 are indicated (from Palmström, 1996a).

From Figure 3 the jointing parameter (JP) can be found using the block volume ( $V_b$ ) and the joint condition factor ( $jC$ ). As shown in the upper left part of the diagram, the volumetric joint count ( $J_v$ ) for various joint sets (and/or block shapes) can be used instead of the block volume. This is based on the correlations given in the Appendix. Also, the RQD can be used, but its inability to characterise massive rock or highly jointed rock masses leads to a reduced quality of the JP.

The classification of RMI is presented in Table 4. Numerical values alone are seldom sufficient for characterizing the properties of a complex material such as a rock mass. Therefore, the RMI and its parameters should be accompanied by supplementary descriptions.

TABLE 4 CLASSIFICATION OF RMI. (from Palmström, 1995)

| TERM           |                               | RMI value    |
|----------------|-------------------------------|--------------|
| for RMI        | related to rock mass strength |              |
| Extremely low  | Extremely weak                | < 0.001      |
| Very low       | Very weak                     | 0.001 - 0.01 |
| Low            | Weak                          | 0.01 - 0.1   |
| Moderate       | Medium                        | 0.1 - 1      |
| High           | Strong                        | 1 - 10       |
| Very high      | Very strong                   | 10 - 100     |
| Extremely high | Extremely strong              | > 100        |

The RMI can be applied in various types of rock engineering with adjustment for features related to the particular project or utilisation of the rock. These applications are briefly described in Section 4.3.

### 3. EXAMPLES

The values of the jointing parameter (JP) found in the following examples are shown in Figure 3.

#### Example 1

The block volume has been measured as  $V_b = 0.003 \text{ m}^3 (= 3 \text{ dm}^3)$ . As given in Tables 1 to 3, the joint condition factor  $jC = 0.75$  is determined from:

- the rough joint surfaces and small undulations of the joint wall which gives  $jR = 3$ ;
- the clay coated joints, i.e.  $jA = 4$ ; and
- the 3 - 10 m long, continuous joints, which gives  $jL = 1$ .

Applying the values for  $V_b$  and  $jC$  in Figure 3, a value of  $JP = 0.02$  is found.<sup>6)</sup> With a compressive strength of the rock  $\sigma_c = 50 \text{ MPa}$ , the value of  $RMI = 0.02 \cdot 150 = 3$  (high)

<sup>6)</sup> Using eq. (2) a value of  $JP = 0.018$  is found

### **Example 2**

The block volume is  $V_b = 0.6 \text{ m}^3$ . The joint condition factor  $j_C = 2$  is determined from Tables 1 to 3, based on:

- smooth joint surfaces and planar joint walls which gives  $j_R = 1$ ;
- fresh joints,  $j_A = 1$ ; and 1 to 10 m long discontinuous joints, i.e.  $j_L = 2$ .

From Figure 3 the value  $JP = 0.25$  is found.<sup>7)</sup> With a compressive strength  $\sigma_c = 50$  MPa of the rock, the value of  $RMi = 12.5$  (very high).

### **Example 3**

Values of  $RQD = 50$  and  $j_C = 0.2$  give  $JP = 0.007$

### **Example 4**

Two joint sets with average spacings 0.3 m and 1 m, and some random joints occur in an area. The volumetric joint count is  $J_v = 1/0.3 + 1/1 + 0.5$ <sup>8)</sup> = 4.5

With a joint condition factor  $j_C = 0.5$  the jointing parameter  $JP = 0.12$  (by using the column for 2 to 3 joint sets in Figure 3).

### **Example 5**

The following jointing features are measured: only one joint set with average spacing  $S = 0.45$  m, and a joint condition factor  $j_C = 8$ . For this massive rock it is seen in Figure 3 that the value of  $JP$  is determined from the scale factor for compressive strength  $f_c = 0.45$ . For a rock with  $\sigma_c = 130$  MPa the value of  $RMi = 59.6$  (very high).

## **4. DISCUSSION**

The  $RMi$  can be applied in various types of rock engineering with adjustment for features related to the particular project or utilisation of the rock. These applications are briefly mentioned in Section 4.3.

### **4.1 Benefits - limitations of the $RMi$**

Some of the benefits of the  $RMi$  system are:

- *The  $RMi$  will give significant improvements in the use of geological input data.* This is mainly achieved by its systematic use of well defined parameters in which the three-dimensional character of rock masses is represented by the block volume.
- *The  $RMi$  can easily be used for rough estimates when limited information is available on the ground conditions.* For example, in early stages of a project where rough estimates are sufficient, eq. (3b) can be applied.

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<sup>7)</sup>  $JP = 0.24$  is found using eq. (2)

<sup>8)</sup> The assumed value for the random joints.

- *The R<sub>Mi</sub> is well suited for comparisons and exchange of knowledge between different locations.*

In this way it may contribute to improved communication between people involved in rock engineering and design.

- *The R<sub>Mi</sub> offers a platform suitable for engineering judgement.*

R<sub>Mi</sub> is a general parameter which characterizes the inherent strength of rock masses, and may be applied in engineering as a quality indicator for this construction material. As R<sub>Mi</sub> is composed of real block volumes and common joint parameters for rock masses, it should easily be related to the field conditions. This is important in application of engineering judgement.

- *The R<sub>Mi</sub> system covers a wide spectrum of rock mass variations.*

It therefore has possibilities for wider applications than the existing rock mass classification and characterization systems of today.

Any attempt to mathematically express the variable structure and properties of jointed rock masses in a general failure criterion, may result in complex expressions. By restricting the R<sub>Mi</sub> to uniaxial compressive strength only, it has been possible to arrive at the relatively simple expressions in eqs. (1) and (2). Because simplicity has been preferred in the structure as well as in the selection of parameters in R<sub>Mi</sub>, it is clear that such an index may result in inaccuracy and limitations, of which the main are connected to:

- *The range and types of rock masses covered by the R<sub>Mi</sub>.*

Both the intact rock materials as well as the joints exhibit great directional variations in composition and structure which may result in a large range in compositions and properties of rock masses. It is, therefore, not possible to characterize all these combinations in one, single number. Nevertheless, the R<sub>Mi</sub> probably characterizes a wider range of materials than most classification systems.

- *The accuracy in the expression of R<sub>Mi</sub>.*

The value of the jointing parameter (JP) is calibrated from a few large scale compression tests. Both the evaluation of the various factors ( $j_R$ ,  $j_A$  and  $V_b$ ) used in obtaining JP and the relatively small size of some of the samples tested, may be sources of error in the expression for JP. The value of R<sub>Mi</sub> found may therefore be approximate. In some cases, however, errors in the various parameters may partly have neutralized each other.

- *The effect of combining parameters that vary in range.*

The parameters used to calculate the R<sub>Mi</sub> will in general express a certain range of values. As with any classification system, combination of such variables may cause errors. In some cases the result is that the R<sub>Mi</sub> may be inaccurate. For these reasons, the R<sub>Mi</sub> may best be considered as a *relative* index in its characterization of the rock mass strength.

#### 4.2 Other similar rock mass characterization methods

A similar approach to a strength characterization of rock masses has been proposed by Hansagi (1965a, 1965b), who introduced a reduction factor ( $C_g$ ) comparable to the jointing parameter (JP) to arrive at an expression for the *compressive strength* of the rock mass, expressed as

$$\sigma_{cm} = \sigma_c \cdot C_g \quad \text{eq. (6)}$$

Hansagi named  $C_g$  as a 'gefüge-factor' (= joint factor) being "*representative for the jointed effect of a rock mass*". This factor consists of two inputs: a factor for the "structure of jointing" (core length), and a scale factor. Hansagi (1965b) mentions that the value of  $C_g$  is 0.7 for massive rock and 0.47 for jointed rock (containing small joints) for two test locations in Kiruna, Sweden. Hansagi did not, however, as far as the author knows, publish more on his method.

In its original form the Hoek-Brown criterion is expressed in terms of the major and the minor principal stresses at failure as

$$\sigma_1' = \sigma_3' + (m \cdot \sigma_c \cdot \sigma_3' + s \cdot \sigma_c^2)^{1/2} \quad \text{eq. (7)}$$

where  $\sigma_1'$  = is the major principal effective stress at failure.

$\sigma_3'$  = is the minor principal effective stress.

$\sigma_c$  = is the uniaxial compressive strength of the intact rock material from which the rock mass is composed.

$s$  and  $m$  are empirical constants representing inherent properties of jointing conditions and rock characteristics.

For  $\sigma_3' = 0$ , eq. (7) expresses the unconfined *compressive strength* of a rock mass

$$\sigma_{cm} = \sigma_c \cdot s^{1/2} \quad \text{eq. (8)}$$

This expression is similar in structure to the expression  $RMi = \sigma_c \cdot JP$

#### 4.3 Possible applications of the *RMi*

The main purpose during development of the *RMi* has been to work out a practical system to characterize rock masses which can be used in rock engineering and design. When applied, the *RMi*-value or its parameters are adjusted for local features of importance for the engineering purpose, as indicated in Figure 4.

Figure 5 shows the main areas for application of *RMi* together with the influence of its parameters in different fields. The *RMi*-value can seldom be used directly in classification systems as many of them are systems made for a particular purpose. Some of the input parameters included in the *RMi* are sometimes similar to those used in the classifications and may then be applied more or less directly.

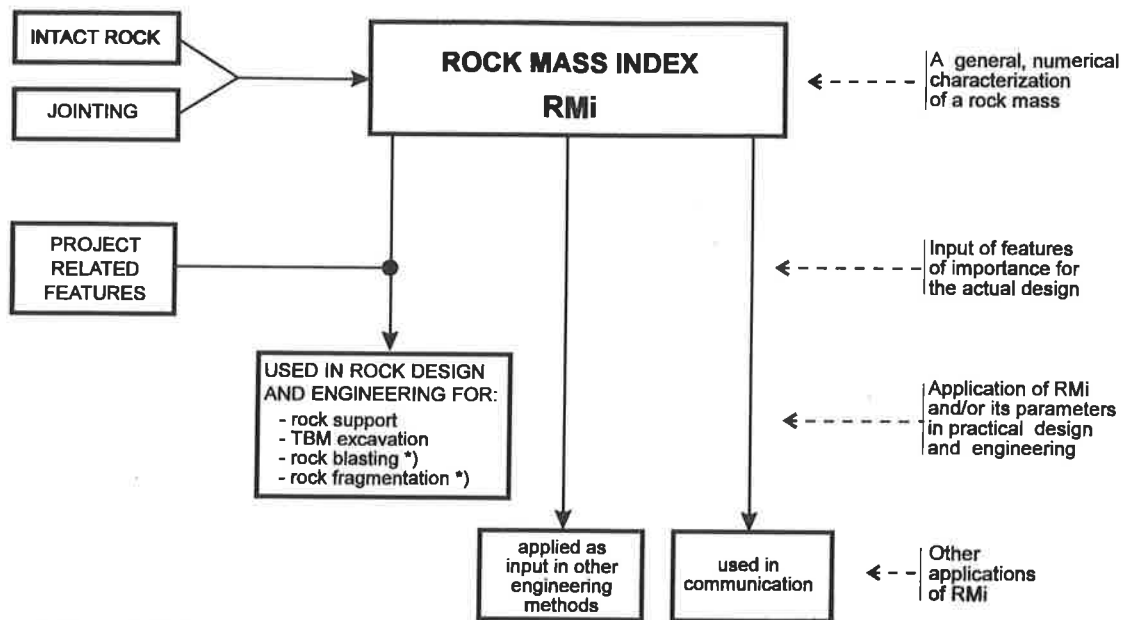


Figure 4 The principle application of RMi in rock engineering (from Palmström, 1995)

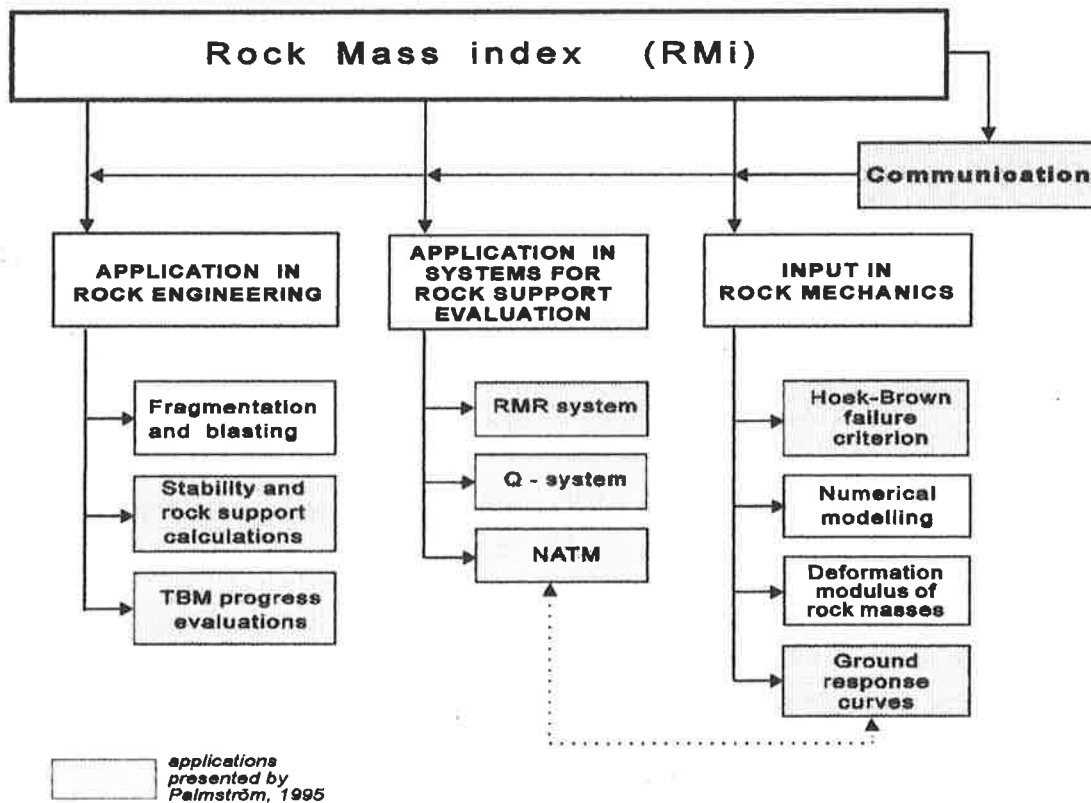


Figure 5 The main applications of RMi in rock mechanics and rock engineering (from Palmström, 1995).

According to Hoek and Brown (1980) the constants  $m$  and  $s$  in the Hoek-Brown failure criterion for rock masses<sup>9)</sup> depend on the properties of the rock and the extent to which it has been broken before being subjected to the [failure] stresses. Both constants are dimensionless.

The value of  $s$  ranges from 0 for jointed rock masses to 1 for intact rock. It is found using the RMR or the Q classification system as described by Hoek (1983), Hoek and Brown (1980, 1988), and Wood (1991). As seen in eq. (8) the value of  $s = JP^2$  can be found directly by applying the Rmi system.

The constant  $m$  varies with the jointing. In the later publication on the Hoek-Brown failure criterion it has been replaced by  $m_b$ . Palmström (1995) has shown that based on data from Wood (1990) and Hoek et al. (1992) it can be mathematically expressed:

- for undisturbed rock masses as  $m_b = m_i \cdot JP^{0.64}$  eq. (9)

- for disturbed rock masses as  $m_b = m_i \cdot JP^{0.857}$  eq. (10)

where  $m_i$  = values for intact rocks given in Table 5.

TABLE 5 VALUES FOR THE  $m_i$  FACTOR IN THE HOEK-BROWN FAILURE CRITERION  
(from Palmström, 1995a, based on Wood, 1990 and Hoek et al., 1992).

| Sedimentary rocks | Rating of the factor $m_i$ <sup>1)</sup> | Igneous rocks      | Rating of the factor $m_i$ <sup>1)</sup> | Metamorphic rocks   | Rating of the factor $m_i$ <sup>1)</sup> |
|-------------------|------------------------------------------|--------------------|------------------------------------------|---------------------|------------------------------------------|
| Anhydrite         | 13.2                                     | Andesite           | 18.9                                     | Amphibolite         | 31.2                                     |
| Claystone         | 3.4                                      | Basalt             | (17)                                     | Amphibolitic gneiss | 31 ?                                     |
| Conglomerate      | (20)                                     | Diabase (dolerite) | 15.2                                     | Augen gneiss        | 30 ?                                     |
| Coral chalk       | 7.2                                      | Diorite            | 27 ?                                     | Granite gneiss      | 30 ?                                     |
| Dolomite          | 10.1                                     | Gabbro             | 25.8                                     | Gneiss              | 29.2                                     |
| Limestone         | 8.4                                      | Granite            | 32.7                                     | Gneiss granite      | 30 ?                                     |
| Sandstone         | 18.8                                     | Granodiorite       | 20 ?                                     | Greenstone          | 20 ?                                     |
| Siltstone         | 9.6                                      | Monzonite          | 30 ?                                     | Marble              | 9.3                                      |
|                   |                                          | Norite             | 21.7                                     | Mica gneiss         | 30 ?                                     |
|                   |                                          | Rhyolite           | (20)                                     | Mica quartzite      | 25 ?                                     |
|                   |                                          | Syenite            | 30?                                      | Mica schist         | 15 ?                                     |
|                   |                                          |                    |                                          | Phyllite            | 13 ?                                     |
|                   |                                          |                    |                                          | Quartzite           | 23.7                                     |
|                   |                                          |                    |                                          | Slate               | 11.4                                     |
|                   |                                          |                    |                                          | Talc schist         | 10 ?                                     |

<sup>1)</sup> Values in parenthesis have been estimated by Hoek et al (1992); some others with question mark have been assumed by Palmström (1995a)

Thus, Rmi introduces an easier and more direct method to find the values of both constants  $s$  and  $m$ , as JP involves only inherent features which have a direct impact on the behaviour of the rock mass. In this way, Rmi may contribute to a future improvement of the Hoek Brown failure criterion.

<sup>9)</sup> It should be born in mind that the Hoek-Brown failure criterion is only valid for continuous rock masses (Hoek and Brown, 1980), i.e. massive rock or highly jointed and crushed rock masses.

A system for application of R<sub>Mi</sub> in rock support evaluations in underground excavations have been presented by Palmström (1995a). It is based on two main groups of ground; the continuous and the discontinuous rock masses determined by the ratio between block diameter and the tunnel diameter. For each group a chart has been presented, see Palmström (1995a or 1996a). In addition to the parameters involved in the R<sub>Mi</sub>, the joint orientation, the rock stress level in the excavation, and the number of joint sets are used. For weakness zones additional information on the thickness of the zone and the quality of the adjacent rock masses are used. Thus the R<sub>Mi</sub> method for rock support evaluation contains more information on the ground conditions than other similar classification systems for rock support determination.

For estimates of the penetration rate of full face tunnelling machines (TBM) a system applying JP and  $\sigma_c$  in addition to specification of the TBM has been presented by Palmström (1995a).

Finally, the system for characterizing block geometry (volume, shape factor, angles) in the R<sub>Mi</sub> system may be of use in numerical models.

#### ABSTRACT

*The Rock Mass index, R<sub>Mi</sub>, has been developed to satisfy a need for a strength characterization of rock masses for use in rock engineering and design. The method gives a measure of the reduction of intact rock strength caused by joints given by  $R_{Mi} = \sigma_c \cdot JP$ . Here  $\sigma_c$  is the uniaxial compressive strength of the intact rock measured on 50 mm diameter samples, and JP is the jointing parameter which is a combined measure of block size (or intensity of jointing) and joint characteristics as measured by joint roughness, alteration and size.*

*This paper describes the method of determining the R<sub>Mi</sub> for a rock mass using various common field observations. The determination of a meaningful equivalent block size is a key issue which is described in detail.*

#### SAMMENDRAG

R<sub>Mi</sub> (Rock Mass index) som er et nytt system for karakterisering av bergmasser, tilbyr bedre kvalitet ved bruk av geologiske data i bergteknikk. Det er basert på parametre for bergart (trykkfasthet) og oppsprekning (mengde av sprekker uttrykt ved blokkvolum, ruhet og karakter av sprekkeflater, samt lengde av sprekken). For å kunne kombinere alle disse parametrene i et uttrykk for en bergmasses fasthet, er det benyttet kalibrering mot 7 kjente tester av bergmasser og en 'back analysis'. Ved å karakterisere bergmassers styrke, er R<sub>Mi</sub> godt egnet for å kunne benyttes som basis inngangsparameter i ulike bergtekniske beregninger, som for eksempel stabilitet- og sikringsbestemmelser, inndrift ved fullprofilboring, og Hoek-Brown bruddkriterium for bergmasser. Blokkstørrelsen er viktigste inngangsparameter i systemet. Måter å måle denne på er derfor viet stor plass i artikkelen.

### *Acknowledgement*

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## APPENDIX

### METHODS AND CORRELATIONS TO DETERMINE THE BLOCK VOLUME

*"The success of the of the field investigation will depend on the geologist's ability to recognise and describe in a quantitative manner those factors which the engineer can include in his analysis."*

Douglas R. Piteau, 1970

#### A1 INTRODUCTION

In most cases the block size is the most important factor in the R<sub>Mi</sub>. Block size is also used in some of the applications of R<sub>Mi</sub> in engineering, especially for design of rock support. Consequently, the accuracy of this parameter has a significant impact on the quality of the R<sub>Mi</sub> and hence of the calculations performed. This appendix presents methods to determine the block volume from various types of jointing observations and measurements.

The block dimensions are determined by joint spacings and the number of joint sets. Individual or random joints and possibly other planes of weakness may further influence the size and shape of blocks. Impact from rock blasting may also influence. As the joint spacings generally vary greatly, the difference in size between the smaller and the larger blocks can be significant. Therefore, the characterization of block volume should be given rather as an interval than as a single value.

If less than 3 joint sets occur, defined blocks may not be found. However, in many cases the presence of random joints or other weakness planes may contribute to defining blocks. Also where the jointing is irregular, or many of the joints are discontinuous, it can sometimes be difficult to recognize the actual size and shape of individual blocks. Thus, from time to time the block size and shape therefore have to be determined using a simplification where an *equivalent block volume* is used, as is described in Section A8.

The correlations between various joint measurements are shown in Figure A1. As the blocks generally have varying sizes and shapes, the measurements of characteristic dimensions can be time-consuming and laborious. To remedy this, easy recognizable dimensions of the blocks and simple correlations between the different types of jointing measurements have been preferred, as is presented in this Appendix.

#### A2 BLOCK VOLUME MEASURED DIRECTLY IN SITU OR IN DRILL CORES

Where the individual blocks can be observed in a surface, their volume can be directly measured from relevant dimensions by selecting several representative blocks and measuring their average dimensions. For small blocks or fragments having volumes in dm<sup>3</sup> size or less, this method of block volume measurement is often beneficial as it is much easier to estimate the volume of a block compared to all the measurements which

have to be made to include all joints. Block volume can also be measured in drill cores where small fragments occur for example in faulted or crushed zones.

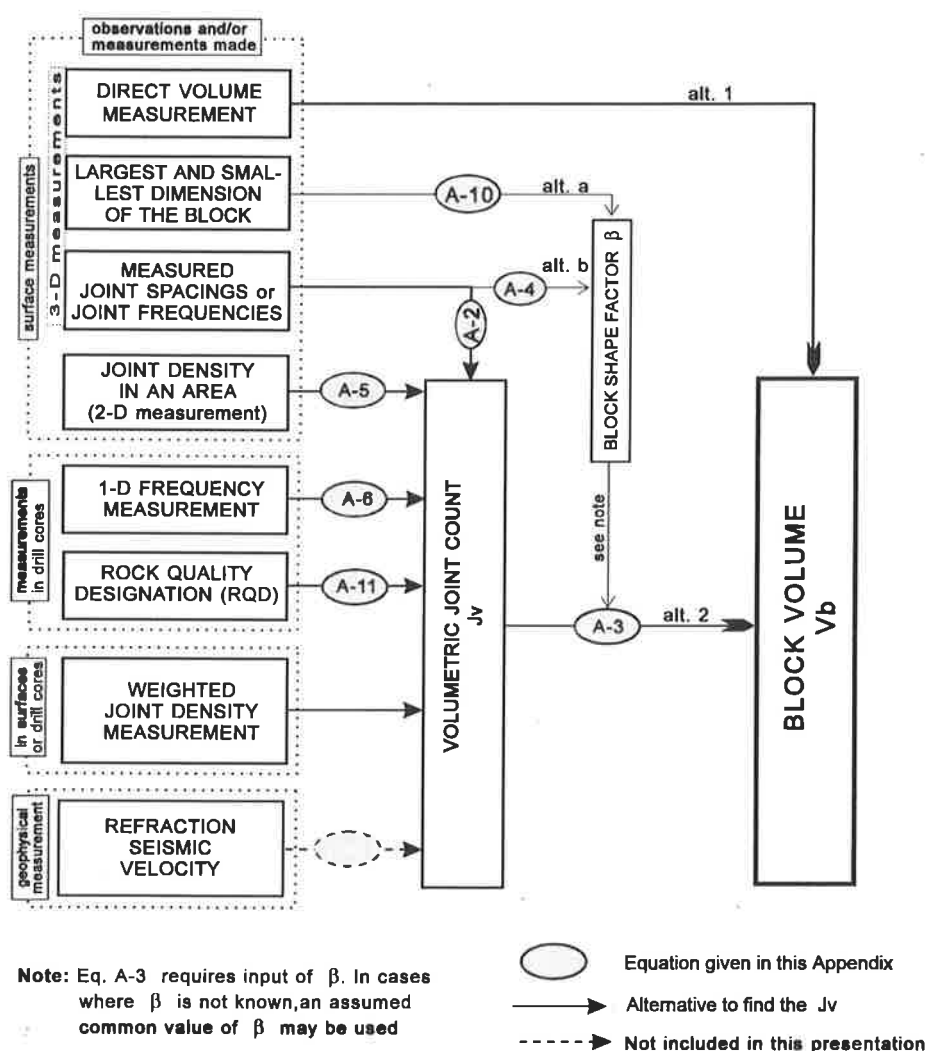


Fig. A1 The principles in estimating the block volume from various types of joint density measurements. (Revised from Palmström, 1995a)

### A3 BLOCK VOLUME FOUND FROM JOINT SPACINGS

The terms *joint spacing* and *average joint spacing* are often used in the description of rock masses. Joint spacing is the distance between individual joints within a joint set. Where more than one set occurs, this measurement is, in the case of surface observations, often given as the average of the spacings for these sets. There is often some uncertainty as to how this average value is found. For instance, the average spacing for

3 joint sets having spacings:  $S_1 = 1$  m,  $S_2 = 0.5$  m, and  $S_3 = 0.2$  m is  $S_a = 0.125$  m, and not 0.85 m which initially may seem appropriate.<sup>10)</sup>

When logging drill cores the average length of core pieces<sup>11)</sup> or frequencies are seldom true spacings, as joints of different sets probably are included in the measurement. In addition, random joints which do not necessarily belong to any joint set, have an influence.

As the term 'joint spacing' does not indicate what it includes, it is frequently difficult to determine whether a 'joint spacing' referred to in the literature represents the true joint spacing. Thus, there is often much confusion related to joint spacing recordings.

Especially where irregular jointing occurs, it is time-consuming to measure all (random) joints in a joint survey. In such cases, as well as for other jointing patterns, it is often much quicker - and also more accurate - to measure the block volume directly in the field as is mentioned in Section 3.

Where three regular joint sets occur, the block volume can be found from the joint spacings as

$$V_b = \frac{S_1 \cdot S_2 \cdot S_3}{\sin \gamma_1 \cdot \sin \gamma_2 \cdot \sin \gamma_3} = \frac{V_{b_0}}{\sin \gamma_1 \cdot \sin \gamma_2 \cdot \sin \gamma_3} \quad \text{eq. (A-1)}$$

where  $\gamma_1, \gamma_2, \gamma_3$  are the angles between the joint sets, and  
 $S_1, S_2, S_3$  are the spacings between the individual joints in each set.  
 $V_{b_0}$  is the block volume in cases where joints intersect at right angles.

For a rhombohedral block with two angles between  $45^\circ$  and  $60^\circ$ , two between  $135^\circ$  and  $150^\circ$  and the last two being  $90^\circ$ , the volume will be between  $V_b = 1.3 V_{b_0}$  and  $2 V_{b_0}$ . Compared to the variations caused by the joint spacings, the effect from the intersection angle between joint sets is generally relatively small.

#### A4 BLOCK VOLUME FOUND FROM JOINT FREQUENCY MEASUREMENTS

##### A4.1 The volumetric joint count ( $J_v$ )

The volumetric joint count ( $J_v$ ) has been described by Palmström (1982, 1985, 1986) and Sen and Eissa (1991, 1992). It is a measure of the number of joints within a unit volume of rock mass, defined by

<sup>10)</sup> The average spacing is found from  $1/S_a = 1/S_1 + 1/S_2 + 1/S_3$  and not from  $S_a = (S_1 + S_2 + S_3)/3$  which often seems to be applied.

<sup>11)</sup> Joint or fracture intercept is the appropriate term for measurement of the distance between joints along a line or borehole.

$$J_v = \sum (1/S_i) \quad \text{eq. (A-2a)}$$

where  $S_i$  = the joint spacing in metres for the each joint set  $i$ .

Also random joints can be included by assuming a random spacing for each of these. Experience indicates that this can be set to  $S_r = 5$  m; thus, the volumetric joint count can be generally expressed as

$$J_v = \sum (1/S_i) + N_r/5 \quad \text{eq. (A-2b)}$$

where  $N_r$  = the number of random joints. A more accurate method to determine  $N_r$  has been described by Palmström (1995a or 1995d).

$J_v$  can easily be calculated from joint observations, since it is based on measurements of joint spacings or frequencies. In the cases where mostly random or irregular jointing occur,  $J_v$  can be found by counting all the joints observed in an area of known size as described by Palmström (1995a or 1995d).

#### A4.2 The correlation between block volume ( $V_b$ ) and volumetric joint count ( $J_v$ )

Since both the volumetric joint count ( $J_v$ ) and the size of blocks in a rock mass vary according to the degree of jointing, there is a correlation between them (Palmström, 1982).  $J_v$  varies with the joint spacings, while the block size also depends on the type of block. A correlation between the two parameters has therefore to be adjusted or corrected for the block shape and the angle between the joint sets, given as

$$V_b = \beta \cdot J_v^{-3} \frac{1}{\sin \gamma_1 \cdot \sin \gamma_2 \cdot \sin \gamma_3} \quad \text{eq. (A-3a)}$$

For cases where all angles between the block faces are  $90^\circ$ , the block volume is

$$V_{b_0} = \beta \cdot J_v^{-3} \quad \text{eq. (A-3b)}$$

$$\text{The factor } \beta = \frac{(\alpha_2 + \alpha_2 \cdot \alpha_3 + \alpha_3)^3}{(\alpha_2 \cdot \alpha_3)^2} \quad \text{eq. (A-4)}$$

(where  $\alpha_2 = S_2/S_1$  and  $\alpha_3 = S_3/S_1$ ) depends mainly on the differences between the joint set spacings. It has therefore been named the *block shape factor*. It is further described in Section A8.

As the volumetric joint count ( $J_v$ ) by definition takes into account in an unambiguous way all the occurring joints in a rock mass, it is often appropriate to use  $J_v$  in the correlation between joint frequency measurements and block volume estimates (Palmström, 1982). Important here is the block shape factor  $\beta$  which is included in all equations to estimate the block volume, see Figure A1.

When the frequency is given for each joint set, it is, as mentioned, possible to determine the block volume directly. In other cases, when an 'average frequency' is given, it is as for joint spacings, uncertain whether this frequency value refers to one-, two- or three-dimensional measurements; hence no accurate general correlation can be presented. The use of joint frequency measurements presented in the following paragraph are similar to the joint spacing measurements shown in Section A3.

#### *A4.3 Block volume found from 2-D joint frequency measurements on an area or surface*

The 2-D joint frequency is the number of joints measured in an area. A simple correlation between 2-D and 3-D frequency ( $J_v$ ) values can be done using the empirical expression

$$J_v = N_a \cdot ka \quad \text{eq. (A-5)}$$

where  $ka$  = correlation factor. It varies mainly between 1 and 2.5 with an average value  $ka = 1.5$ . The factor has its highest value where the observation plane is parallel to the main joint set.

The joint frequency ( $N_a$ ) varies with the orientation of the observation plane and with respect to the attitude of the joints. Recording of  $N_a$  in several surfaces of various orientation gives a more accurate measure of the jointing. Being an average measure,  $N_a$  should be measured in selected areas showing the same type and density of jointing. Thus, a large area should be divided into smaller, representative areas containing similar jointing, and the variation in jointing for the whole area calculated based on these observations.

As the length of the joints compared to the size of the area will influence on the frequency observed, some sort of adjustments should be made where more accurate estimates are required.

#### *A4.4 From 1-D jointing frequency measurements along a scanline or drill core*

This is a record of the joint frequency along a borehole or a scanline given as the number of joints intersecting a certain length. This 1-D joint frequency is an average measure along a selected length of the core. As in other core logging methods, it is important to measure the joints in sections along the line or core which shows a similar joint frequency. At the start of the logging it is rational to divide the length of the borehole/scanline into such sections.

The correlation between 1-D joint frequency observations in drill holes (or scanlines) and volumetric 3-D frequency ( $J_v$ ) can be done using an expression similar to eq. (A-5). The joint frequency, given as the number of joints per metre, can be expressed as:

$$J_v = N_l \cdot kl \quad \text{eq. (A-6)}$$

where  $kl$  = correlation factor, which varies between 1.25 and 6, with an average value  $kl = 2$ . There is generally a rather poor correlation between  $J_v$  and  $N_l$ .

#### A5 WEIGHTED JOINT DENSITY MEASUREMENTS (wJd)

R. Terzaghi (1965) points out that the accuracy of jointing measurements can be increased by replacing the number of joints measured on a surface or in a bore hole ( $N_\alpha$ ) intersected at an angle  $\alpha$ , by a value  $N_{90}$ .  $N_{90}$  represents the number of joints with the same orientation which would have been observed at an intersection angle of  $90^\circ$ . This is expressed as

$$N_{90} = N_\alpha / \sin \alpha \quad \text{eq. (A-7)}$$

Terzaghi stresses the problem connected to small values of  $\alpha$ , because, in these cases, the number of intersections will be significantly affected by local variations in spacing and continuity. *"No correction whatsoever can be applied if  $\alpha$  is zero. Hence  $N_{90}$  would fail to correctly indicate the abundance of horizontal and gently dipping joints in a horizontal observation surface."*

The weighted joint density method is based on measuring the intersection angle between each joint and the observation surface or borehole. To solve the problem of small intersection angles and to simplify the observations, the angles have been divided into intervals as shown in Table A1.

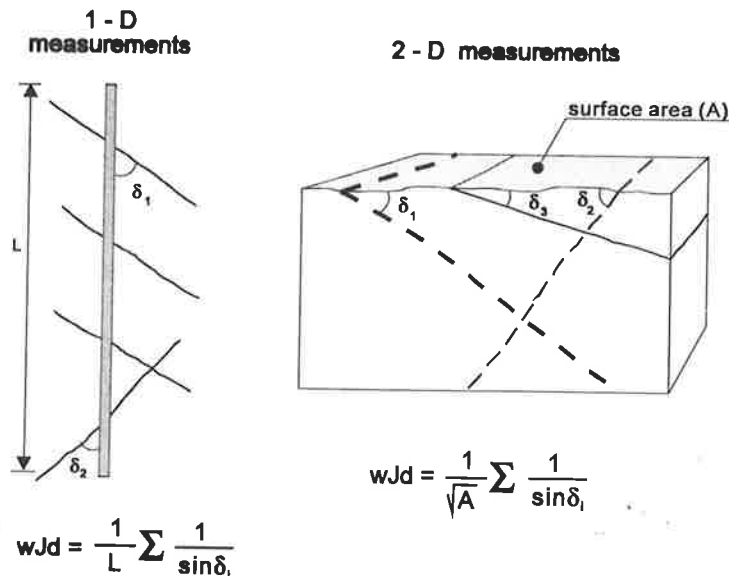


Fig. A2. The intersection between joints and a drill core hole (left) and a surface (right) (from Palmström, 1995).

For 2-D measurements (surface observations) the weighted joint density is defined as

$$wJd = (1/\sqrt{A}) \Sigma(1/\sin\delta_i) = (1/\sqrt{A}) \Sigma(f_i) \quad \text{eq. (A-8)}$$

and, similarly, for 1-D measurements along a scan line or in drill cores

$$wJd = (1/L) \Sigma(1/\sin\delta_i) = (1/L) \Sigma(f_i) \quad \text{eq. (A-9)}$$

where  $\delta_i$  = the angle between the observation plane (surface) and the individual joint.

A = the size of the area in  $m^2$ , see Figure A2.

L = the length of the measured section along core or line.

$f_i$  = the interval factor given in Table A1; its ratings have been determined by Palmström (1995a) from trial and error of various angles and joint densities.

In practice, each joint is multiplied by the value of ( $f_i$ ) for the actual angle interval. After some training it should be possible to quickly determine the intervals in Table A1 for the angle  $\delta_i$ . By applying these intervals the strong influence of the smallest angles, i.e. angles parallel or nearly parallel to the observation plane or bore hole, are removed

TABLE A1 SELECTED INTERVALS OF THE ANGLE ( $\delta_i$ ) AND THE CORRESPONDING FACTOR ( $f_i$ ) (from Palmström, 1995)

| angle $\delta_i$ | factor $f_i$ ( $= 1/\sin\delta_i$ ) |
|------------------|-------------------------------------|
| $> 60^\circ$     | 1                                   |
| 31 - $60^\circ$  | 1.5                                 |
| 16 - $30^\circ$  | 3.5                                 |
| $< 16^\circ$     | 6                                   |

As the weighted joint density method reduces the inaccuracy caused by the orientation of the observation surface or bore hole, it leads to a better characterization of the rock mass, which in turn may result in a reduced amount of bore holes required in an investigation.

The weighted joint density is approximately equal to the volumetric joint count, i.e.  $J_v \approx wJd$ .

## A6 BLOCK TYPES AND SHAPES

Methods to determine the block shape factor  $\beta$  given in eq. (A-4) and its characterization are described in this section. The *type and shape* of blocks are determined by:

- the number of joint sets;
- the differences in joint spacings; and
- the angles between the joints or joint sets.

For a rock mass with 3 joint sets intersecting at right angles the values of  $\beta$  are given in Figure A3. The types of blocks delineated by joints have in the literature been characterized in different ways and by different terms. Where relatively regular jointing exists, it

may be possible to give adequate characterization of the jointing pattern according to the system presented by Dearman (1991). In most cases, however, there is no regular jointing pattern; a rough characterization of the blocks is therefore generally more practical, for example a division into three main groups only, as presented by Sen and Eissa (1991). The terms applied by Palmström (1995a) are shown in Figure A3. For  $\beta = 27$  to 32 the block term 'compact' has been introduced to include cubical, equidimensional, blocky and other existing terms for blocks not being long or flat.

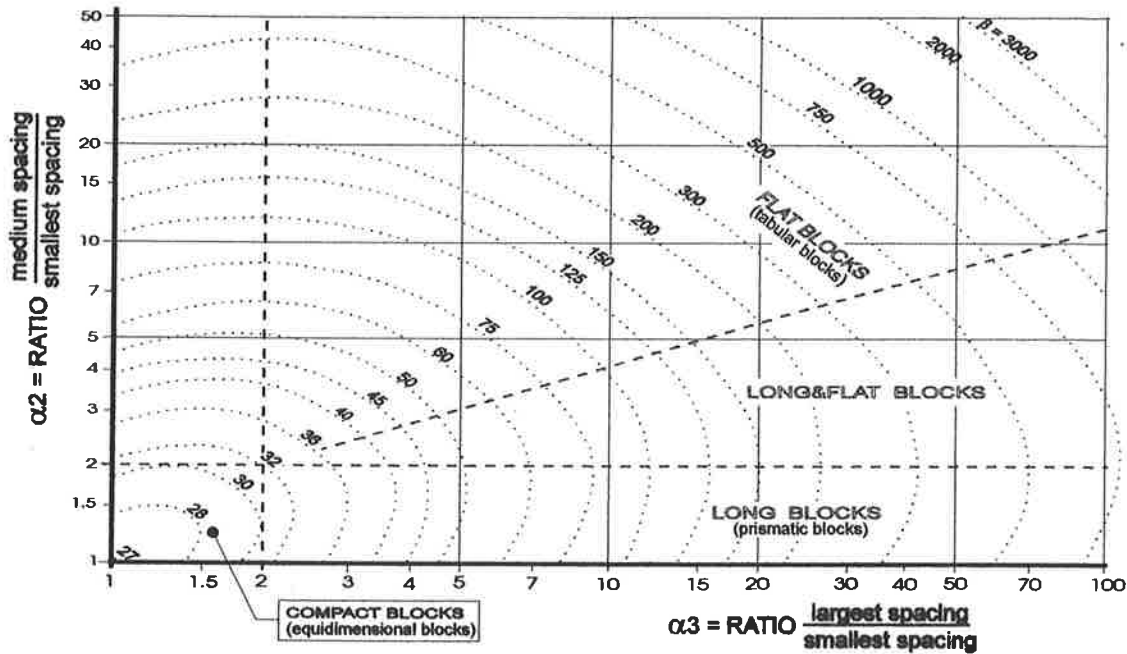


Fig. A3 Block types characterized by the block shape factor ( $\beta$ ) found from the ratio between spacings of the joint sets. The data are based on 3 joint sets intersecting at right angles (from Palmström, 1995a). Example: For  $\alpha_2 = 4$  and  $\alpha_3 = 15$ ,  $\beta = 135$ .

The use of Figure A3 requires 3 joint sets. As blocks often have more than six faces or have irregular shape, it can be difficult to estimate  $\beta$ . Therefore, the following simplified method to estimate  $\beta$  has been developed by Palmström (1995a), in which the longest and shortest dimension of the block are applied:

$$\beta = 20 + 7 a_3/a_1 = 20 + 7 \alpha_3 \quad \text{eq. (A-10)}$$

where  $a_3$  and  $a_1$  are the longest and shortest dimension of the block.

The evaluations made by Palmström (1995a) have shown that eq. (A-10) covers most types of blocks (where  $\beta < 1000$ ) within reasonable accuracy ( $\pm 25\%$ ). For very flat to extremely flat blocks (see Figure A3) eq. (A-9) has limited accuracy.

Where  $\beta$  is not known, it is recommended to use a 'common' value of  $\beta = 40$ .

#### A7 CORRELATION BETWEEN THE RQD AND THE VOLUMETRIC JOINT COUNT ( $J_v$ )

It is not possible to obtain good correlations between RQD and  $J_v$  or between RQD and other measurements of jointing. Palmström (1982) presented the following simple expression:

$$J_v = 35 - 0.3 \text{ RQD} \quad \text{eq. (A-11)}$$

Especially where many of the core pieces have lengths around 0.1 m, the correlation above may be inaccurate. However, when RQD is the only joint data available, eq. (A-11) has been found to be the best simple transition from RQD via  $J_v$  to block volume.

The block volume ( $V_b$ ) can be found from the volumetric joint count ( $J_v$ ) using input of the block shape factor ( $\beta$ ) (see eqs. (A-3a) and (A-3b)).

#### A8 METHODS TO FIND AN EQUIVALENT BLOCK VOLUME WHERE JOINTS DO NOT DELIMIT BLOCKS

As mentioned in Section A1 a minimum of three joint sets in different directions are theoretically necessary to delimit blocks in a rock mass. There are, however, cases with irregular jointing where blocks are formed mainly from random joints, and other cases where the blocks are delimited by one or two joint sets and additional random joints. In cases where the jointing is composed of one or two joint sets with no or few random joints, the joints do not define individual blocks. In such cases an *equivalent block volume* is used in the calculations of  $RM_i$ . Such block volume may be found from one of the following methods:

1. Where only *one joint set* occurs, the equivalent block volume may be considered to be similar to the area of the joint plane<sup>12)</sup> multiplied by distance between the two joints:  $V_b = L^2 \cdot S$  Here  $L$  is the joint length and  $S$  is the spacing between the joints. [Example: For foliation partings with lengths  $L = 0.5 \text{ m}$  to  $2 \text{ m}$  and average joint spacing  $S = 0.2 \text{ m}$ , the equivalent block volume will vary between  $V_b = S \cdot L^2 = 0.2 \cdot 0.5^2 = 0.05 \text{ m}^3$  and  $V_b = 0.2 \cdot 2^2 = 0.8 \text{ m}^3$ ]
2. For *two joint sets* the spacing for the two sets ( $S_1$  and  $S_2$ ) and the length ( $L$ ) of the joints can be applied:  $V_b = S_1 \cdot S_2 \cdot L$
3. For most cases the equivalent block volume can be found from eq. (A-3b) [ $V_b = \beta \cdot J_v^{-3}$ ] which requires input from the block shape factor ( $\beta$ ).  $\beta$  can be estimated from eq. (A-10)<sup>13)</sup> [ $\beta = 20 + 7 a_3/a_1$ ]

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<sup>12)</sup> Here is assumed that the joint plane is circular, i.e.  $A = \pi \cdot L^2/4 \approx L^2$

A method to arrive at a better estimate of  $\beta$  using the length and spacing of the joints, is given in the following:

Eq. (A-4) was developed for defined blocks formed by three joint sets. Where less than three sets occur,  $\beta$  can be adjusted by a factor  $n_j$ , which represents a rating for the actual number of joint sets, to characterize an *equivalent* block shape factor:

$$\beta = 20 + 7 (S_{\max}/S_{\min})(3/n_j) = 20 + 21 S_{\max} / (S_{\min} \cdot n_j) \quad \text{eq. (A-12)}$$

The ratings of  $n_j$  are given as:

|                              |             |                  |           |
|------------------------------|-------------|------------------|-----------|
| 3 joint sets + random        | $n_j = 3.5$ | 3 joint sets     | $n_j = 3$ |
| 2 joint sets + random joints | 2.5         | 2 joint sets     | 2         |
| 1 joint set + random joints  | 1.5         | 1 joint set only | 1         |

4. For small discontinuities (fissures, partings and small joints) for which the lengths can be measured or easily estimated, the length and spacing of the joints correspond to the longest and shortest block dimension, hence the ratio length/spacing =  $L/S$  can replace  $S_{\max}/S_{\min}$  in eq. (A-12):  $\beta = 20 + 21 L / (S \cdot n_j)$  eq. (A-13)

For long joints it is often sufficiently accurate to use a length  $L = 4$  m.

#### A8.1 Example

For one joint set ( $n_j = 1$ ) spaced at  $S_1 = 0.2$  m with an average joint length  $L_1 = 2$  m, the block shape factor according to eq. (A-13) is  $\beta = 20 + 21 L_1 / (S_1 \cdot n_j) = 230$ .

The volumetric joint count for this set is  $J_v = 1/S_1 = 5$  which gives

$$V_b = \beta \cdot J_v^{-3} = 1.84 \text{ m}^3.$$

[For a defined block limited by 3 joints sets crossing at right angles with spacings  $S_1, L_1, L_1$ , the volume is  $V_b = 0.2 \cdot 2 \cdot 2 = 0.8 \text{ m}^3$ ]

<sup>13)</sup> As the volumetric joint count can be measured also where joints do not delimit defined blocks, this approach may be applied where few joints sets are found.



## BRUK AV Q-METODEN I TBM TUNNELER, VED FELTKARTLEGGING OG BORKJERNELOGGING

### The Q-method used in TBM tunnels, field mapping and core logging

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#### INTRODUCTION

The Q-method is a system for classification of rock masses related to stability. From six different parameters a Q-value can be calculated using the following formula:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

The parameters are:

|       |   |                                       |
|-------|---|---------------------------------------|
| RQD   | = | Rock Quality Designation (Deere 1962) |
| $J_n$ | = | Joint set number                      |
| $J_a$ | = | Joint alteration number               |
| $J_r$ | = | Joint roughness number                |
| $J_w$ | = | Joint water reduction factor          |
| SRF   | = | Stress reduction factor               |

The different parameter values are estimated during geological mapping according to a table with descriptions, see Table 1. A high Q-value indicates good stability and a low Q-value indicates poor stability.

The Q-method was developed more than 20 years ago (Barton *et al.*, 1974) and has since been revised (Grimstad and Barton, 1993). It is now one of the most widely used classification systems in the world for design and support estimation for all types of tunnels and caverns. The support recommendation in the Q-system is based on empirical data, and in the last revision more than 1000 new examples from recently constructed tunnels are included.

The Q-method was developed from data on tunnels and caverns excavated by drill and blast. Later on, data from TBM tunnels has also been included. The method is also useful during pre-investigation, *i.e.*, field mapping and core logging. However, one should bear in mind that the Q-method can not always be used without reservations in TBM tunnels, in field mapping and in core logging.

Table 1 Q-parameter values

| 1. Rock Quality Designation                                                                                                                                                                        |           | RQD      |
|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------|----------|
| A                                                                                                                                                                                                  | Very poor | 0 - 25   |
| B                                                                                                                                                                                                  | Poor      | 25 - 50  |
| C                                                                                                                                                                                                  | Fair      | 50 - 75  |
| D                                                                                                                                                                                                  | Good      | 75 - 90  |
| E                                                                                                                                                                                                  | Excellent | 90 - 100 |
| Note: i) Where RQD is reported or measured as $\leq 10$ (including 0), a nominal value of 10 is used to evaluate Q.<br>ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate. |           |          |

| 2. Joint Set Number                                                                         |                                                                      | $J_n$     |
|---------------------------------------------------------------------------------------------|----------------------------------------------------------------------|-----------|
| A                                                                                           | Massive, no or few joints                                            | 0.5 - 1.0 |
| B                                                                                           | One joint set                                                        | 2         |
| C                                                                                           | One joint set plus random joints                                     | 3         |
| D                                                                                           | Two joint sets                                                       | 4         |
| E                                                                                           | Two joint sets plus random joints                                    | 6         |
| F                                                                                           | Three joint sets                                                     | 9         |
| G                                                                                           | Three joint sets plus random joints                                  | 12        |
| H                                                                                           | Four or more joint sets, random, heavily jointed, "sugar cube", etc. | 15        |
| J                                                                                           | Crushed rock, earthlike                                              | 20        |
| Note: i) For intersections, use $(3.0 \times J_n)$<br>ii) For portals, use $2.0 \times J_n$ |                                                                      |           |

| 3. Joint Roughness Number                                                                                                                                                                                                      |                                                                           | $J_r$ |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------|-------|
| a) Rock-wall contact, and b) rock-wall contact before 10 cm shear                                                                                                                                                              |                                                                           |       |
| A                                                                                                                                                                                                                              | Discontinuous joints                                                      | 4     |
| B                                                                                                                                                                                                                              | Rough or irregular, undulating                                            | 3     |
| C                                                                                                                                                                                                                              | Smooth, undulating                                                        | 2     |
| D                                                                                                                                                                                                                              | Slickensided, undulating                                                  | 1.5   |
| E                                                                                                                                                                                                                              | Rough or irregular, planar                                                | 1.5   |
| F                                                                                                                                                                                                                              | Smooth, planar                                                            | 1.0   |
| G                                                                                                                                                                                                                              | Slickensided, planar                                                      | 0.5   |
| Note: i) Descriptions refer to small scale features and intermediate scale features, in that order.                                                                                                                            |                                                                           |       |
| c) No rock-wall contact when sheared                                                                                                                                                                                           |                                                                           |       |
| H                                                                                                                                                                                                                              | Zone containing clay minerals thick enough to prevent rock-wall contact   | 1.0   |
| J                                                                                                                                                                                                                              | Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact | 1.0   |
| Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.<br>ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength. |                                                                           |       |

| 4. Joint Alteration Number                                      |                                                                                                                                                                               | $\phi_i$<br>approx. | $J_a$            |
|-----------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------|------------------|
| a) Rock-wall contact (no mineral fillings, only coatings)       |                                                                                                                                                                               |                     |                  |
| A                                                               | Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote                                                                                             |                     | 0.75             |
| B                                                               | Unaltered joint walls, surface staining only                                                                                                                                  | 25-35°              | 1.0              |
| C                                                               | Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.                                                             | 25-30°              | 2.0              |
| D                                                               | Silty- or sandy-clay coatings, small clay fraction (non-softening)                                                                                                            | 20-25°              | 3.0              |
| E                                                               | Softening or low fraction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.                | 8-16°               | 4.0              |
| b) Rock-wall contact before 10 cm shear (thin mineral fillings) |                                                                                                                                                                               |                     |                  |
| F                                                               | Sandy particles, clay-free disintegrated rock, etc.                                                                                                                           | 25-30°              | 4.0              |
| G                                                               | Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5mm thickness)                                                                              | 15-24°              | 6.0              |
| H                                                               | Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5mm thickness)                                                                          | 12-16°              | 8.0              |
| J                                                               | Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5mm thickness). Value of $J_a$ depends on percent of swelling clay-size particles, and access to water, etc. | 6-12°               | 3-12             |
| c) No rock-wall contact when sheared (thick mineral fillings)   |                                                                                                                                                                               |                     |                  |
| KLM                                                             | Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)                                                                      | 6-24°               | 6, 8, or 3-12    |
| N                                                               | Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)                                                                                                   |                     | 5.0              |
| OPR                                                             | Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)                                                                                      | 6-24°               | 10, 13, or 13-20 |

| 5. Joint Water Reduction Factor                                                                                                                                     |                                                                                 | approx. water pres. (kg/cm <sup>2</sup> ) | $J_w$    |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------|-------------------------------------------|----------|
| A                                                                                                                                                                   | Dry excavations or minor inflow, i.e., < 5 l/min locally                        | < 1                                       | 1.0      |
| B                                                                                                                                                                   | Medium inflow or pressure, occasional outwash of joint fillings                 | 1-2.5                                     | 0.65     |
| C                                                                                                                                                                   | Large inflow or high pressure in competent rock with unfilled joints            | 2.5-10                                    | 0.5      |
| D                                                                                                                                                                   | Large inflow or high pressure, considerable outwash of joint fillings           | 2.5-10                                    | 0.33     |
| E                                                                                                                                                                   | Exceptionally high inflow or water pressure at blasting, decaying with time     | > 10                                      | 0.2-0.1  |
| F                                                                                                                                                                   | Exceptionally high inflow or water pressure continuing without noticeable decay | > 10                                      | 0.1-0.05 |
| Note: i) Factors C to F are crude estimates. Increase $J_w$ if drainage measures are installed.<br>ii) Special problems caused by ice formation are not considered. |                                                                                 |                                           |          |

| 6. Stress Reduction Factor                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                |                                                                                                                                  | SRF    |          |         |
|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------|--------|----------|---------|
| a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                |                                                                                                                                  |        |          |         |
| A                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth) |        | 10       |         |
| B                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq 50$ m)                         |        | 6        |         |
| C                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m)                               |        | 2.5      |         |
| D                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)                                           |        | 7.5      |         |
| E                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Single shear zones in competent rock (clay-free) (depth of excavation $\leq 50$ m)                                               |        | 5.0      |         |
| F                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Single shear zones in competent rock (clay-free) (depth of excavation > 50m)                                                     |        | 2.5      |         |
| G                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)                                                            |        | 5.0      |         |
| Note: i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             |                                                                                                                                  |        |          |         |
| b) Competent rock, rock stress problems                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                                                                                                                                  |        |          |         |
| H                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Low stress, near surface, open joints                                                                                            | > 200  | < 0.01   | 2.5     |
| J                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Medium stress, favourable stress condition                                                                                       | 200-10 | 0.01-0.3 | 1       |
| K                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.                      | 10-5   | 0.3-0.4  | 0.5-2   |
| L                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Moderate slabbing after > 1 hour in massive rock                                                                                 | 5-3    | 0.5-0.65 | 5-50    |
| M                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Slabbing and rock burst after a few minutes in massive rock                                                                      | 3-2    | 0.65-1   | 50-200  |
| N                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock                                               | < 2    | > 1      | 200-400 |
| Note: i) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$ , reduce $\sigma_3$ to $0.75\sigma_3$ . When $\sigma_1/\sigma_3 > 10$ , reduce $\sigma_3$ to $0.5\sigma_3$ , where $\sigma_3$ = unconfined compression strength, $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses, and $\sigma_3$ = maximum tangential stress (estimated from elastic theory).<br>ii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).<br>c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure |                                                                                                                                  |        |          |         |
| O                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Mild squeezing rock pressure                                                                                                     |        | 1-5      | 5-10    |
| P                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Heavy squeezing rock pressure                                                                                                    |        | > 5      | 10-20   |
| Note: iv) Cases of squeezing rock may occur for depth $H > 350 Q^{1/3}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $q = 0.7 \gamma Q^{1/3}$ (MPa) where $\gamma$ = rock density in kN/m <sup>3</sup> (Singh, 1993).<br>d) Swelling rock: chemical swelling activity depending on presence of water                                                                                                                                                                                                                                                                                                                                                        |                                                                                                                                  |        |          |         |
| R                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Mild swelling rock pressure                                                                                                      |        |          | 5-10    |
| S                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         | Heavy swelling rock pressure                                                                                                     |        |          | 10-15   |
| Note: $J_r$ and $J_a$ classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, $r$ (where $r = \sigma_n \tan^{-1} (J_r/J_a)$ ). Choose the most likely feature to allow failure to initiate.                                                                                                                                                                                                                                                                                                                                                                                   |                                                                                                                                  |        |          |         |
| $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                  |        |          |         |

Another important thing to have in mind is that the Q-system is a system for *rock mass* classification (as are most other classification systems) and therefore cannot be used for evaluation of the stability of single large unstable wedges or blocks. The stability of single blocks is dependent on additional parameters. The Q-values describe the general stability conditions for a section of a tunnel. They can be used to determine what the mean bolt spacing should be in a tunnel; not, however, the exact position of each bolt.

## TBM TUNNELS

Engineering geological mapping is usually more difficult in a TBM tunnel than in a tunnel excavated by drill-and-blast methods since the walls in a TBM tunnel are as a rule quite smooth. It is therefore difficult to study the joint faces in order to make use of the Q-method. However, use of a classification system is always advantageous, since it focuses attention on key parameters.

Blasting will often open the joints around a tunnel excavated by the drill-and-blast method. This does not occur in a TBM tunnel, where most of the joints seem to be closed, and it may therefore be difficult to say what is a real joint and what is a mineral vein or a foliation plane. The calculation of RQD may therefore be difficult. It is also difficult to study the joint surfaces due to the usual lack of outbreak so  $J_r$  and  $J_a$  may be difficult to evaluate. To find out if there is any filling of clay or another soft mineral in a joint, it is often useful to try to press a thin knife into the joint. It is also important to study joint surfaces where they are found, *i.e.*, where blocks have fallen down. Joints belonging to a specific set will often have a characteristic roughness and filling.

In a drill-and-blast tunnel most of the loose blocks will fall down during the blasting. This will not occur to the same degree in a TBM tunnel. In a TBM tunnel potentially unstable blocks may be found even if the Q-values are high. These blocks may fall down at any time and will represent a danger for people working in the tunnel. Therefore the mapping must be carried out with great care, and in addition to the general Q-classification, the joint geometry must be studied carefully to determine whether any unstable blocks exist.

## FIELD MAPPING

Field mapping is usually an important part of the pre-investigations for tunnels, and the Q-system can with advantage be used during such mapping. Of course there may be uncertainties regarding some of the Q-parameters during the planning stage. This will among other things depend on the exposures. If there are only a few or small rock exposures, these exposures usually will represent only the rock masses of best quality, whereas more jointed rocks and fault zones will be covered by soil. The surface rocks may also be weathered and therefore generally of poorer quality than the rock masses at the tunnel level.

The Q-parameters RQD,  $J_n$  and  $J_r$  can usually be successfully estimated during field mapping.  $J_a$  may be underestimated since joint fillings of clay may easily have been washed away near the surface.  $J_w$  and SRF are related to the tunnel location and may be more difficult to estimate. If no plans for a tunnel exist,  $J_w$  and SRF often are given the value 1, and in that way a form of "maximum Q-value" is calculated. If some information about the permeability of the rock masses is available, for example from boreholes or from nearby tunnels, it may be possible to make an estimate of  $J_w$ . The stress reduction factor, SRF, may be estimated from the overburden of a tunnel or determined from stress measurements. If the planned tunnel is to cross marked fault zones, a SRF-value of 5-10 may be relevant for some sections. Rocks with swelling or squeezing properties may also be given rather high SRF-values.

## CORE DRILLING

Core drilling is often a part of the pre-investigations for tunnels and caverns. It is very important to make a good description of the cores, and therefore the Q-system should be used during the core logging.

Since RQD is normally defined from drill cores, this parameter value can be measured correctly during the logging. This RQD value is only along the direction of the borehole, and may not be quite representative for the rock mass if for example the main joint set is parallel to the borehole.  $J_n$  can be estimated by noting the angles between the joints and the cores and their different character, but joints nearly parallel to the borehole will be underrepresented in the cores. In order to find a correct  $J_n$  value it is advantageous to make comparison with field mapping from the area around the borehole. Whereas RQD is often calculated for one or two metres of core,  $J_n$  must be estimated for sections of several metres. The fact that one joint set is missing in sections of one or two metres, will not necessarily mean that this joint set is missing in the rock mass. Joints nearly parallel to the borehole may seldom be seen in the cores even if the joint spacing is rather small, see Figure 1.

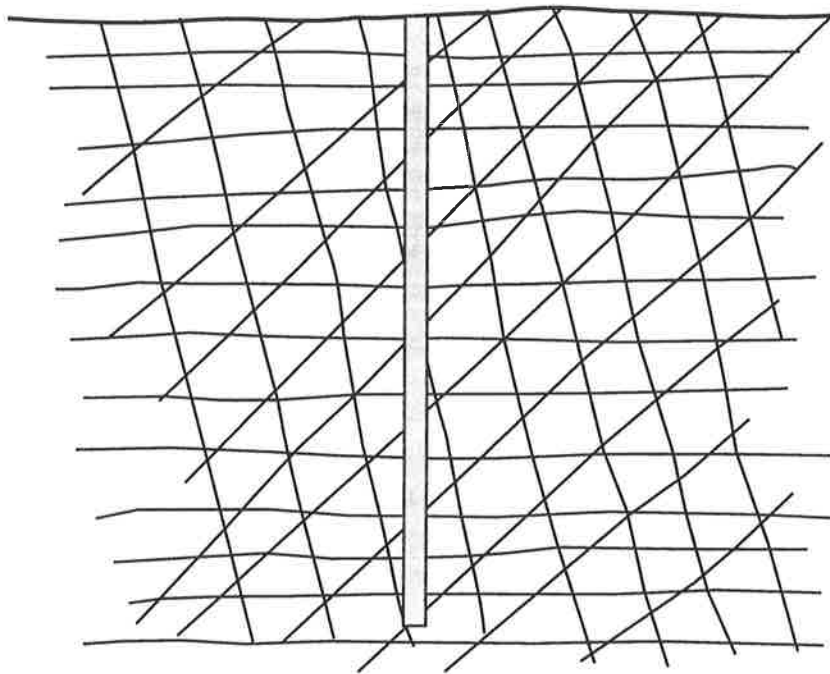


**Figure 1** Borehole in rock masses with three joint sets where one of the sets is nearly parallel to the borehole.  
*Borhull i bergmasser med tre sprekkesett hvor det ene settet er nær parallelt med borhullet.*

The small scale roughness of the joint surfaces can be studied in the cores, but since the joint samples found in the cores are usually only a few centimetres or decimetres long, the larger scale undulations are often difficult to estimate. The  $J_r$  values found during core logging may therefore be subject to some uncertainties, but will generally be conservative.

As for field mapping there will also be some uncertainties concerning the  $J_a$  values. Since water usually is used during the drilling, joint fillings of clay may have been washed away.

Many of the same reservations as for field mapping will also concern  $J_w$  and SRF values since these parameters are related to the tunnel location. Permeability tests are often carried out in conjunction with core drilling, and the results from such tests will be used for to estimate  $J_w$ . SRF depends among other things on overburden, fault zones and the presence of swelling or squeezing rocks; and by using the data from the cores it is often possible to make a preliminary estimate of the SRF value.

In connection with the Q-logging the cores must be divided into structural units of suitable length. The Q-value is then calculated for each unit. This means that there

should not be considerable variation in rock mass quality within each unit. On the other hand the unit should at least have a length of some metres to give a correct picture of the rock mass conditions. If there are marked weakness zones in rock masses of rather good quality, these zones must be given their own Q-value even if they are only one metre thick. Experience is required in evaluating whether the better quality wall rock should be given a reduced Q-value due to a fault-related SRF value at this location, *i.e.*, if loosening could occur during the construction due to this feature.

Sometimes parts of the cores are missing. This is often caused by poor rock quality, *i.e.*, the rock is crushed or altered to clay. Where cores are missing, it is of course impossible to estimate the Q-value; but probably the quality will be poorer than where cores are found. Usually RQD will be 0 (10 in the Q-system) where cores are missing,  $J_n$  will often be 15-20,  $J_r$  often 1 and  $J_a$  at least 6. Sections of several metres of missing core may also indicate a SRF-value of 5-10.

Generally it is very important during the planning stage of a tunnel to combine all available data from different sources such as field mapping, core drilling, geophysical measurements and empirical data from other tunnels. In such cases it is very advantageous to use a classification system such as the Q-method. To present the data, a spreadsheet as shown in Figure 2 may be useful. Here the variation in the different Q-parameters is shown by histograms, and the maximum, minimum and mean Q-values are calculated. From this diagram it is also possible to make an estimate of the percentage of the different quality classes.

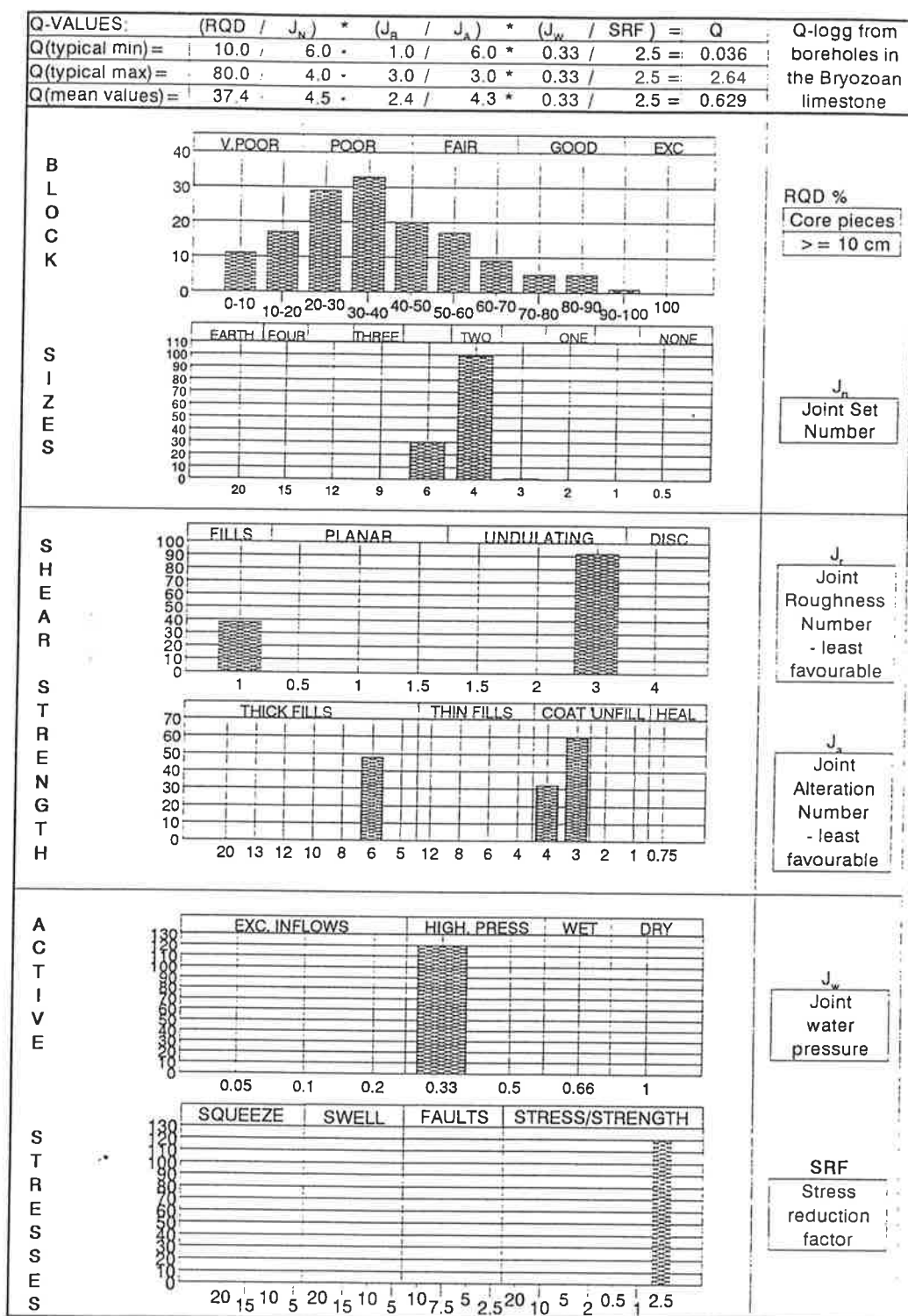


**Figure 2** Spreadsheet presenting core logging data from Bryozoan limestone at Malmø

Diagram som viser kjerneloggingsdata fra Bryozokalk i Malmø

## EXAMPLES

### VEAS Sewage Tunnel, Norway

The central sewage treatment plant for Oslo is situated 30 km west of the centre of the city. The sewage is transported to the plant through a tunnel system. Most of these tunnels have been excavated by TBM and have a diameter of 3-3.5m. Usually stability problems only occurred in connection with fault zones. However, at some locations block falls occurred where the rock mass quality seemed to be rather good. In one case a 7m long wedge fell down a few days after the head of the TBM had passed, see Figure 3.

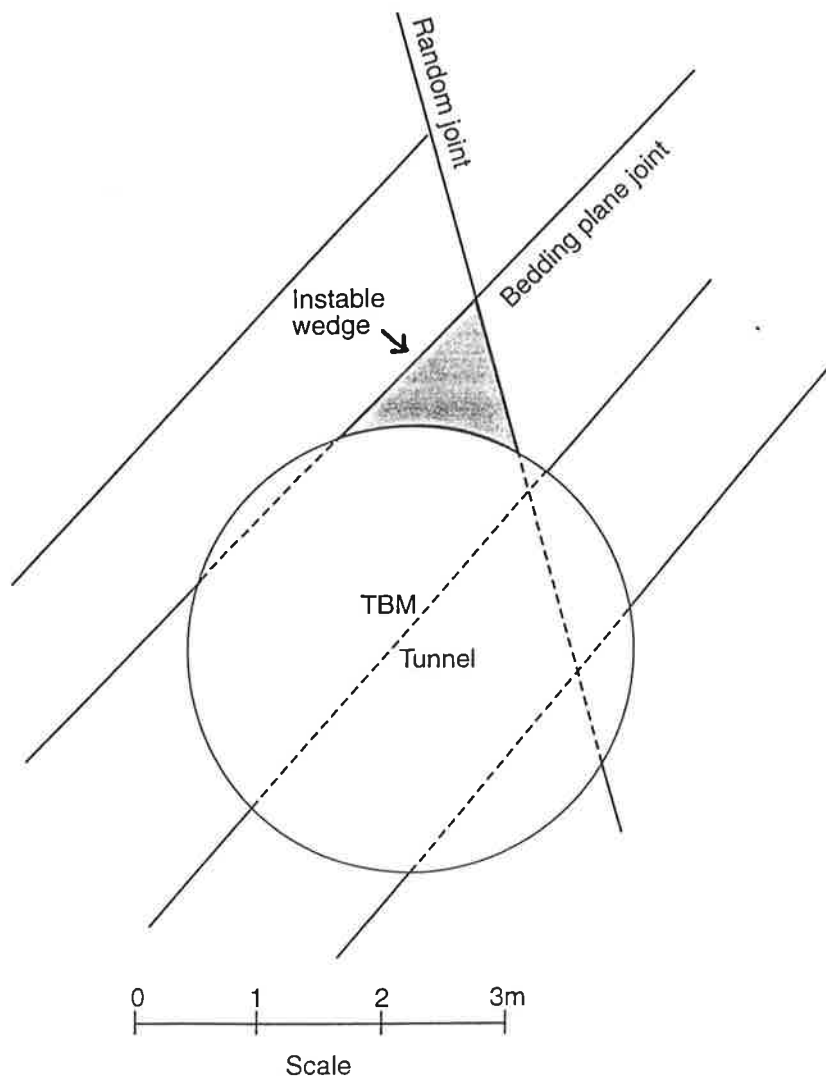


**Figure 3** *Unstable wedge in a sewage tunnel*  
*Ustabil kile i kloakktunnel*

At this location the rock was Ordovician shale and the tunnel direction was nearly parallel to the strike direction in the shale. The wedge was limited by a bedding plane joint, vertical joints at right angles to the strike direction of the bedding and a random joint with the same strike direction as the bedding but with opposite dip direction. The joints forming the wedge were generally undulating, but smooth, with a thin coating containing some clay. No water or stress problems seemed to occur in the tunnel. From this the following Q-value was found:

$$Q = \frac{90}{6} \times \frac{2}{4} \times \frac{1}{1} = 7.5$$

A Q-value of 7.5 in this type of tunnel generally means no support. A comment to the Q-value may be that the special wedge was completely de-stressed and therefore  $SRF = 2.5$  could possibly have been used instead of 1. Another point is that the mechanism of the block fall was not sliding along a joint surface, but the wedge fell directly down, and therefore the parameters  $J_r$  and  $J_a$  are rather irrelevant. The problem here, however, was not the Q-system, but to locate the wedge before it fell down. Generally the Q-value was correct, and no systematic support was needed. In a drill-and-blast tunnel the wedge would surely have fallen down during the blasting. This shows the need for a very exact mapping in a TBM tunnel, and in addition to the Q-classification, a geometrical study of the joint pattern is also needed to locate unstable, single blocks.

### **Svartisen Road Tunnel, Norway**

Svartisen road tunnel between Glomfjord and Kilvik in northern Norway is 7.6 km long (Løset 1992). 4.4 km of the tunnel was excavated by TBM with a diameter of 6.3m. The TBM cross section was, however, too small, and the tunnel had to be widened by drill-and-blast to a span width of about 7m, see Figure 4. The rocks in this part of the tunnel are meta-sediments mainly mica gneiss and marble.

This tunnel provided a rather unique opportunity to study the relation between TBM boring and drill-and-blast in exactly the same rock masses. A Q-classification of the TBM tunnel was carried out and then repeated after widening by drill-and-blast. The results showed that the Q-value was reduced by the drill-and-blast. This reduction in Q-value only occurred in rock masses of medium quality, *i.e.*  $Q = 4-30$ . The average Q-value in the TBM tunnel was found to be 20.5, but this value was reduced to 18.5 after the drill-and-blast. The reason for this was that the blasting had created new fractures, and weakness planes had been transformed to open joints giving a reduction of the RQD value. By definition, only natural joints shall be taken into consideration when calculating RQD. However, around a tunnel all fractures will influence the

stability, and therefore both natural and artificial fractures must be included in the calculation of RQD for use in the Q-system.

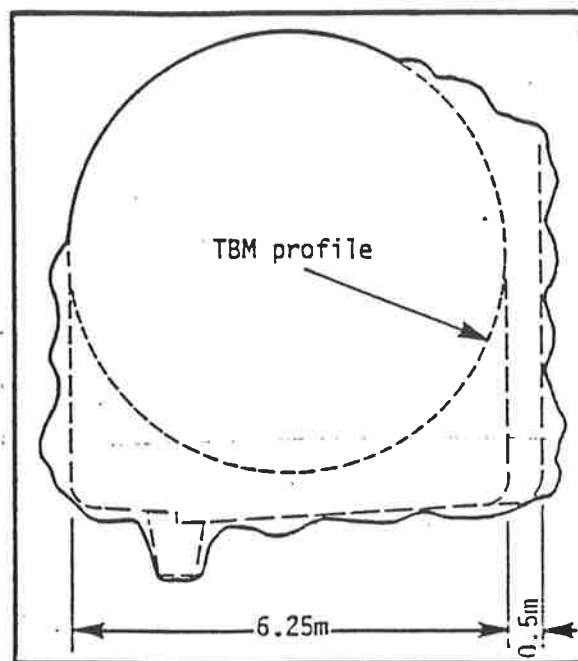


**Figure 4** *Cross-section of the Svartisen road tunnel*  
*Tverrsnitt av Svartisen vegtunnel*

Another interesting topic here was the stress situation. As the rock overburden was up to 1100m, rather high stresses were expected. Shortly after excavation there was usually no sign of spalling, slabbing or rock burst tendencies, and therefore SRF was visually estimated to 1. However, after some weeks or months rock stress caused spalling in some sections, and the SRF value had to be changed to 2.5. The visually estimated SRF value was dependent on when the mapping was done.

#### **Äspö Hard Rock Laboratory, Sweden**

The ZEDEX project carried out at Äspö is an *in situ* study of the excavation disturbed zone for TBM and drill-and-blast excavation. The experiment was carried out in two parallel tunnels of 5m span at a depth of 420m below the surface. One tunnel was excavated by drill-and-blast and the other by TBM. NGI, under contract for UK Nirex Ltd, has, in collaboration with SKB, made a Q-classification of the two tunnels and of more than 800m of drill cores from the rock masses around the tunnels. This project provided a good opportunity to compare Q-logging from a TBM tunnel, from a drill-and-blast tunnel and from drill cores all belonging basically to the same general

volume of rock masses. Unfortunately there was not much variation in the good quality rock in this area.

**Table 2** Average values for  $Q$  and  $Q$ -parameters.

|                            | $Q$  | RQD | $J_n$ | $J_r$ | $J_a$ | $J_w$ | SRF |
|----------------------------|------|-----|-------|-------|-------|-------|-----|
| Drill cores (835m)         | 24.0 | 97  | 4     | 2.7   | 1.8   | 0.66  | 1   |
| TBM tunnel (50m)           | 22.0 | 99  | 3.7   | 2.5   | 2     | 0.66  | 1   |
| Drill & blast tunnel (38m) | 21.5 | 98  | 3.8   | 2.4   | 1.9   | 0.66  | 1   |

As seen in the above table, only small differences in values were registered between the tunnels and the drill cores. These differences may be caused by the fact that not exactly the same rock masses had been classified. On the other hand, the average  $Q$ -value is lowest in the drill-and-blast tunnel which should be expected since the blasting would have created new fractures. It should also be taken into consideration that there may be small errors in the parameter values for the drill cores and the TBM tunnel. In the drill cores there were some uncertainties concerning the  $J_n$ ,  $J_r$  and  $J_a$ , and in the TBM tunnel the estimation of  $J_r$  was difficult since almost no joint surface could be studied in detail. The conclusion is, however, that the  $Q$ -method can be used with a high degree of accuracy both in evaluating TBM tunnels and drill cores.

### **The Copenhagen and Bryozoon Limestone in Copenhagen and Malmø**

NGI has in collaboration with VBB VIAK, DGU and DGI been involved in pre-investigations for two tunnel projects in the Copenhagen and Bryozoon limestone. These projects are the City tunnel in Malmø in Sweden and a metro project in Copenhagen. The investigations have consisted of drill core logging and some geological mapping in excavations. This type of limestone has a bedded structure with alternating hard and soft beds. Some thin beds rich in flint also occur. The thickness of each beds varies, but is usually 0.1-2m. The use of the  $Q$ -method in such a rock type obviously presents some principal problems. The hard and soft beds are very different and have very different  $Q$ -values if considered separately. A single bed, however, is usually too thin to be considered as unit in the  $Q$ -logging. Sections consisting of several beds have therefore been used as logging units.

The stability of a tunnel in such rock will depend on where the soft beds will cut through the tunnel. Since the bedding is almost horizontal, a soft bed in the tunnel wall would only have a slight influence on the stability, but a soft bed in the roof would be very unfavourable. Therefore the soft beds have been considered as weakness zones during the  $Q$ -classification. In such weakness zones shear strength ( $J_r/J_a$ ) is evaluated as no rock-to-rock contact, which gives a  $J_r$  value of 1 and a  $J_a$  value of at least 6. These values for  $J_r$  and  $J_a$  have been used in logging units

containing soft beds. In the hard beds the joints are rough with a thin sandy/silty filling and  $J_r$  and  $J_a$  have therefore usually both been estimated to be 3.

Most of the joints seen in the vertical drill cores are bedding plane joints. Field observations indicate that more or less vertical joints also exist, and this fact has been kept in mind during the logging.  $J_n$ -values of 4-6 have often been used. RQD can be measured in the cores, but very soft beds classified as incompetent must be given a RQD value of 0 (10 in the Q-system) even if no joints occur. Since the tunnels in question will have little rock overburden and the soft beds are considered as weakness zones, a SRF value of 2.5-5 will be relevant. Results of the core logging are shown in Figure 2.

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## SAMMENDRAG

Q-metoden er et av de mest brukte klassifikasjonssystemer for bergmasser med hensyn til stabilitet i tunneler og bergrom. Ved å vurdere 6 bergmasseparametre kan en Q-verdi regnes ut, og denne verdien kan brukes ved stabilitetsvurdering og bestemmelse av sikring.

Opprinnelig er Q-metoden bygget opp på grunnlag av data fra sprengte tunneler. Senere er det også kommet inn en betydelig mengde data fra TBM tunneler. Q-metoden kan også brukes ved forundersøkelser for bergrom, det vil si ved feltkartlegging og borkjerne logging, men man må være oppmerksom på at det ved slike undersøkelser kan være vanskelig å bestemme enkelte av parametrene slik at det kan bli en viss usikkerhet med hensyn til Q-verdiene.

Ved hjelp av Q-metoden får man en beskrivelse av en bergmasse, men dette betyr ikke at Q-verdien gir et uttrykk for stabiliteten av hver enkelt blokk i denne massen. Man kan derfor ha løse enkeltblokker i bergmasser som ut fra Q-verdien skulle ha god stabilitet. Dette kan spesielt være tilfelle i TBM tunneler fordi boringen av slike tunneler i liten grad vil påvirke bergmassene omkring, slik at løse blokker kan bli hengende igjen. I slike tunneler er det derfor viktig at man i tillegg til vanlig Q-klassifisering også gjør en nøyaktig generell geologisk kartlegging, slik at løse blokker kan oppdages. Q-klassifisering i TBM tunneler kan være vanskeligere enn i en sprengt tunnel fordi tunnelveggene oftest er helt glatte, og det er derfor vanskelig å studere sprekkflatene.

Ved feltkartlegging og borkjernelogging kan bestemmelsen av de enkelte parameterverdiene være forbundet med en viss usikkerhet. I en borkjerne ser man oftest bare noen få centimeter av en sprekkeflate, og ruhet i større skala kan derfor være vanskelig å vurdere. Da det brukes vann under boringen, kan sprekkefyllinger av leire bli vasket vekk, og parameterverdien for sprekkgeomvandling kan derfor være vanskelig å bestemme. Det samme kan være tilfellet ved feltkartlegging da man ofte får en utvasking av sprekkene nær overflaten. Til tross for disse usikkerhetene vil det alltid være en fordel å bruke Q-systemet både ved feltkartlegging og kjernelogging.

I eksemplene blir de forskjellige bruksområdene for Q-metoden belyst. I kloakktunneler i Osloområdet boret med TBM ble metoden systematisk brukt. Det samme gjaldt i Svartistunnelen i Nord-Norge som først ble boret med TBM, og deretter ble tverrsnittet utvidet ved sprengning.

Ved Äspölaboratoriet i Sverige ble Q-verdiene bestemt i borkjerner, i en TBM tunnel og i en sprengt tunnel. I forbindelse med tunnelprosjekter i Malmø og København er Q-metoden brukt ved logging av borkjerner fra lagdelte kalksteiner hvor det er svært stor forskjell mellom de enkelte lagene.



# **KLASSIFICERING I INTERNATIONELLA KONTRAKT**

## **Classification systems used in international contracts**

*Gunnar Nord, Atlas Copco Rock Drills AB*

### ***Inledning:***

Man kan konstatera att vid undermarksprojekt som går till internationell budgivning nästan undantagslöst värderar bergförhållandena med hjälp av någon form av klassificering. Många gånger använder man klassificeringen som instrument för att fastställa ersättningsnivåerna till entreprenören men det är långt ifrån någon regel. Många gånger ges resultatet av klassificeringen endast som en upplysning till en budgivare utan några kontraktuella konsekvenser.

Uppenbart är emellertid att man finner det värdefullt att göra någon form av klassificering. Att nöja sig med en generell och ibland detaljerad geologisk beskrivning är nästan aldrig tillfyllest. Det är egentligen inte märkligt eftersom kostnadsvärderingen av projekten grundar sig på en mängdförteckning som är svår att upprätta utan att det föreligger någon form av värdering av undermarksförhållandena av typ klassificering.

Ambitionen är inte här att ge en heltäckande bild av hur klassificering tillämpas i internationella sammanhang utan belysa vilka problemställningar man fokusera intresset på.

### ***Klassificeringssystem***

Vid de flesta kontrakterade undermarksarbetena tillämpar man någon form av klassificering av bergförhållandena. Syftet med den är att ge en både kvantitativ och kvalitativ uppfattning om vilka förhållanden man kommer att möta. Väsentliga avvikelser i utfallet i förhållande till ställd prognos kan ge upphov till krav på ekonomisk ersättning från entreprenören. Det har ibland medfört att en del klienter eller beställare inte har velat se klassificeringen som en del av anbudshandlingarna utan endast för entreprenörens information. Det här är ett krypskytte som sällan lönar sig.

Vilka är då de discipliner som intresset fokuseras kring vid klassificering i internationella kontrakt. Med internationella kontrakt menas här kontrakt utformade så att inte enbart nationella företag har möjlighet att bjuda på arbetena. D v s de är utformade språkligt och kontraktuellt så att internationella entreprenörer kan ge anbud. I många fall är det en klart uttryckt önskan från ägaren (klienten) att internationella entreprenörer skall utföra arbetena eller åtminstone leda dem eftersom de nationella inte bedömdes kapabla att utföra arbetena till erforderlig kvalite och inom rimliga tidsramar. För att få med de kapabla internationella aktörerna i seriös anbudsgivning krävs en kvalificerad värdering av bergförhållandena och då utgör internationellt accepterade klassificeringar en väl framkomlig väg.

Vilka är då de egenskaper hos berget som man önskar klassificera. Nästan alltid förekommer en värdering av bergets stabilitet med avseende på det aktuella hålrummet. Det finns flera sätt att göra klassificeringen. I den tyskspråkiga världen lutar man sig gärna mot utvecklingen av det Lanfferska systemet där man talar om

stehzeit eller vad vi med engelska termer kallar stand up time. Detta system finns väl beskrivet i de schweisiska normerna SIA 199.

Vilka är då slutsatserna av studien av ett antal tunnel-projekt som har gått ut på internationell budgivning? Den generella slutsatsen är att man handskats med klassificeringen på ett relativt intelligent sätt. Man strävar efter att prestera klassificeringen som skall fungera som en mätare på vad projekt kommer att kosta. De utgör oftast ett mätningssinstrument för att fastställa ersättning till entreprenörerna. Det kommer att framgå av exemplifieringen nedan att klassificeringarna kan se ganska olika ut. Det är inte att förvåna sig över eftersom bergförhållandena kan vara mycket varierande och likaså skiftar karaktären på uttagsmetoderna. När det gäller TBM-drivning förekommer ofta två klassificeringssystem ett som avser stabilitet och därav följande förstärkningsinsatser samt ett klassificeringssystem som vägleder anbudsgivaren i sin bedömning av framdriftshastigheten.

Det är uppenbart att man strävar efter att ansluta sig till mera välkända klassificeringssystem men i många fall är det inte tillräckligt eftersom systemen inte är tillräckligt känsliga för de platsspecifika förhållandena. Det finns även exempel speciellt vid högbergstäckning då man har givit upp. Det är alltför kostsamt att anständigt klarlägga bergsförhållandena på de aktuella djupen.

Man kan konstatera att klassificeringssystem utarbetade av Geomechanics och NGI används relativt flitigt, åtminstone i den engelskspråkiga världen och där bergsförhållandena är sådana att man kan tillämpa blockmodeller för att beskriva berget.

Det finns även stora undermarksprojekt där man inte gör någon klassificering alls, trots att beställaren är väl så kvalificerad.

För att belysa hur klassificeringen utformas skall här kort återges några exempel på klassificering vid några större projekt som är tämligen färska, inget projekt är äldre än 8 år. Projekten är:

|                                 |           |
|---------------------------------|-----------|
| ► Lesotho highland water scheme | 1989-1994 |
| ► Channel tunnel                | 1988-1991 |
| ► Mauvoisin                     | 1998-     |
| ► Köpenhamn mini metro          | 1996-     |
| ► St Marguerite                 | 1996-     |

några är avslutade och någon är inte ännu påbörjad.

## Köpenhamns mini metro

Köpenhamns mini metro skiljer sig något från de övriga eftersom det har från början deklarerats att huvuddelen av tunnlarna skall utföras med sköldad TBM som kan motstå ett yttre vattentryck. Omedelbart bakom skölden skall man montera betonginklädnad i form av segment.

I samband med projekteringen av metro genomförde man ett omfattande undersökningsprogram i syfte att beskriva berget på ett adekvat sätt. Längs hela tunnellinjen genomfördes en stor mängd undersökningshål både kärnhål och hammarborrhål. I många av hålen har man genomfört geofysisk mätning man har värderat upptagna kärnor med hjälp av NGI som självfallet har tillämpat Q-värderingen. Vidare har borrhärdigheter testats rent mekaniskt och mycket av resultaten ingår i dokumentationen som kan erhållas vid anbudsgivning. Anbudsdokumentationen omfattar dock ej undersökningresultaten.

Vad har man nu gjort med alla data? För de delar som berör den TBM borrarde tunneln har man sökt ge en karakteristik åt berget i form av fördelningsdiagram. Man har alltså inte ägnat sig åt att dela in tunneln i sektioner utmed tunnelns längdaxel och i varje sektion beskriva berget enligt en fastställd standard. Här skall ges några exempel på hur man har beskrivit berget.

Bergets hårdhet har man benämnt induration, som översatt betyder hårdnande, i ett antal steg H1-H5 där H1 är mycket mjukt berg och H5 är karakteriserat av silicifierad kalksten och flinta. Skalan som är dansk har man också relaterat till ISRM:s hårdhetsbeskrivning för spricktytor och som lyder enligt tabell 1 nedan.

|    |                       |                                                                                                                         |          |
|----|-----------------------|-------------------------------------------------------------------------------------------------------------------------|----------|
| R0 | Extremely weak rock   | Indented by thumbnail                                                                                                   | 0.25-1.0 |
| R1 | Very weak rock        | Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife                              | 1.0-5.0  |
| R2 | Weak rock             | Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer | 5.0-25   |
| R3 | Medium strong rock    | Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer   | 25-50    |
| R4 | Strong rock           | Specimen requires more than one blow of geological hammer to fracture it                                                | 50-100   |
| R5 | Very strong rock      | Specimen requires many blows of geological hammer to fracture it                                                        | 100-250  |
| R6 | Extremely strong rock | Specimen can only be chipped with geological hammer                                                                     | >250     |

Tabell 1 Hårdhetsbeskrivning enligt ISRM

I dokumenten som hör till anbudshandlingarna har man gjort en sammanställning över sannolikheten att stöta på berg av olika hårdhetsgrad (H1-H5) och man har därvid skilt på övre och mitre Köpenhamnskalksten tabell 2. Som framgår av värdena så kan man förvänta sig relativt stor spridning i fördelningen. Det är inte

| ISRM<br>Rock<br>Grade | Induration       | Distribution of Induration (%) |        |                            |                                |        |                            |
|-----------------------|------------------|--------------------------------|--------|----------------------------|--------------------------------|--------|----------------------------|
|                       |                  | Upper Copenhagen<br>Limestone  |        |                            | Middle Copenhagen<br>Limestone |        |                            |
|                       |                  | 5% fractile <sup>2)</sup>      | Median | 95% fractile <sup>2)</sup> | 5% fractile <sup>2)</sup>      | Median | 95% fractile <sup>2)</sup> |
| R0                    | H1               | 0                              | 1      | 15                         | 0                              | 1      | 10                         |
| R1                    | H2               | 35                             | 40     | 50                         | 15                             | 34     | 55                         |
| R2                    | H3               | 10                             | 27     | 50                         | 10                             | 28     | 40                         |
| R3-R4                 | H4               | 5                              | 15     | 30                         | 5                              | 20     | 35                         |
| R5-R6                 | H5 <sup>1)</sup> | 5                              | 15     | 20                         | 5                              | 12     | 40                         |

Table 2.3.3 Distribution of induration within the Copenhagen Limestone

uppenbart hur man skall tolka denna uppställning, och därför skall ett exempel göras. Välj H2 i övre Köpenhamnskalksten och uppgifterna tolkas som följer: Sannolikheten för att materialet H2 skall utgöra mindre än 35% är 5% och likaså 5% för att den skall utgöra 50% eller mer och median värdet är 40%.

I den geologiska sammanfattningen som ej utgör del av anbudsdocumenten finns frekvensstudier av bergprovernans enaxliga tryckhållfasthet relaterad till hela population både viktad och oviktad, till övre respektive mitre Köpenhamnskalkster och slutligen fördelat på graden av hårdhet. Dessa diagram kommenteras inte här eftersom de här bara är exempel på hur klassificering kan utföras.

På samma sätt har man beskrivit sprickigheten. Man har utgått från en 5-gradig skala S1-S5 där varje grad är definierad och tabell 3 på nästa sida skall läsas på samma sätt som den föregående tabellen.

| Degree of fissuring | Remarks            | Distribution of fissures (%) |        |                            |                             |        |                            |
|---------------------|--------------------|------------------------------|--------|----------------------------|-----------------------------|--------|----------------------------|
|                     |                    | Upper Copenhagen Limestone   |        |                            | Middle Copenhagen Limestone |        |                            |
|                     |                    | 5% fractile <sup>1)</sup>    | Median | 95% fractile <sup>1)</sup> | 5% fractile <sup>1)</sup>   | Median | 95% fractile <sup>1)</sup> |
| S1                  | Unfractured        | 0                            | 0      | 5                          | 0                           | 0      | 5                          |
| S2                  | Slightly fractured | 15                           | 23     | 70                         | 10                          | 29     | 45                         |
| S3                  | Fractured          | 15                           | 25     | 50                         | 5                           | 28     | 45                         |
| S4                  | Very fractured     | 15                           | 34     | 45                         | 10                          | 32     | 60                         |
| S5                  | Crushed, blocky    | 0                            | 14     | 25                         | 5                           | 8      | 30                         |

Table 2.3.4 Distribution of fissuring within the Copenhagen Limestone

Note: 1) Fractile rounded off the nearest 5%.

För att snabbt få en mera gripbar känsla för hur söndersprucket berget är så har man även studerat fördelningen av RQD och den framgår av tabell 4 nedan.

| Soil Type                   | RQD (%)     |        |              |
|-----------------------------|-------------|--------|--------------|
|                             | 5% fractile | Median | 95% fractile |
| Upper Copenhagen Limestone  | 0           | 24     | 67           |
| Middle Copenhagen Limestone | 0           | 18     | 60           |

Table 2.3.5 Rock Quality Designation of Copenhagen Limestone

Till sifferexercisen har man naturligtvis givit typiska geologiska profiler det är inte vad man i dagligt tal kallar klassificering.

Klassificering av bergförhållandena på detta projekt skiljer sig något från den traditionella. Man frågar sig varför och svaret är sannolikt att söka i alt 1) man har förlagt tunnarna i några få strata 2) drivningsmetoden som påbjudes är relativt okänslig för förändringar i bergkvaliten inom de specificerade ramarna.

### Channel tunnel

Projektet kräver inte någon närmare presentation, tunneln finns väl förankrad i de flestas medvetande. Kontraktet mellan ET(Eurotunnel) och TML (Trans Manche Link) skrivs under i augusti 1986. Kontraktet säger att TML skall designa, bygga, testa, genomföra kommission och ET skall driva transportleden. Tunneldesign och byggande var baserade på en target kostnadsuppgörelse.

Undersökningar för tunneln hade inletts redan under 1950-talet och utförts i tre omgångar 1958-1965, 1972-1973 samt 1986-1987 där den sista kampanjen utfördes av TML. Man kunde då konstatera att den geologiska strukturen längs tunneln är relativt enkel och det föreligger inga stora förskjutningar. Den största förkastningen låg på franska sidan och hade en vertikal rörelse av 8-10 meter.

Redan efter borrhkampanjen 1973-1974 hade man konstaterat att lagret med chalk marl var det mest lämpade. Vid borrhkampanjen 1986-1987 fördjupade man sitt kunnande om detta strata i vilket tunneln skulle förläggas.

Någon egentlig klassificering av berget längs tunnelsträckningen gjorde man alltså inte. Bergförhållandena var tämligen ensartade med undantag för förkastningarna på den franska sidan. Man konstaterade efter att ha utfört ett antal klassificeringar på blottat berg vid Sheakspeare-Cliff såväl under som ovan jord att RMR och Q-systemet var relativt okänsliga för de variationer som förelåg i Strata Chalk marl man erhöll spannet 58-68 för RMR och 5-14 för Q. Trots det så tillämpade man en klassificering på de delar som schaktades ut med radheader och där berget exponerades. Den klassificeringen utgjorde mer en dokumentation än underlag för beslut och avtal. Huvuddelen av tunnarna skulle utföras med sköldad TBM och man hade dåliga möjligheter att kontrollera huruvida en bergsklassificering stämde eller ej. Man koncentrerade sig på att så långt det var meningsfullt beskriva den aktuella geologiska horisonten. Här skall kommenteras vilka faktorer som var styrande för förläggningen av tunneln. Man koncentrerade sig på RQD-värden, permeabilitet kalkhalt, svällegenskaper, odränerad skjuvhållfasthet och Youngs modulus. Man sökte ett optimum för dessa.

Publicerade värden från undersökningarna visar måttlig spridning hos värdena man har alltså lyckats lokalisera tämligen ensartade horisonter där det inte är lönt att söka särskilja olika partier med traditionella klassificeringsmetoder.

Den uppföljning man kunde genomföra i samband med tunneldrivning var att lokalisera ett tunt med väldefinierat lager och styra TBM i vertikalled så att man befann sig inom ett förutbestämt avstånd från detta.

### ***Lesotho transfer tunnel***

Lesotho highland water project tillhör sannolikt de större enskilda undermarksprojektet som har erbjudits de internationella entreprenörerna under senare tid. Projektet omfattar hela 80 km tunnel med en diameter på 5 meter. Tunneln är förlagd i två olika formationer nämligen basalt och sandsten. Tunneln kom att drivas med hjälp av tunnelborrmaskin. Inte mindre än 5 TBM engagerades för projektet varav 4 var helt öppna maskiner och en dubbelsköldad TBM som hade kapacitet att sätta betongelement under skölden. Det fanns alltså ett klart intresse från beställaren att med någon form av klassificering beskriva och kvantifiera bergförhållandena, för att få ett instrument att reglera kontraktssumman med. Det var möjligt att bjuda både med TBM och borra spräng. Man skapade två klassificeringssystem nämligen klassificering av berg och klassificering av förstärkning. Att endast tunneldel i basalt behandlas här beror på att projektet var uppdelat i två designkontrakt och endast uppströmsdelen, den i basalt, kommer att presenteras.

Basalten ligger i huvudsak i horisontella lager med varierande tjocklek beroende av hur kraftigt lavaflödet och vilka viskositetsförhållandena var. Basaltens karaktär varierar således och det är den variationen som återspeglas i klassificeringssystemet som återges nedan.

- Bergklass A      Lesotho basalt som innehåller mer än 1 ytprocent av amygduler synbara för ögat. Till A räknas även tufflager.
- B      Lesotho basalt med mindre än 1 ytprocent av amygduler synbara för ögat.
- C      Intrusiva bergarter i huvudsak dominerade av doleriter men även andra intrusiver ingår i denna klass.

Men eftersom man längs hela tunneln kommer att möta s k mixed face d v s A och B och eventuellt C, så har man på basis av den första klassificeringen gjort en tunnelanpassad bergsklassificering som ser ut som nedan.

- i       $A > 70\%$   
          $B + C < 30\%$
- ii       $70\% > A > 30\%$   
          $70\% > B + C > 30\%$   
          $C < 30\%$
- iii       $A < 30\%$   
          $B + C > 70\%$   
          $C < 30\%$
- iv       $A + B < 70\%$   
          $C > 30\%$

Varför har man nu gjort en bergbeskrivning enligt detta mönster? Syftet med klassificering är att den skall återspegla de insatser som entreprenören måste vidtaga i form av variationer i insatser men även återspegla uppnåbara kapaciteter. I det här fallet är det uppenbart att man erhåller högre TBM-indrift i A-berg och sämst i C-berg. Likaså kan det vara en variation i cutterslitaget. Beställaren har i detta fall inte företagit någon klassificering av berget över definierade sträckor längs tunneln. Man har istället sammanställt totalmängderna av de olika bergklasserna för de skilda byggnadsdelarna. Exempel på uppdelning framgår av tabell 5 på nästa sida som täcker  $\frac{1}{2}$  transfertunneln (tunneln i basalt) eller 23 km.

RATES OF EXCAVATION ADVANCE IN MIXED FACE CONDITIONS (m/week)  
(TBM Option)

|                                                 | Percentage of Rock Classes In Face |                                          |                                |                      | TOTAL LENGTH OF CLASSIFIED EXCAVATION |
|-------------------------------------------------|------------------------------------|------------------------------------------|--------------------------------|----------------------|---------------------------------------|
|                                                 | A > 70<br>B + C < 30               | 70 > A > 30<br>30 < B + C < 70<br>C < 30 | A < 30<br>B + C > 70<br>C < 30 | A + B < 70<br>C > 30 |                                       |
| Intake Adit (DBM)<br>Rate of advance =          | ( 65 )                             | ( 69 )                                   | ( 78 )                         | ( 4 )                | ( 216 )                               |
| Pelaneng Adit (DBM)<br>Rate of advance =        | ( 566 )                            | ( 604 )                                  | ( 679 )                        | ( 38 )               | ( 1 887 )                             |
| Intake North Drive (TBM)<br>Rate of advance =   | ( 3 410 )                          | ( 3 637 )                                | ( 4 091 )                      | ( 227 )              | ( 11 365 )                            |
| Pelaneng North Drive (TBM)<br>Rate of advance = | ( 2 780 )                          | ( 2 965 )                                | ( 3 335 )                      | ( 185 )              | ( 9 265 )                             |
| TOTALS                                          | ( 6 821 )                          | ( 7 275 )                                | ( 8 183 )                      | ( 454 )              | ( 22 733 )                            |

1. Rock Class A indicates amygdaloidal basalt.
2. Rock Class B indicates non-amygdaloidal basalt.
3. Rock Class C indicates Intrusive rocks.

Tabellen är avsedd för TBM-drivning som utgjorde en möjlig drivningsmetod. För borrhning och sprängning är bergförhållandena detsamma. Tabellen är avsedd för anbudsgivaren att gå in med sina långtidskapaciteter. Genom att fylla i tabellen så förbinder sig entreprenören att prestera ett antal meter per vecka. Fastställelse av rock class görs gemensamt för var 20 meter i samband med utförande.

Man har också en sk supportklassificering och syftet med denna är också att utgöra underlag för ersättning till entreprenör. Klassificering framgår av tabell 6 nedan.

TABLE 11/1 SUPPORT CLASSIFICATION

| Support Class |      | Level of Initial Support                                                       |
|---------------|------|--------------------------------------------------------------------------------|
| TBM           | DBM  |                                                                                |
| BM I          | BI   | Random rockbolting                                                             |
| BM II         | BII  | Systematic light pattern bolting                                               |
| BM III        | BIII | Systematic pattern bolting with mesh over crown                                |
| BM IV         | BIV  | Systematic pattern bolting with reinforced shotcrete over crown                |
| BM V          | BV   | Steel arches with reinforced shotcrete                                         |
| BM VI         | BVI  | Systematic pattern bolting with short intermediate bolts in overstressed zones |

## 11.8.4 Determination of the Rock Classes and Mixed Face Conditions

Tabellen är tillgänglig på både TBM-drivning och konventionell drivning, och avser endast förstärkning som måste utföras vid tunnelfront. Det är sådan förstärkning som ibland benämns temporär och som inverkar på framdriftshastigheten. Anbudsgivaren får ange i vilken utsträckning som de skilda supportklasserna reducerar framdriftshastigheten. Det gjordes i prognosen över supportklasserna som framgår av tabell 7 nedan.

TENDER SCHEDULE L      CONTRACT NO. LHDA 124  
SCHEDULE L(V)

EFFECT OF INITIAL SUPPORT INSTALLATION IN ALL MIXED FACE CONDITIONS (m/week)  
(TBM Option)

| Support Class                 |                                                            |              |                |              |            |         |
|-------------------------------|------------------------------------------------------------|--------------|----------------|--------------|------------|---------|
| TBM<br>DBM                    | BM I<br>BI                                                 | BM II<br>BII | BM III<br>BIII | BM IV<br>BIV | BM V<br>BV | TOTALS  |
| make Adt<br>(DBM)             | (185)                                                      | (14)         | (7)            | (0)          | (0)        | (216)   |
|                               | Reduction in rate of advance due to support installation = |              |                |              |            |         |
| Pelaneng Adt<br>(DBM)         | (1793)                                                     | (52)         | (42)           | (0)          | (0)        | (1887)  |
|                               | Reduction in rate of advance due to support installation = |              |                |              |            |         |
| Intake North<br>Drive (TBM)   | (10157)                                                    | (593)        | (393)          | (167)        | (55)       | (11365) |
|                               | Reduction in rate of advance due to support installation = |              |                |              |            |         |
| Pelaneng North<br>Drive (TBM) | (7 892)                                                    | (936)        | (346)          | (68)         | (23)       | (9265)  |
|                               | Reduction in rate of advance due to support installation = |              |                |              |            |         |
| TOTALS                        | (20 038)                                                   | (1595)       | (788)          | (234)        | (78)       | (22733) |

Tabellen ovan täcker supportklasser i-V. Klass Vi som är allokerad för smällberg har givits en särskild tabell eftersom man räknar med att korta bultar ej kommer att tillämpas på hela tvärsnittet.

EFFECT OF INITIAL SUPPORT CLASS VI INSTALLATION IN ALL MIXED FACE CONDITIONS (m/week)  
(TBM Option)

## Pelaneng North Drive

| No. of<br>1,20m Bolts/m | 8     | 16    | 24    | 32    | TOTALS  |
|-------------------------|-------|-------|-------|-------|---------|
| BMI &<br>BMVI           | (190) | (585) | (175) | (145) | (1 095) |
| BMI &<br>BMVI           | (10)  | (20)  | (10)  | (10)  | (50)    |
| BMI &<br>BMVI           | (10)  | (10)  | (10)  | (10)  | (40)    |
| TOTALS<br>Reduction =   | (210) | (615) | (195) | (165) | (1 185) |

Man kan konstatera att man på detta projekt har gått inför en mycket pragmatisk klassificering där entreprenören skall få ersättning för skilda bergförhållande helt beroende av hur han anser att de påverkar honom. Man har vidare undvikit att lägesbestämma de skilda bergtyperna sannolikt för man har inte kunnat pågå den bitvis höga bergtäckningen. Merparten av tunneln har mer än 500 meter berg över sig.

### **Mauvoisin II**

Mauvoisin II är ett vattenkraftprojekt beläget i södra delen av Schweiz. Projektet var ute för anbudsräkning under 1995 men har skjutits på framtiden några år. Det betyder dock inget i detta sammanhang då bergsklassificeringen i förundersökningen ej är beroende av detta.

Tunneln var från början avsedda att utföras med hjälp av TBM och därför har både undersökningar och klassificeringssystem anpassats för detta. Man har här använt två parallella löpande klassificeringssystem. Man har det schweiziska traditionella systemet där man har sk "unsbruchklassen" (brytningsklasser) som är numrerade från I-T. Dessa klasser har funnits ett 20-tal år och innebär på intet sätt någon nyhet. Trots detta visas tabell 9 på nästa sida.

I denna tabell som är speciellt anpassad för TBM har man en klassangivelse och därtill hörande specificering av erforderlig förstärkning. Man har också angivit var förstärkningarna skall sättas in i förhållande till TBM:n, L1 är i maskinområdet, L2 vid bakriggen och L3 bakom bakriggen. Normalt medföljer en längdsektion över hela tunneln i vilken man anger i vilka sektioner man kan förvänta sig de olika brytningsklasserna.

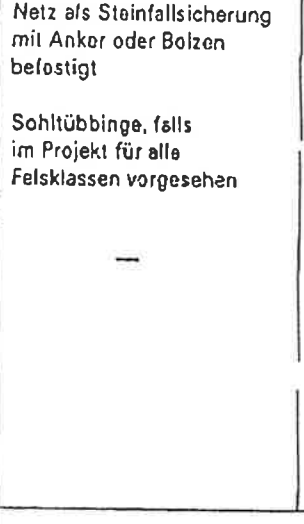
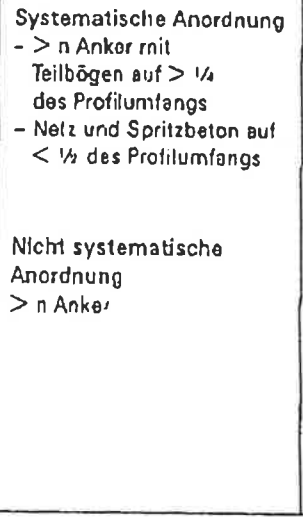
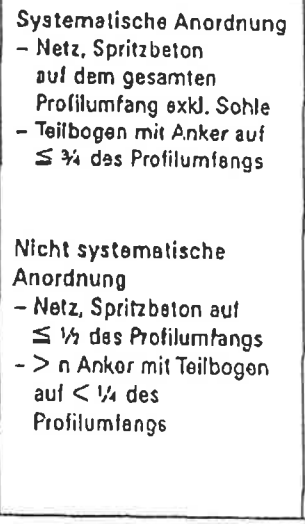
|                                                                                                                                                                                                                                                 | Klasse I                                                                                                                                                                                                                    | Klasse II                                                                                                                                                                                                                                                                                                                             | Klasse III                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | Klasse IV                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             | Klasse V                                                                                                                              | Klasse T                                                                            |                          |        |        |        |         |   |   |   |   |   |
|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|--------------------------|--------|--------|--------|---------|---|---|---|---|---|
| Ausbruchklassen für Tunnel                                                                                                                                                                                                                      |                                                                                                                                                                                                                             |                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                       |                                                                                     |                          |        |        |        |         |   |   |   |   |   |
| Maschinenbereich L1                                                                                                                                                                                                                             |                                                                                                                                            | $\leq n$ Anker am Profilumfang                                                                                                                                                                                                                                                                                                        | $> n$ Anker am Profilumfang mit Netz und/oder einzelnen Verzugsblechen                                                                                                                                                                                                                                                                                                                                                                                                                            | <ul style="list-style-type: none"><li>- Anker mit Netz und Teilbogen</li><li>- Spritzbeton als Versiegelung auf <math>&gt; \frac{1}{4}</math> des Profilumfangs</li><li>- Druckverteilende Elemente im Bereich der Verspannplatten</li></ul>                                                                                                                                                                                                                                                                          | <ul style="list-style-type: none"><li>- geschlossener Stahleinbau mit oder ohne Verzug</li><li>- Vollflächige Stahlelemente</li></ul> | geschlossener Tübbing-einbau                                                        |                          |        |        |        |         |   |   |   |   |   |
| Nachläuferbereich L2                                                                                                                                                                                                                            | <p>Netz als Steinfallsicherung mit Anker oder Bolzen befestigt</p> <p>Sohltübbinge, falls im Projekt für alle Felsklassen vorgesehen</p>  | <p>Systematische Anordnung</p> <ul style="list-style-type: none"><li>- <math>&gt; n</math> Anker mit Teilbögen auf <math>&gt; \frac{1}{4}</math> des Profilumfangs</li><li>- Netz und Spritzbeton auf <math>&lt; \frac{1}{2}</math> des Profilumfangs</li></ul> <p>Nicht systematische Anordnung</p> <p><math>&gt; n</math> Anker</p> | <p>Systematische Anordnung</p> <ul style="list-style-type: none"><li>- Netz, Spritzbeton auf dem gesamten Profilumfang exkl. Sohle</li><li>- Teilbogen mit Anker auf <math>\leq \frac{3}{4}</math> des Profilumfangs</li></ul> <p>Nicht systematische Anordnung</p> <ul style="list-style-type: none"><li>- Netz, Spritzbeton auf <math>\leq \frac{1}{2}</math> des Profilumfangs</li><li>- <math>&gt; n</math> Anker mit Teilbogen auf <math>&lt; \frac{1}{4}</math> des Profilumfangs</li></ul> | <p>Systematische Anordnung</p> <ul style="list-style-type: none"><li>- Sohlensicherung mit Spritzbeton, falls keine Sohlübbinge vorgesehen</li><li>- geschlossener Stahleinbau, ev. auf Sohlübbbing abgestützt</li></ul> <p>Nicht systematisch Anordnung</p> <ul style="list-style-type: none"><li>- Netz, Spritzbeton auf dem gesamten Profilumfang exkl. Sohle</li><li>- Teilbogen mit Anker auf <math>\leq \frac{3}{4}</math> des Profilumfangs</li><li>- geschlossener Stahleinbau mit oder ohne Verzug</li></ul> |                                                     |  |                          |        |        |        |         |   |   |   |   |   |
| Rückwärtiger Bereich L3 bis 200 m hinter Nachläufer                                                                                                                                                                                             | <p>Die Ausbruchsicherung im L3 ist für die Ausbruchklasseneinteilung unerheblich. Falls eine solche vorgesehen wird, sind Stillstandspositionen vorzusehen.</p>                                                             |                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                       |                                                                                     |                          |        |        |        |         |   |   |   |   |   |
| <p>Werte für n</p> <table><tr><td>max. Ausbruchdurchmesser</td><td>4.00 m</td><td>6.00 m</td><td>9.00 m</td><td>12.00 m</td></tr><tr><td>n</td><td>2</td><td>3</td><td>4</td><td>6</td></tr></table> <p>(Anzahl Anker pro Lautmeter Tunnel)</p> |                                                                                                                                                                                                                             |                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                                                                                                                                       |                                                                                     | max. Ausbruchdurchmesser | 4.00 m | 6.00 m | 9.00 m | 12.00 m | n | 2 | 3 | 4 | 6 |
| max. Ausbruchdurchmesser                                                                                                                                                                                                                        | 4.00 m                                                                                                                                                                                                                      | 6.00 m                                                                                                                                                                                                                                                                                                                                | 9.00 m                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            | 12.00 m                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                                                                                                                                       |                                                                                     |                          |        |        |        |         |   |   |   |   |   |
| n                                                                                                                                                                                                                                               | 2                                                                                                                                                                                                                           | 3                                                                                                                                                                                                                                                                                                                                     | 4                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                 | 6                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                       |                                                                                     |                          |        |        |        |         |   |   |   |   |   |

Tabelle 12 Bestimmung der Ausbruchklassen aufgrund der Ausbruchsicherung für Vortrieb mit Tunnelbohrmaschine im Fels <sup>1)</sup>

<sup>1)</sup> Sind in einem Tabellenfeld mehrere Sicherungsmassnahmen aufgeführt, so genügt jede Massnahme für sich allein für die Klassierung.

Det andra klassificeringssystemet som här tillämpas parallellt skall ge entreprenören en uppfattning om hur snabbt han kan ta sig fram och vilken förslitning han kan förvänta sig på diskrullar och borrhäklar. I den tar man upp faktorerna kvartshalt, hårdhet avstånd mellan diskontinuiteter och motstånd mot uppsprickning i diskontinuiteterna. Genom att ta hänsyn till så många faktorer har man fått ett system med inte mindre än 36 klasser och systemet framgår av tabell 10 på nästa sida.

Vid klassificering av liknande arbeten där man inte har tänkt sig TBM tar man inte med den senaste klassificeringen då den är nästan direkt riktad till mekanisk bearbetning. Däremot har man den tidigare klassificeringen men där då L1=Brust bereich, L2=Vortriebs bereich och L3=Rückwärtiger bereich.

Som framgår av detta exempel har man en mycket praktisk syn på klassificeringen. Det skall finnas ett klart samband mellan bergklass och kostnad för utförande.

TABELL 10

MAUVOISIN II - LOT 1 - Conditions particulières

Annexe 7.2

GALERIE D'AMENEE - CLASSES D'APTITUDE  
AU FORAGE - DEFINITIONS

| QUARTZ % | DURETE DU<br>ROCHER<br>INTACT | SCHISTOSITE/<br>LITAGE | RESISTANCE AU<br>CISAILLEMENT SELON<br>LA SCHISTOSITE | CLASSE<br>D'APTITUDE<br>AU FORAGE |
|----------|-------------------------------|------------------------|-------------------------------------------------------|-----------------------------------|
| < 20 %   | FAIBLE                        | SERREE                 | FAIBLE                                                | 1                                 |
|          |                               |                        | ELEVEE                                                | 2                                 |
|          |                               | ESPACEE                | FAIBLE                                                | 3                                 |
|          |                               |                        | ELEVEE                                                | 4                                 |
|          | MOYENNE                       | SERREE                 | FAIBLE                                                | 5                                 |
|          |                               |                        | ELEVEE                                                | 6                                 |
|          |                               | ESPACEE                | FAIBLE                                                | 7                                 |
|          |                               |                        | ELEVEE                                                | 8                                 |
|          | FORTE                         | SERREE                 | FAIBLE                                                | 9                                 |
|          |                               |                        | ELEVEE                                                | 10                                |
|          |                               | ESPACEE                | FAIBLE                                                | 11                                |
|          |                               |                        | ELEVEE                                                | 12                                |
| 20%-40%  | FAIBLE                        | SERREE                 | FAIBLE                                                | 13                                |
|          |                               |                        | ELEVEE                                                | 14                                |
|          |                               | ESPACEE                | FAIBLE                                                | 15                                |
|          |                               |                        | ELEVEE                                                | 16                                |
|          | MOYENNE                       | SERREE                 | FAIBLE                                                | 17                                |
|          |                               |                        | ELEVEE                                                | 18                                |
|          |                               | ESPACEE                | FAIBLE                                                | 19                                |
|          |                               |                        | ELEVEE                                                | 20                                |
|          | FORTE                         | SERREE                 | FAIBLE                                                | 21                                |
|          |                               |                        | ELEVEE                                                | 22                                |
|          |                               | ESPACEE                | FAIBLE                                                | 23                                |
|          |                               |                        | ELEVEE                                                | 24                                |
| >40%     | FAIBLE                        | SERREE                 | FAIBLE                                                | 25                                |
|          |                               |                        | ELEVEE                                                | 26                                |
|          |                               | ESPACEE                | FAIBLE                                                | 27                                |
|          |                               |                        | ELEVEE                                                | 28                                |
|          | MOYENNE                       | SERREE                 | FAIBLE                                                | 29                                |
|          |                               |                        | ELEVEE                                                | 30                                |
|          |                               | ESPACEE                | FAIBLE                                                | 31                                |
|          |                               |                        | ELEVEE                                                | 32                                |
|          | FORTE                         | SERREE                 | FAIBLE                                                | 33                                |
|          |                               |                        | ELEVEE                                                | 34                                |
|          |                               | ESPACEE                | FAIBLE                                                | 35                                |
|          |                               |                        | ELEVEE                                                | 36                                |

### ***St Marguerite III***

Vid vattenkraftsbygget St Marguerite i Quebec Kanada, som just nu är inne i anbudsfasen, har man bedrivit förundersökningar i nästan 10 år. Det har resulterat i en stor mängd dokument. Beställaren Hydro Quebec har på ett snyggt sätt redovisat alla undersökningsresultaten. De utgör dock inte del av anbudsdocumenten men finns tillgängliga för studier på ett kontor i Montreal. Man får emellertid inte göra några kopior av dem.

Varken i anbudsdocumenten eller i de utställda handlingarna har man gjort någon egentlig sammanställning av resultaten och än mindre har man utfört någon klassificering av berget. Man kan naturligtvis fråga sig om detta är en internationell entreprenad eftersom anbudsgivarna är tvungna att ha ett operativt kontor i Quebec och likaså har man styckat upp det stora arbetet för att ge de lokala entreprenörerna en fördel. Likafullt har de möjlighet att slå sig samman med större internationella entreprenörer.

Hur ser då projektet ut? Det innehåller en damm, en tilloppstunnel 8 km lång 170 m<sup>2</sup> stor, svallschakt, men inte pennstock, en kraftstation under jord och en avloppstunnel av samma storlek som tilloppstunnel och cirka 2 km lång. Bergtäckningen är måttlig 50-300 meter. Berget utgörs av en anorthosite, som består till cirka 90% av fältspat och är mycket kompetent. Förundersökningarna har koncentrerats till dammläget och kraftstationen. Den stora och långa tilloppstunneln har bara tilldelats två kärnborrhål ett vid svallschaktet och ett genom en större förväntad förkastning. Förutom borrhåll har man utfört satellitbildstolkningar, ytkartering samt seismiska undersökningar. Resultatet av dessa visar på ett näst intill otroligt gott berg om man bortser från dammläget. Kärnåtervinningen är generellt 100% när man har kommit igenom ytberget och RQD-värdena är generellt 90% eller mer. Det lutande kärnhålet genom den förväntade förkastningen visade på en sänkning av RQD-värdet över cirka 40 meter till minimala 45%.

Beställaren, som måste betraktas som mycket erfaren, har därför inte besvärat sig med att göra någon klassificering. Han har istället mycket strikt fastställt villkoren för tunneldrivningen och har till och med givit borrhållsplan för uttag av tunnelpallen. I mängdförteckningen återspeglas hans tro på erforderliga förstärkningar. På 5 km av tilloppstunneln som är entreprenad förväntar han sig endast cirka 7 000 bult och 250 m<sup>3</sup> sprutbetong däremot tror han sig behöva 20 000 m<sup>2</sup> nät som inte skall insprutas med betong.

Att han förväntar sig mycket små problem med stabiliteten återspeglar sig också i tidsplanen där galleriet skall tas ut med cirka 180 m/månad och pallen 300 m/månad sett som enkelfront.

## **Slutord**

Ambitionen har varit att återge skilda sätt att klassificera berg vid större internationella entreprenader. Ambitionen har inte varit att statistiskt försöka klarlägga dominans för ett eller annat system utan mera visa vad som kan förekomma. Det är uppenbart så att det föreligger en klart hög frekvens av användandet av RMR och Q-systemet även om de inte har exemplifierats här. De har emellertid sina begränsningar speciellt vid TBM-tunnlar varför beställare i många fall har tagit fram lokalt anpassade system för att i slutändan uppnå ett billigare och bättre resultat.

**Källor**     Anbudshandlingar  
                 Köpenhamns mini metro  
                 Lesotho highland project  
                 Mauvoisin II  
                 St Marguerite III

## **SUMMARY**

This paper describes different classification systems used in international contracts. Examples are taken from five planned or completed projects, namely the Copenhagen mini metro (Denmark), the Channel railway tunnel (France/ UK), the Lesotho transfer tunnel (South Africa), the Mauvoisin II hydropower project (Switzerland) and the St Marguerite III hydropower project (Canada).



# **"STÖRDA ZONEN" VID BERGBYGGGANDE**

## **"Excavation disturbed Zone" in Rock Engineering**

Introduktion: Christer Svemar, SKB



## JÄMFÖRANDE STUDIE AV DEN STÖRDA ZONEN FÖR SPRÄNGD RESPEKTIVE BORRAD TUNNEL - ZEDEX-FÖRSÖKET I ÄSPÖ

A comparison of the excavation disturbed zone for blasted and bored tunnels -  
Results from the ZEDEX Project at Äspö

Olle Olsson, Svensk Kärnbränslehantering AB

### INTRODUCTION

Previous work has shown that the excavation of a tunnel will cause a disturbance to the rock mass surrounding the tunnel. The character and magnitude of the disturbance is a function of the existence of the air filled void represented by the tunnel, the resulting stress redistribution, the method of excavation used to excavate the tunnel and the properties and nature of the rock mass. The properties of the disturbed zone surrounding excavations is of importance to repository performance and safety assessment as it may provide preferential pathways for radionuclide transport or may affect the efficiency and effectiveness of plugs placed to seal drifts.

ANDRA, UK Nirex, and SKB have jointly set up the ZEDEX Project at the Äspö Hard Rock Laboratory, Sweden, in order to improve understanding of the properties and extent the Excavation Disturbed Zone (EDZ).<sup>1</sup> The main objective was to study the mechanical behavior of the EDZ with respect to its origin, character, magnitude of property change, extent, and its dependance of excavation method. The project also provided a test of equipment and methodology for quantifying the EDZ. The ZEDEX Project includes tests of "normal" excavation by drill and blast, a smooth blasting procedure based on low shock explosives, and excavation by tunnel boring. The results are expected to build confidence in the modelling of the disturbed zone and to support the selection of excavation methods for repository construction.

### EXPERIMENTAL CONFIGURATION

The ZEDEX experiment is performed in two parallel drifts at a depth of 420 m below the ground surface. One drift has been excavated by tunnel boring and the other drift by drill and blast. The shape of the blasted drift is circular, except for the flat floor. The diameter is the same as for the TBM drift, i.e. 5 m. A number of boreholes have been drilled axially and radially relative to the test drifts to assess the properties and extent of the EDZ (Figures 1 and 2).

The excavation disturbance has been quantified through measurements of:

- acceleration
- displacements
- P- and S-wave velocity
- hydraulic conductivity
- natural and induced fracturing
- temperature
- acoustic energy release (micro-crack generation)
- stress state

Remote sensing techniques, such as cross-hole seismic tomography and reflection radar applied from drifts and boreholes, were used to map the spatial change in properties. Measurements were made before and after excavation in order to quantify changes in properties due to excavation.

The drilling and instrumentation of boreholes along the TBM test drift began in April 1994. The excavation of the TBM drift and measurements in the holes along this drift after excavation were completed in October 1994. Tests in the Drill & Blast test drift began in November 1994 and were completed by April 1995. This was followed by an integrated analysis results obtained from both drifts.

## RESULTS

The rock at the ZEDEX site is predominantly Äspö diorite intersected by a few dikes of fine-grained granite. The rock mass is intersected by three dominating sets of fractures, some of which are water bearing with considerable outflow. The D&B drift is only intersected by two water-conducting fractures while the number of water-conducting fractures intersecting the TBM drift is considerably larger. Borehole radar, seismic reflection, and geotechnical mapping were all useful in delineating the features. The main discontinuities are all steeply dipping striking NW and NE (Figure 3).

Q and RMR logging used to determine the rock mass quality gave uniform results of good quality rock, indicating the requirement for no support over the tunnel span.

Seismic tomographic imaging showed variability in velocity and attenuation interpreted to be associated with fracturing. One or two features in the tomograms were seen to correlate with the mapped fractures in the TBM drift. However, identifying smaller features from tomographic imaging alone is generally more problematic. The average P-wave velocity of about 6 km/s indicated very good quality rock. Seismic anisotropy measurements showed good correlation with fracture orientations.

The magnitude of the main principal stress ( $\sigma_1$ ) is estimated to approximately 32 MPa at the ZEDEX site. The direction of  $\sigma_1$  is approximately NW and horizontal. The magnitudes of  $\sigma_2$  and  $\sigma_3$  are estimated to 17 and 10 MPa, respectively.

During TBM excavation, vibration measurements showed that only about 0.03 % of the energy used to excavate the tunnel and bore through the rock was radiated into the surrounding rock as seismic energy. Particle velocities determined 3 m from the tunnel wall during cutting were only about 1 mm/s. For the D&B drift excavated by blasting 4-7 % of the energy applied in the form of explosives was converted into seismic energy. Accelerations measured 3 m from the drift wall reached values in excess of 500 g. The TBM requires much more cumulative energy to create a similar length of tunnel compared to a drill and blast round. However, the time required for excavation and the minimal seismic efficiency of the TBM means that drill and blast methods may input somewhere in the range of a million times more power into the rock as seismic waves or ground vibration.

Acoustic Emission (AE) monitoring was made when TBM excavation was stopped overnight at 9 m, 15 m, 22 m, and 25 m measured from the start of the TBM tunnel. When the TBM stopped at 9.0 meters the majority of the recorded and subsequently located AE activity (232 events) defined a narrow zone directly in front of the position of the TBM face. Additionally other events were located around the tunnel, generally within 1 meter of the tunnel perimeter and ahead of the tunnel face. In the D&B drift AE monitoring was made after each blast round (Figure 4). The spatial distribution of AE events was similar for both drifts but the number of events recorded for the D&B drift excavation was about 10 times more than for the TBM drift. For rounds where blasting failed most AE events were located within the blasted yet intact rock, but there was some evidence that the damage also extended further into the walls than for successful blasts. Estimates of crack initiation stress based on the maximum shear stress ( $\sigma_1 - \sigma_3$ ) at the AE event locations gave an average value of about 25 MPa. This low stress value suggests that the cracking may have been occurring in rock already damaged directly by the excavation process.

Convergence measurements were made at TBM-tunnel lengths of 9 m and 24 m. The results indicate predominantly horizontal convergence (3.6 mm and 1.3 mm, respectively) in both sections, with little displacement in the vertical direction. This pattern of displacements is consistent with the *in situ* stress measurements which indicate a horizontal to vertical stress ratio of about 3:1. The magnitude of the displacements is also consistent with expected magnitudes of the mass modulus.

In the D&B drift displacements were measured after each round. Measured displacements were generally less than 1 mm where most of the displacement occurred within one blast round. The small displacements measured were partly due to that the convergence pins had to be placed about 2 m behind the center of the face due to its curvature.

The seismic tomography results showed no effects of excavation on the seismic properties of the rock at distances larger than 1.5 m from the drift wall for any of the excavation methods. However, in the D&B drift a low velocity zone extending up to about 1 m from the drift floor was observed.

Hydraulic pressure build-up tests in C4 and C5 at the TBM drift showed a general decrease in permeability after the excavation. Hydraulic tests in radial boreholes in the TBM drift did not show any notable induced effect by excavation on matrix permeability. At the D&B drift hydraulic properties in the A- and B-boreholes generally show increased permeability after excavation even though both large increases and decreases are observed locally. Hydraulic test in the short radial holes in the D&B drift show clear evidence of increased permeability in a zone near the drift wall.

The results of the measurements made in the short radial boreholes (permeability, seismic velocity, acoustic impedance, and fracture mapping) can be summarized as follows:

- The comparison and analysis between the different measurements types in general showed a strong correlation.
- The vertical boreholes in the D&B drift indicate the presence of a larger zone of damage than observed in boreholes with other orientations. This can be explained by the high lifter hole charging and the large radius of the invert (flat floor).
- The results of measurements made in the TBM drift and horizontal boreholes in the D&B drift are interpreted to show less damage.
- The extent of the damaged zone in the D&B drift is estimated to be 0.8 m in the floor and about 0.3 m in the walls. In the TBM drift there is very little evidence of damage except within 0.2 m of the tunnel perimeter.

During excavation of the D&B drift, there were two major misfires which had to be reblasted in total. Four rounds had to be reblasted as rock was remaining in the contour of the walls. Only 4 of the 9 rounds did not require any corrective action. The reasons suggested for the low success rate in blasting were misfires due to sympathetic initiation of the charges in the cut, however, successful blasts sometimes had some missing detonations in the cut. Generally, failed rounds showed more intensive and extensive damage than successful rounds.

Differences in initial conditions, coupled with relatively small differences between results from the two drill and blast designs, make it difficult to draw any conclusions

regarding the differences between the effect of excavation method between low shock energy smooth blasting and normal smooth blasting.

## CONCLUSIONS

Generally, the far-field measurements showed little evidence of damage for any of the excavation techniques. MPBX displacement measurements showed that deformation in the far-field was predominantly elastic around the normal-smooth-blasted excavation, despite this being the technique that put the highest energy into the rock. Seismic measurements showed no measurable change in the far-field. AE source locations showed that there was sparse activity beyond about 2 m from the tunnel perimeter, but the distribution of events was similar for both TBM and D&B drifts at that distance. Hydraulic tests of flow rate, specific capacity and transmissivity showed that there were changes occurring in the far field around both tunnels, but the results were inconsistent and difficult to interpret. Hydraulic changes and AE activity in the far-field may result from opening, closing or minor shearing of existing fractures due to stress redistribution.

From the measurements performed it is possible to suggest that the damaged zone formed by the excavation of the D&B drift was greater than that caused by excavating the TBM drift. This damaged zone is characterized by induced fracturing, increased permeability, and reduced seismic velocities. It must, however, be borne in mind that relatively few measurements were made around the TBM drift to determine the extent of the damaged zone. The maximum measurable damage zone was approximately 20 cm around the TBM drift and approximately 80 cm around the D&B drift. However, from the measurements made it was observed that the extent of the damage zone varied around the D&B drift but the maximum 'damage' or disturbance was in the floor and probably was a consequence of the larger charges used in the lifter holes for the rounds and the flat shape of the floor.

The range of methodologies employed during the ZEDEX project has allowed equipment to be both tested and to assess the applicability of certain equipment and methods to quantifying the EDZ and the change in properties in the EDZ around the drifts. Some of the equipment utilized was specially designed and constructed to address the problem of measuring the EDZ and the funding organizations anticipated that they would use the equipment developed within their own programmes. An example of this is the seismic tomography equipment, the use of the equipment as part of the ZEDEX project has indicated where improvements would be required. Additionally, the use of the equipment suggests that if the development of an EDZ is to be detected/investigated using this technique one, or both, of the boreholes to be used must be closer to the designed position of the drift wall than at Äspö. Furthermore, from the work performed some of the techniques used should not be selected for future EDZ experiments.

The laboratory measurements provided a check on the near field in-situ measurements in terms of providing data on rock properties by alternative techniques. In general, the laboratory measurements corroborated the results obtained in-situ and provided greater confidence in the interpreted extent of the damage zone. In addition, the laboratory measurements provided data on parameters such as micro-crack porosity, compressibility and micro-cracking not directly obtainable from in-situ methods.

In general, the suite of methods employed in the ZEDEX Project has been effective in characterizing the extent and magnitude of property change in the EDZ. However, a major problem in evaluating the results which have been obtained in a heterogeneous geologic setting is the relative scarcity of data points. To obtain a better statistical base to substantiate conclusions made above, the ZEDEX Project will be extended by another year. The project extension will include drilling and measurements in additional short radial holes, modelling and extended data evaluations focussed at understanding of the crack initiation mechanisms and observed changes in hydraulic properties.

#### ACKNOWLEDGEMENTS

The author wishes to thank Andre Cournut and Karim Ben Slimane (ANDRA), David Mellor and Nick Davies (UK Nirex), and Göran Bäckblom (SKB) for their invaluable contributions to the design and implementation of the ZEDEX project. The major contribution to the integrated analysis of the results has been provided by Corinne Bauer, ENSG, Stephen Falls, Keele University, and Leif Stenberg, SKB.

In total 19 different acquisition techniques were used by 11 different organizations coming from 6 countries in all. The positive spirit within the project group and the good collaboration with the excavation (Skanska) and drilling (Kärnborning) contractors made it possible to successfully perform all measurements planned and to complete the project within the allocated time and budget.

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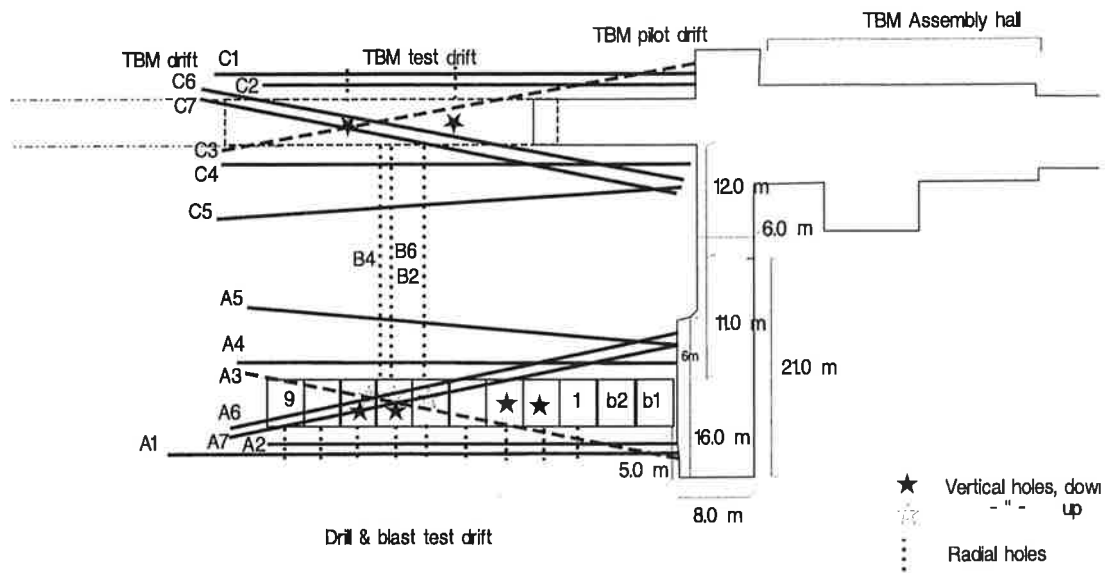


Figure 1 Experimental configuration for the ZEDEX Project, plan section.

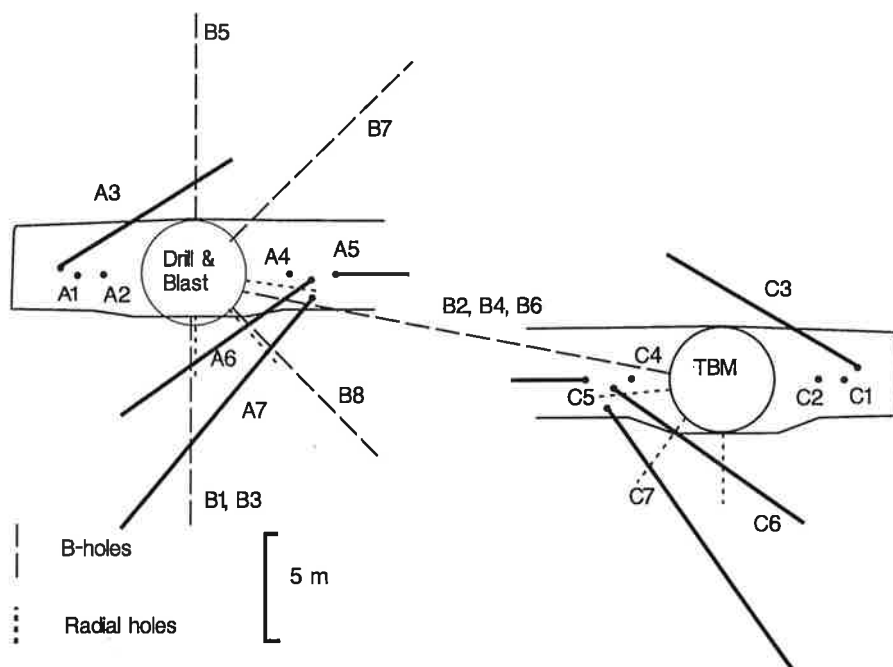


Figure 2 Vertical section showing location of boreholes in relation to test drifts.

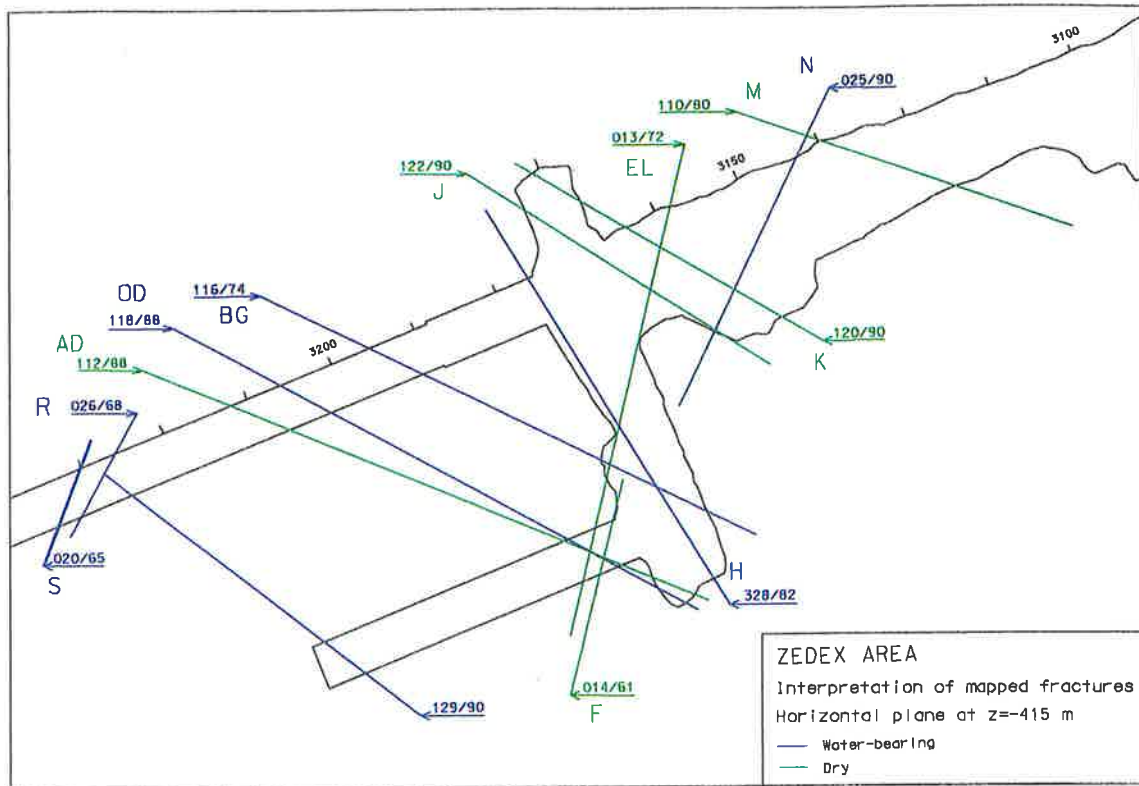


Figure 3 Interpretation of mapped fractures and selected radar and seismic reflectors.

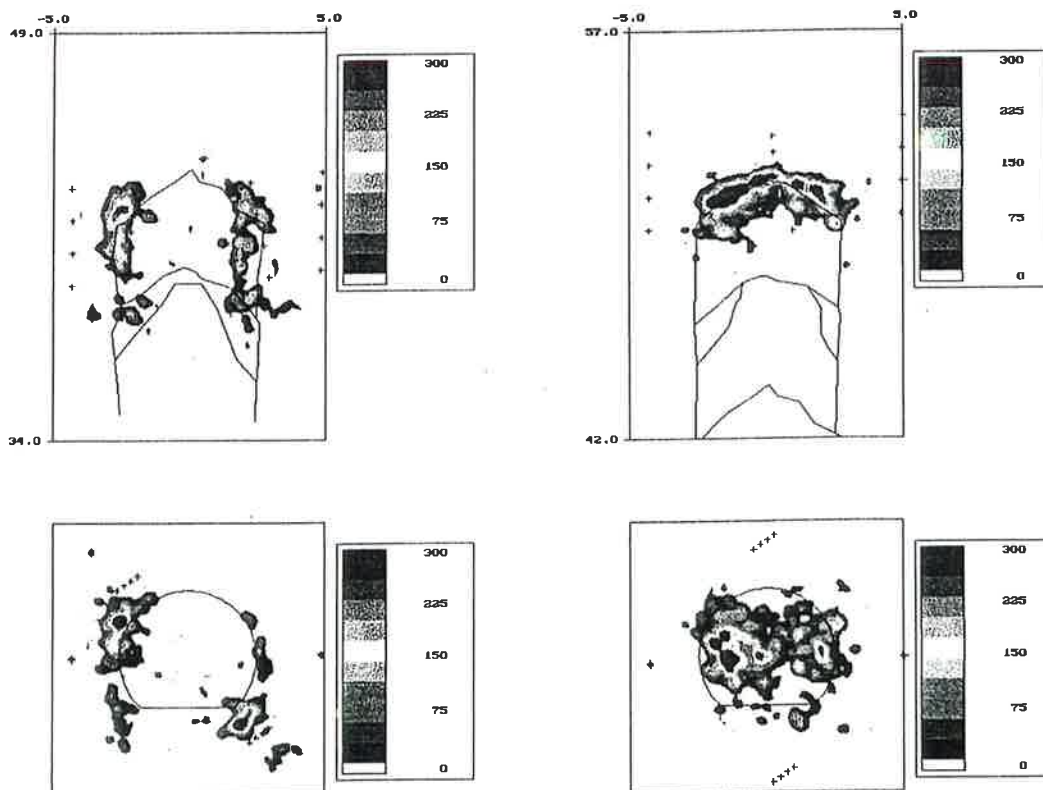


Figure 4 AE source locations recorded after rounds 3 and 5.

## SAMMANFATTNING

Byggande av schakt och tunnlar i ett djupförvar för använt kärnbränsle kommer att orsaka en störning på omgivande berg som kan medföra minskning av bergets stabilitet och förändrade hydrauliska egenskaper som måste beaktas vid utformningen av ett djupförvar för använt kärnbränsle och utvärderingen av dess långsiktiga säkerhet. För att belysa dessa frågor har ANDRA, UK Nirex och SKB gemensamt organiserat ZEDEX-projektet som genomförs vid SKB:s Äspölaboratorium beläget norr om Oskarshamn. Projektets mål är att studera den störda zonen med avseende på orsaken till förändringar i bergets egenskaper, förändringarnas storlek, störda zonen utbredning samt dess beroende av brytmetod. Projektet innefattar också test av utrustning och teknik för att kvantifiera den störda zonen. ZEDEX-projektet innefattar test av "normal" drivning medelst sprängning, försiktig sprängning och tunnelborrning.

ZEDEX genomförs i två parallella orter belägna 420 m under markytan. En tunnel har borrats med TBM och en har sprängts. Båda tunnlarna har cirkulär profil och 5 m diameter. Ett antal undersökningshål har borrats dels längs med de båda tunnlarna och dels radiellt. Den störda zonen egenskaper har kvantifierats genom mätning av: acceleration, bergrörelser, seismisk hastighet, observation av naturliga och inducerade sprickor, temperatur, akustisk emission från mikrosprickbildning och bergspänningar. Dessutom har seismik och radar använts för att beskriva rumsliga förändringar i bergets egenskaper.

Resultaten visar på små förändringar i bergets egenskaper på avstånd större än 1 m från ortväggen (fjärrzonen) för samtliga brytmetoder. Mätningarna av bergrörelserna visar på elastiska rörelser i fjärrzonen. De seismiska mätningarna visade inte på några mätbara förändringar i fjärrzonen. Mycket lite akustisk emission observerades på avstånd större än 2 m från ortväggarna och fördelningen av de observerade händelserna var tämligen lika för den TBM borrarde och den sprängda orten. Hydrauliska mätningar visade på förändringar i fjärrzonen runt båda tunnlarna men resultaten är inkonsistenta och svåra att tolka. Akustisk emission och förändringar i hydrauliska egenskaper i fjärrzonen kan orsakas av förändringar i sprickapertur eller skjuvning längs befintliga sprickor.

Mätningarna har visat att det finns en skadad zon runt orterna som är större för den sprängda orten än för den borrarde. Den skadade zonen karakteriseras av inducerade sprickor, ökad permeabilitet och minskad seismisk hastighet. Skadezonens utbredning runt TBM tunneln uppskattas till mindre än 20 cm, till c:a 30 cm i väggen och c:a 80 cm i sulan på den sprängda tunneln. Utbredningen av skadezonen varierade beroende på de geologiska förhållandena och utfallet av sprängningen. Den större skadezonen i sulan är troligen en effekt av den större laddningen i bottenhålen och sulans platta form. Sprängsalvor med två olika borrh- och laddplaner provades men någon signifikant skillnad i skadezon erhöles inte.



# CHARACTERIZATION OF THE EXCAVATION DISTURBED ZONE CAUSED BY THE BORING OF THE FULL SCALE DEPOSITION HOLES IN THE RESEARCH TUNNEL AT OLKILUOTO

Jorma Autio, Saanio&Riekkola Oy

## 1 INTRODUCTION

Three holes of the size of deposition holes (depth 7.5 m and diameter 1.5 m) were bored in hard rock in the Research Tunnel at Olkiluoto in Finland, Figure 1. A novel full face boring technique was used which was based on rotary crushing of rock and removal of crushed rock by vacuum flushing through the drill string, Figure 2 & 3.

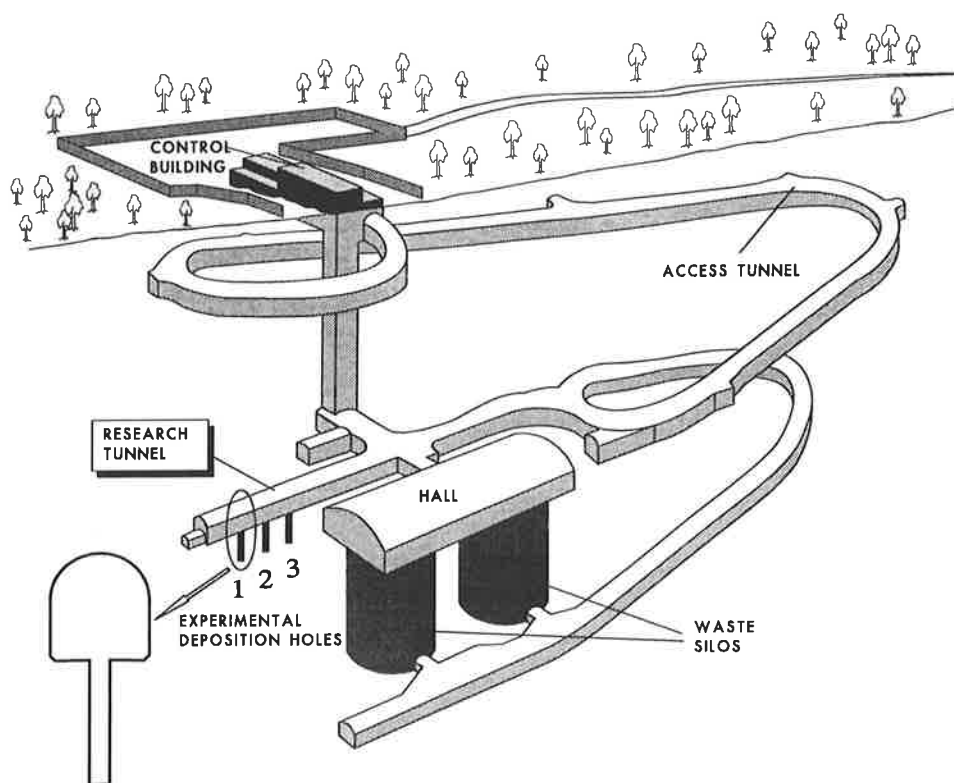


**Figure 1.** VLLJ-repository, Research Tunnel and full scale experimental deposition holes.

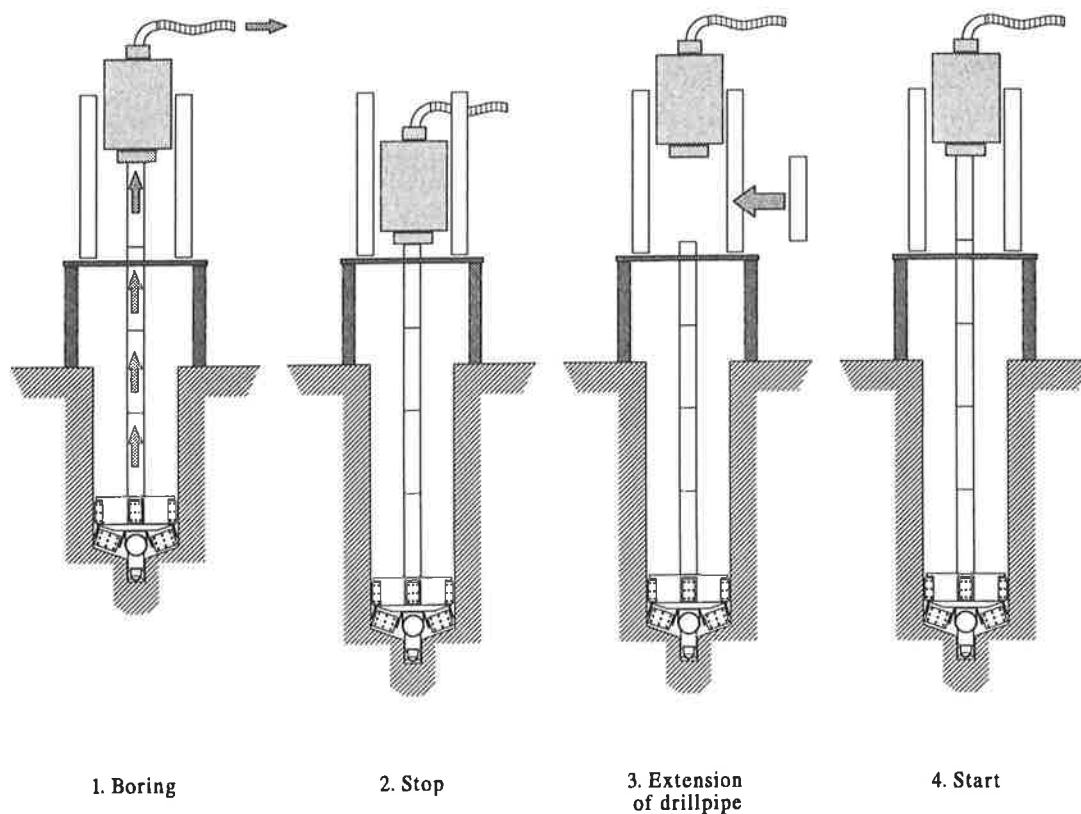


**Figure 2.** *Operating principle of the boring method.*

The boring of experimental full scale deposition holes was accompanied by comprehensive pre- and post-characterization of the rock in the near field of the holes with geophysical, geological, hydraulic and rock mechanical tests.

Tests were carried out during the boring in order to find out the effect of changes in operating parameters to the performance of the boring machine and the quality of the hole. The evaluation of the quality of the holes included studies on the geometry of the holes, measurement of surface roughness with a laser profilometer and study of excavation disturbance in the zone close to the surface of the holes with two novel methods, namely He-gas method and  $^{14}\text{CPMMA}$ -method.

Both the boring of the full scale experimental deposition holes and the characterization of the holes was carried out by Swedish Nuclear Fuel and Waste Management Co. (SKB) and Teollisuuden Voima Oy (TVO) in Finland in co-operation.

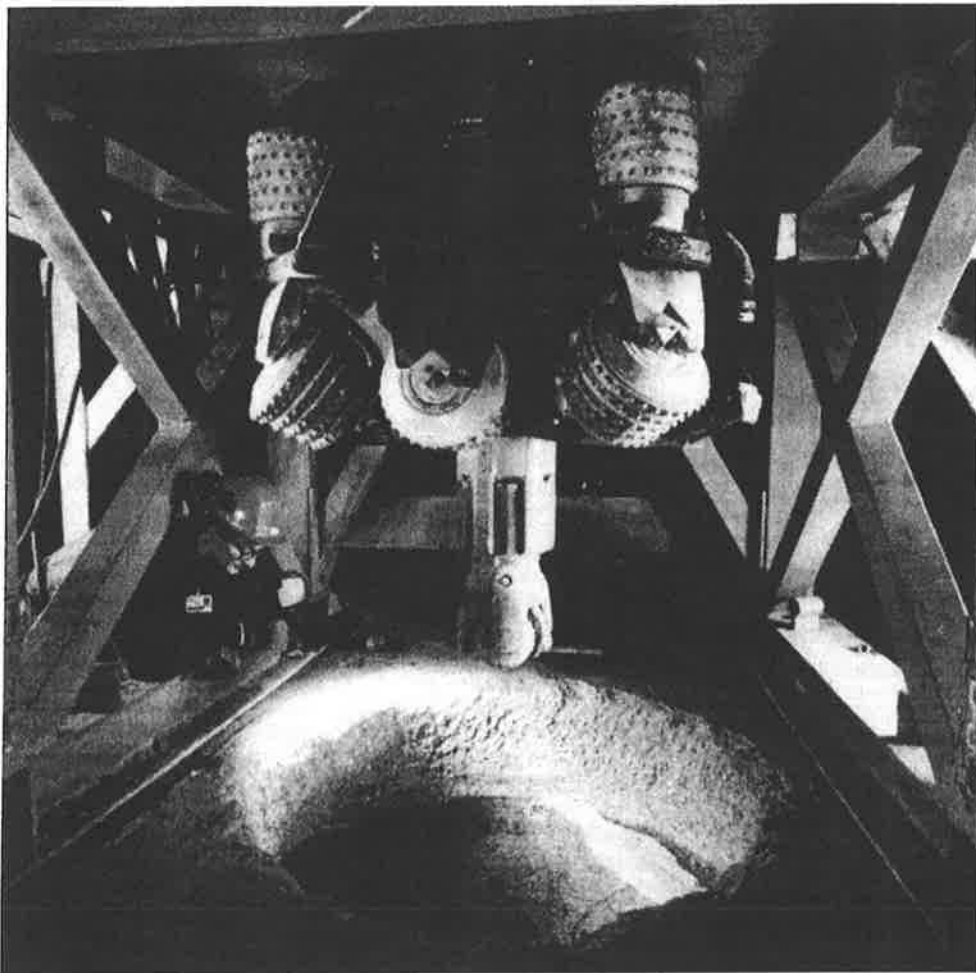


*Figure 3. The cutter head on top of one of the experimental deposition holes.*

## 2 THE RESEARCH TUNNEL AT OLKILUOTO

The Research Tunnel is located 60 m deep in the VLJ-repository which is an underground disposal facility for the low and medium level waste generated at the Olkiluoto nuclear power plant. The repository is located within 1 km from TVO I and TVO II BWR's on the Olkiluoto island on the southwest coast of Finland. The repository was taken into operation in 1992.

The Research Tunnel has been excavated using conventional drill and blast technique. It is located in an east-west striking tonalite formation which is surrounded by mica gneiss, Figure 4. The rock types found in the Research Tunnel are gneissic tonalite and pegmatite. Tonalite is usually slightly foliated, medium grained, massive and sparsely fractured. Besides gneissic tonalite, which is referred as anisotropic tonalite, a fine

grained, isotropic tonalite variant is met. Pegmatite is non-foliated, coarse grained, massive and sparsely fractured. The main minerals of the anisotropic tonalite were quartz, plagioclase, biotite and hornblende which total to about 94 % of the total content. The mafic minerals, especially oblong grains of biotite and hornblende are oriented. The grains are subhedral and the alteration to secondary minerals is insignificant.

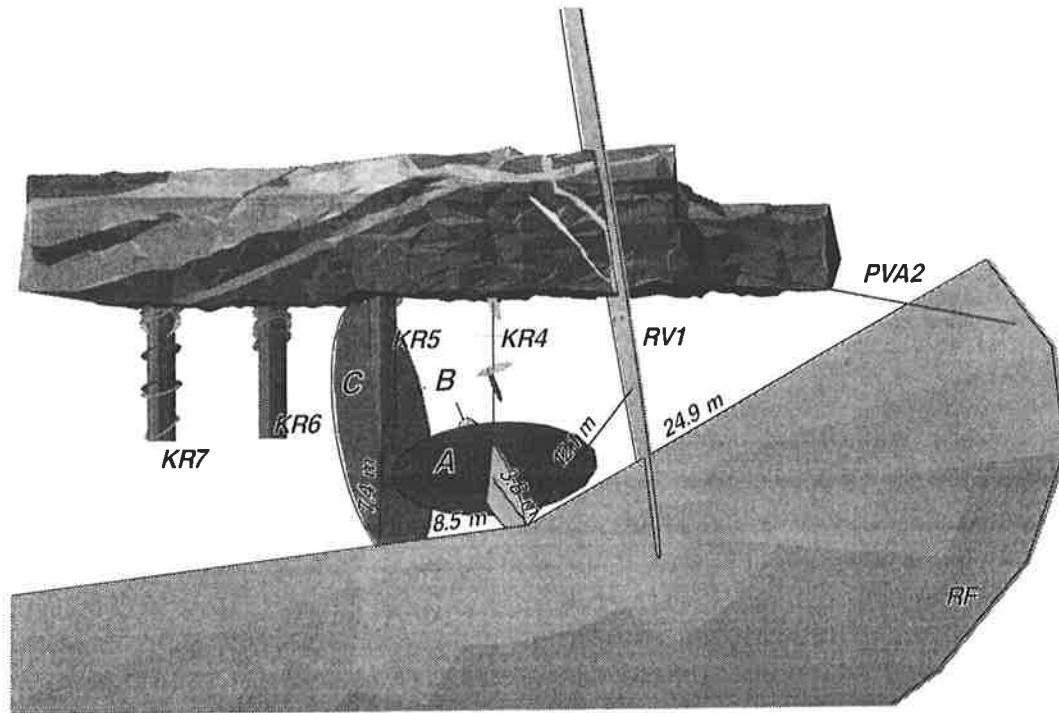


**Figure 4.** 3-D model of the Research Tunnel showing the hydraulically conductive fracture zones in the vicinity of the tunnel and the three full scale experimental deposition holes.

The state of stress at the VLJ-repository and Research Tunnel area is low. The measured values of in situ stresses were in average from 5 to 6 MPa at the level of the repository.

The rockmechanical properties, drillability values, abrasiveness and mineralogy were determined (Johansson & Autio 1993, Johansson & Autio 1995). The compressive strength of the anisotropic tonalite was 80 MPa, the tensile strength was 9 MPa, the average DRI-value was 55 (Figure 5), the average brittleness value  $S_{20}$  was 51.5, the Sievers' SJ value 32.4 and the CAI-value (Cerchar Abrasivity Index) of the anisotropic tonalite which describes the abrasivity was 3.8, Figure 6.

The geology of the full scale holes was mapped by Äikäs&Saclén (1995) and the rock was observed to be sparsely fractured.

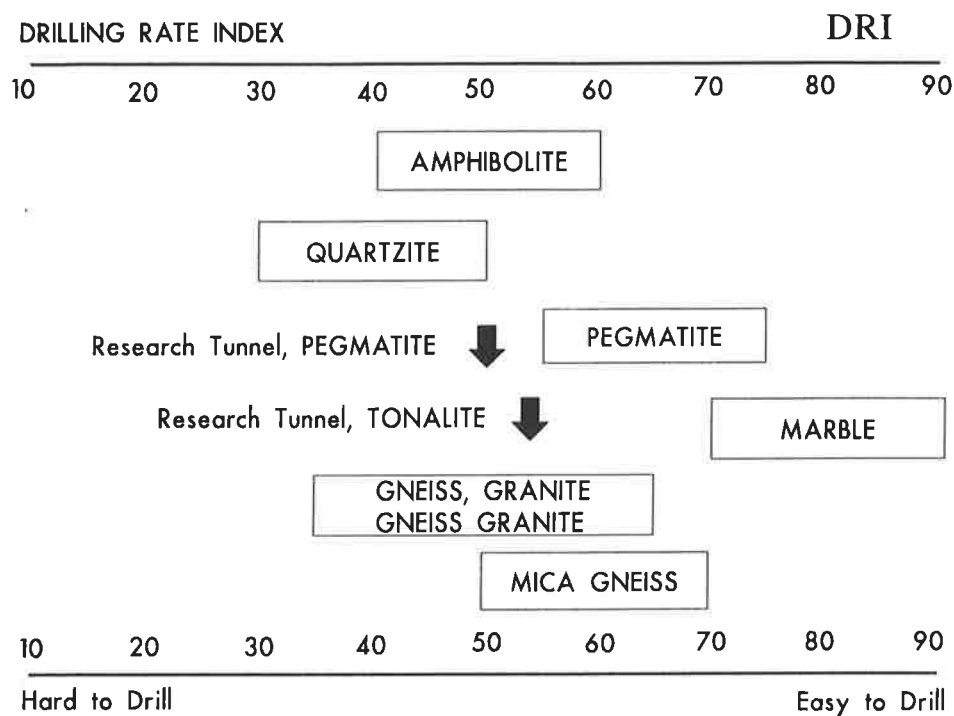


**Figure 5.** Variation of DRI in various rock types and the average DRI-values determined for the rock types of the Research Tunnel (Johansson 1993).

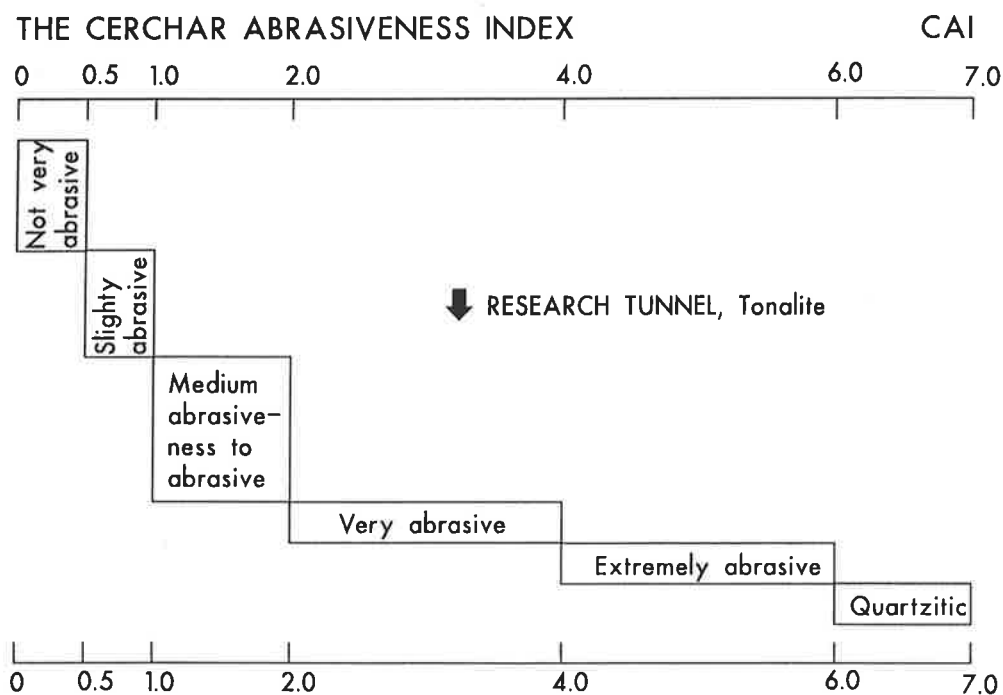


**Figure 6.** The average CAI-value of anisotropic tonalite with respect to abrasiveness classification.

### GEOMETRY OF THE EXPERIMENTAL FULL SCALE DEPOSITION HOLES AND SURFACE ROUGHNESS

The diameter of the holes was measured as the minimum peak to peak diameter to different directions with a depth interval of 1 m. The ratio of the diameters was calculated to find out the ellipticity of the holes.

The coordinates of the centerline of the hole were measured and the straightness and curvature of the holes were determined with these. The measurements were carried out at a depth interval of 1 m.

The geometry was measured with standard type of surveying equipment such as tacheometer of type Kern E2-DM504 and theodolite of type Kern DKM-2A.

The surface roughness of three experimental full-scale deposition holes was measured by optical linear laser profilometer, Figure 7 & 8 (Halttunen 1995).

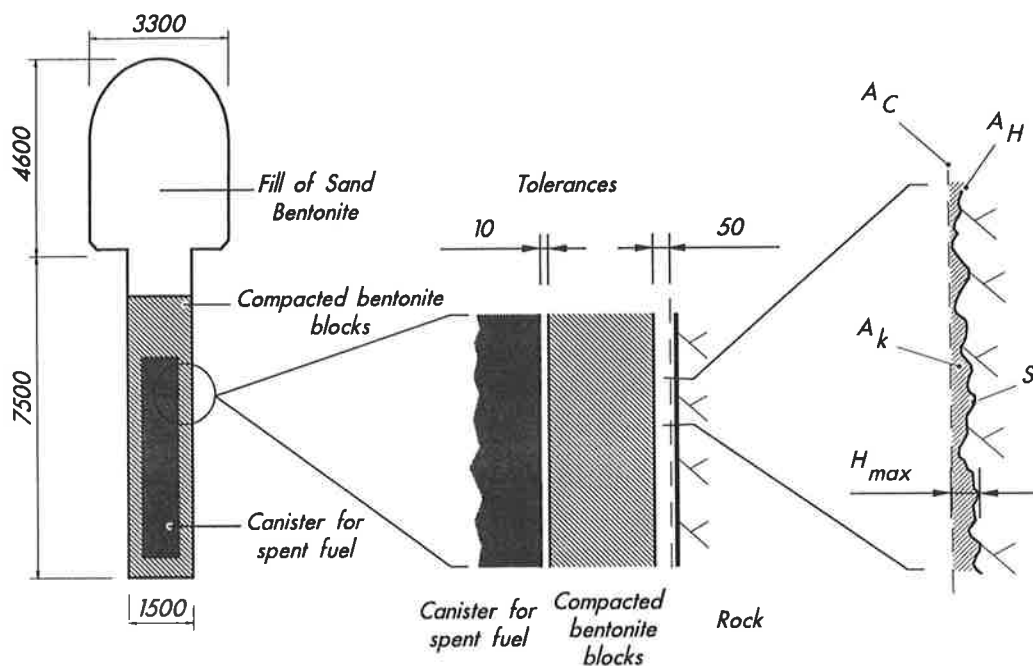


**Figure 7.** Unevenness of the surface of deposition holes.

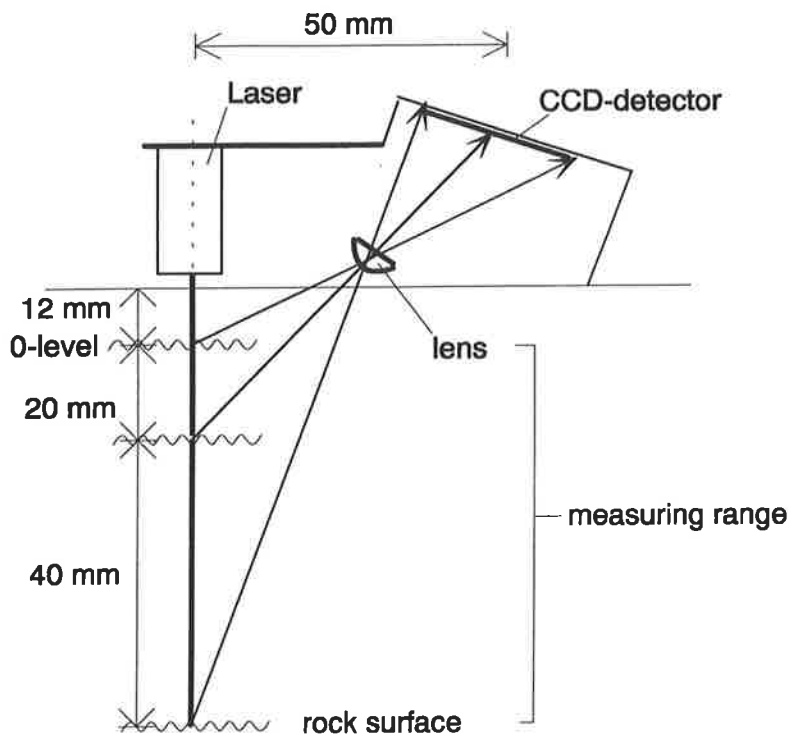


**Figure 8.** The principle of laser measurement.

The roughness parameters which were calculated from the measurements of the surface topography were:  $R_a$  and  $R_z$ (ISO), which describe differences in profile height, and  $S_m$ , which describes the differences between the spacing of peaks in the profile.

Furthermore, two  $R_q$  parameters (root mean square parameters of amplitude), determined from the filtered profiles were included. These so called mega and macro roughness values were calculated from two filtered profiles. The macro-value is estimated by eliminating the wavelengths below 0.5 mm and above 50 mm from the measured profiles. The filters of mega-values are 50 mm and 500 mm.

The average values of the roughness parameters in different holes did not differ significantly. Distinct variations were found in amplitude parameters of holes in relation to hole depth as seen in Figure 9. The variations were most distinct in the  $R_z$ -parameter although consistent variations occurred also in parameters  $R_a$  and mega-roughness. Most of the culmination points of the roughness parameter curves corresponded to positions where the drill string of the boring machine had been extended and therefore the boring had stopped and the cutter head had been lifted.

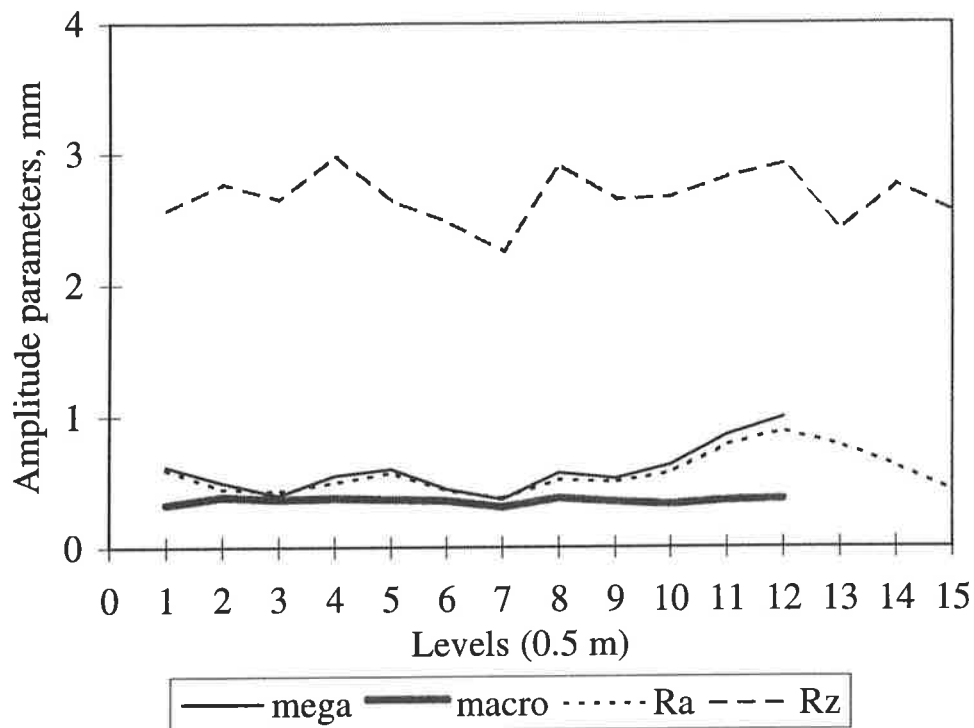


**Figure 9.** Values of amplitude parameters of surface roughness on different levels in the full scale hole 2. The values are averages of four vertical lines.

#### 4 EXCAVATION DISTURBANCE CAUSED BY BORING

The disturbance caused by boring on the rock was studied with two novel methods, namely the  $^{14}\text{CPMMA}$ -method (Siitari-Kauppi 1995) and the He- gas method (Hartikainen et al. 1995).

The disturbance was studied in laboratory from  $\varnothing 98$  mm core samples which were taken from different parts of the holes and represented different boring parameters (Autio et al. 1995), Figure 10.

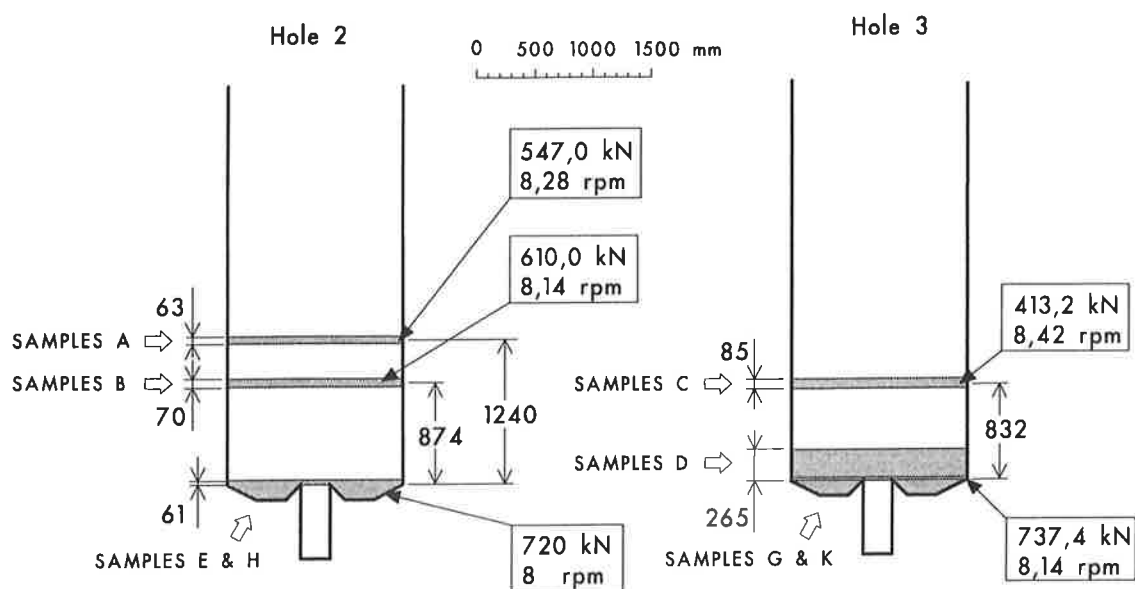


**Figure 10.** Location of sampling sections of experimental full-scale deposition holes with corresponding boring parameters (Autio et al. 1995)

## 5 <sup>14</sup>CPMMA-METHOD

The mechanical disturbance of the side walls were determined in terms of geometry of microfractures and porosity profiles with <sup>14</sup>CPMMA-method (Siitari-Kauppi 1995). The novel method and the applications of it have been described earlier in detail by Hellmut et al. (1993&1994).

The method involved impregnation of the rocks with <sup>14</sup>C methylmethacrylate, irradiation polymerization, sample partitioning (Figure 11), autoradiography (Figure 12) and optical densitometry with digital image processing techniques.

Scanning electron microscopy (SEM) and energy dispersive X-ray analysis (EDX) were used in addition to the <sup>14</sup>CPMMA-method to investigate in greater detail the pore apertures and minerals in some porous regions. SEM/EDX studies were conducted on <sup>14</sup>CMMA-impregnated samples.

The disturbed zone occurred as a zone of increased porosity and a zone of distinct porous tracelines which extended deeper in the rock matrix than the zone of increased porosity (Figure 12, 13 & 14). The total porosities increased at depths of 0-10 mm in the disturbed zone in samples cored from the walls of the holes.

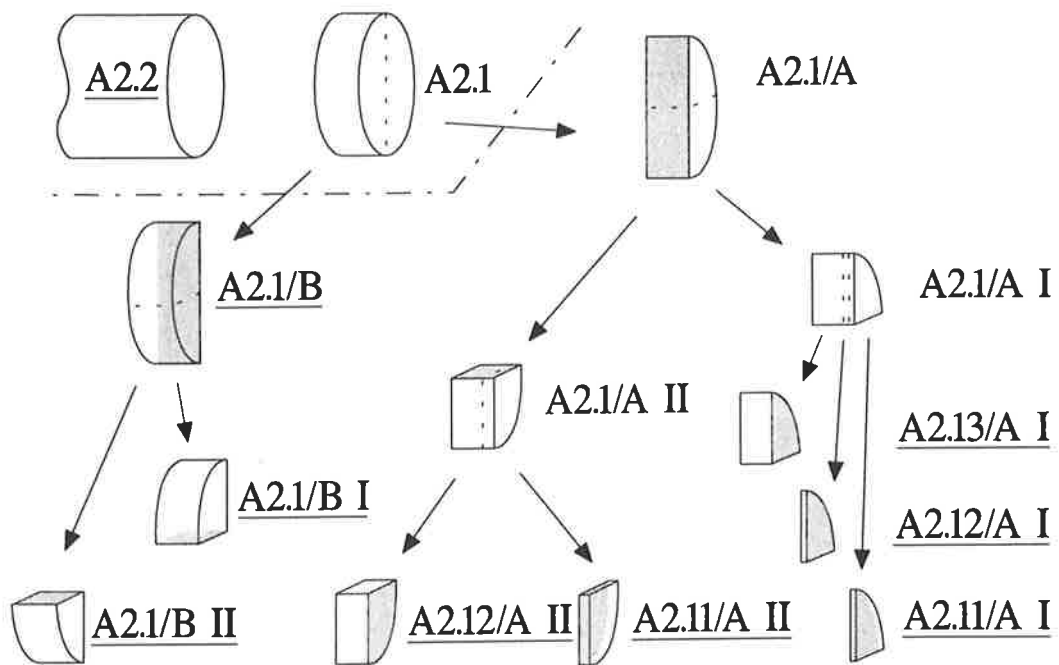


**Figure 11.** Partition diagram for Ø 98 mm core sample A3 for the  $^{14}\text{CPMMA}$  study from the full scale experimental deposition hole 2. The exposed surfaces were studied.

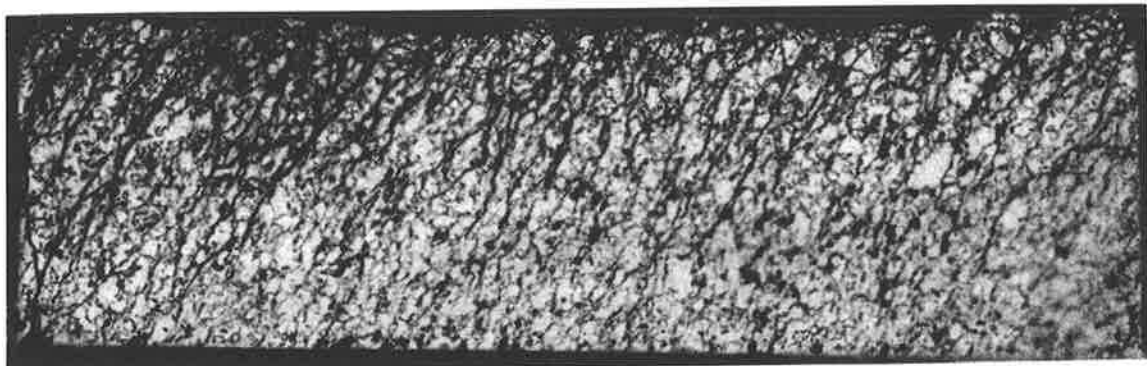


**Figure 12.** Example of a autoradiograph, sample C2 (Siitari-Kauppi 1995). The upper surface of the sample is section of the surface of the hole 3. The plane of the section is perpendicular to the surface. Different levels of gray shades represent different levels of porosity.

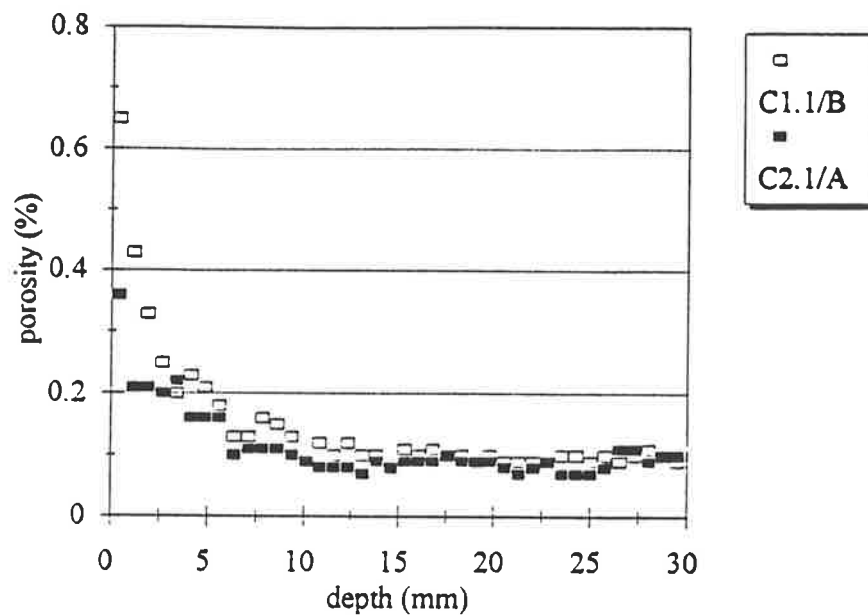


**Figure 13.** Porosity profiles of samples of C-series. Section of sample C2 is shown in Figure 12. The porosity value is presented in relation to depth from the disturbed surface (Siitari-Kauppi 1995).

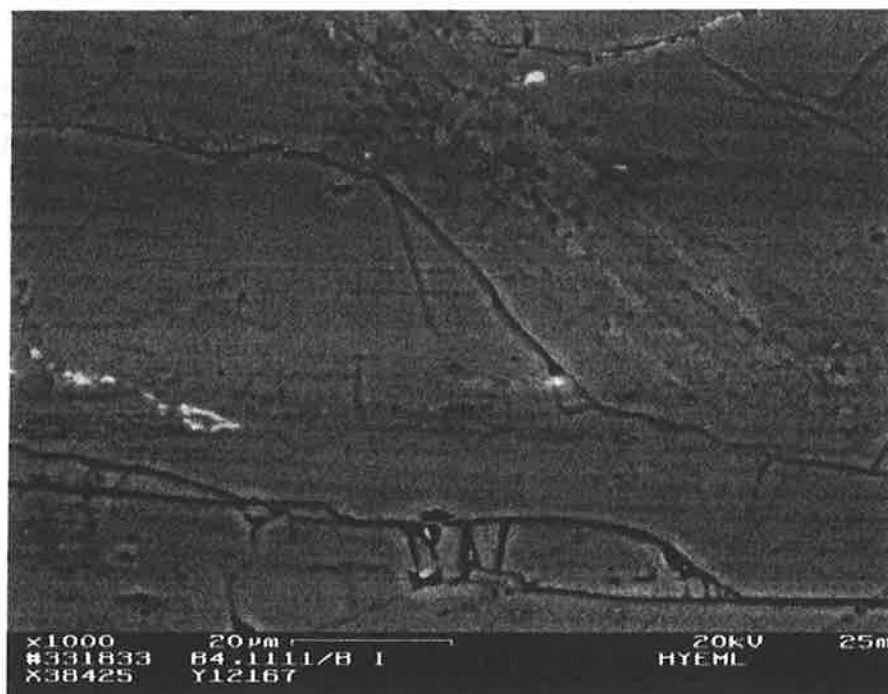


**Figure 14.** The microstructure of disturbed tonalite, SEM/EDX, 1000x magnification (Siitari-Kauppi 1995).

The disturbance of the side wall rock was determined from samples taken from six positions in the walls of the holes. The disturbance was determined in terms of effective diffusion coefficients and permeability coefficients with a novel He- gas method (Hartikainen et al. 1994, 1995).

The method is used with two techniques, one is based on pure diffusion of helium gas (Figure 15) and the other on flow of helium through the sample caused by a pressure gradient (Figure 16).

Both techniques allow easy and effective determination of diffusion and permeability coefficients. Examples of measured and calculated breakthrough curves of through diffusion measurement of sample B3.1 and flow rates for permeability measurement are shown in Figure 17. The measuring time of the diffusion coefficient of the sample B3.1 was 23 hours. The length of the sample was 55.6 mm and the diameter was 98 mm.

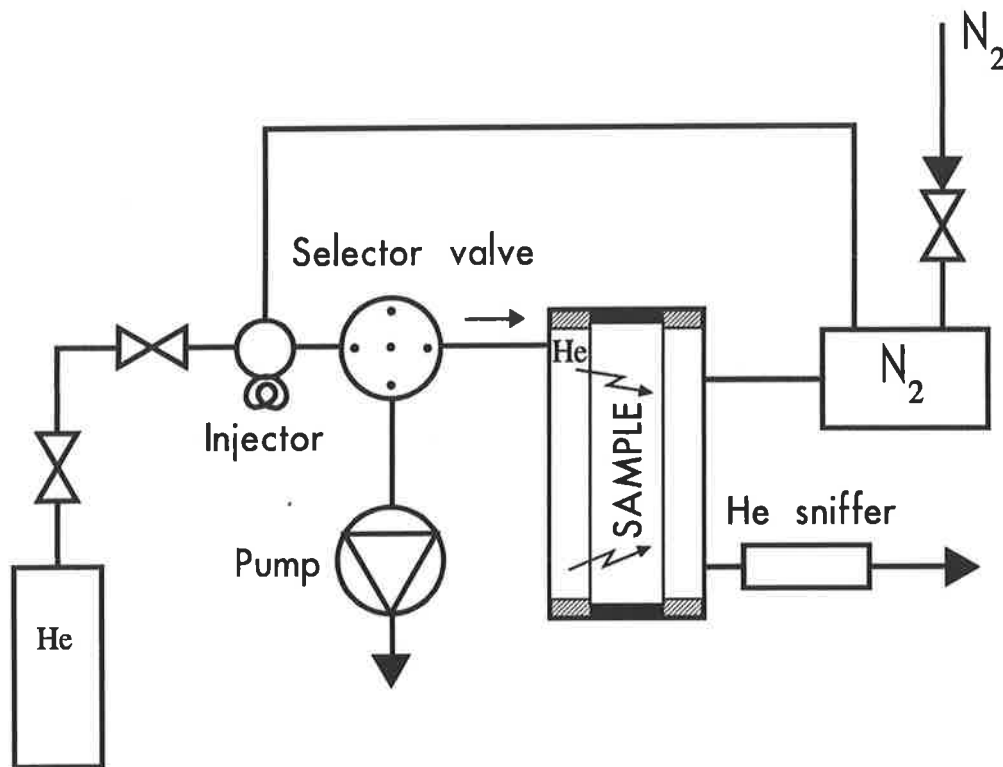


**Figure 15.** The experimental set up for the through diffusion measurement (Hartikainen et al. 1995).

The diffusion and permeability coefficients were measured from 30 samples of both undisturbed and disturbed rock. According to the measurements there is a zone of higher permeability and diffusivity next to the surface of the holes. The thickness of this zone was calculated and the maximum depth extent of separate microcracks was estimated on the basis of the measurements.

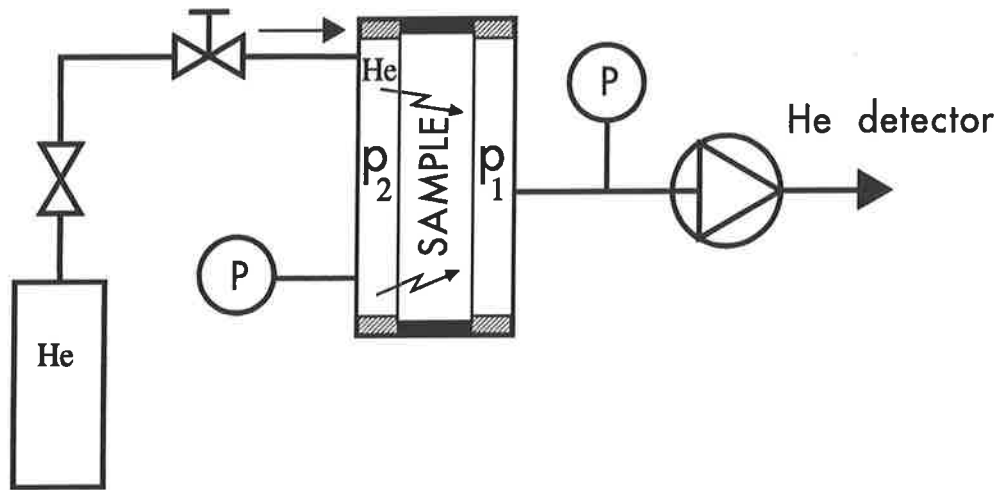


**Figure 16.** The experimental set up for the permeability measurement (Hartikainen et al. 1995).

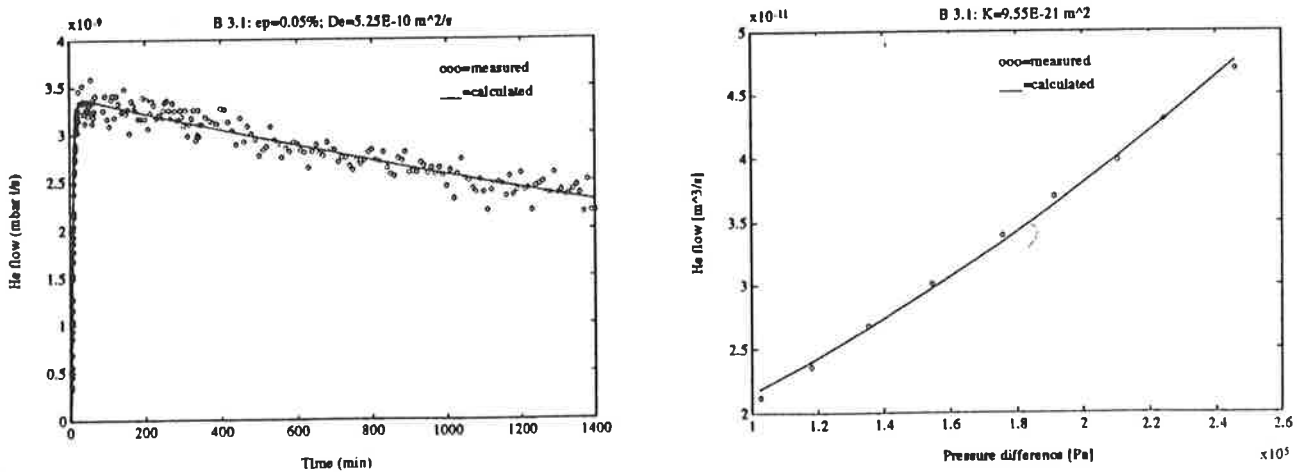


**Figure 17.** Sample B3.1 of length 55.6 mm and diameter 98 mm. Measured and calculated breakthrough curves of through diffusion measurement (on left) and measured flow rates and corresponding curve fit of permeability measurement (on right).

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## SUMMARY

Three holes of the size of deposition holes (depth 7.5 m and diameter 1.5 m) were bored in hard rock in the Research Tunnel at Olkiluoto in Finland. A novel full face boring technique was used which was based on rotary crushing of rock and removal of crushed rock by vacuum flushing through the drill string. During the boring tests were carried out in order to find out the effect of changes in operating parameters to the performance of the boring machine and the quality of the hole.

The interest in the quality of the hole was focused on the surface roughness of the hole and the mechanical disturbance specified as increased microfracturing and porosity in a zone next to the surface. The evaluation of the quality of the hole included studies on the geometry of the holes, measurement of surface roughness with a laser profilometer and study of excavation disturbance in the zone close to the surface of the holes with two novel methods, namely He-gasdiffusion method and  $^{14}\text{CPMMA}$ -method.

The total porosities which were measured with the  $^{14}\text{CPMMA}$ -method indicated a disturbed zone of increased porosity up to a certain depth from the surface. The porous tracelines were observed to penetrate even deeper from the surface.

The diffusion and permeability coefficients were measured with the He-gas method indicated a zone of higher conductivity and diffusivity next to the surface of the holes.



# **BERGFÖRSTÄRKNING**

## **Rock Reinforcement**

Introduktion: Ove Stephansson, KTH



# FULLY GROUTED BOLTS SUBJECTED TO BLAST VIBRATIONS

Gyste bergbolter utsatt for sprengningsrystelser

Dr.ing. Gisle Stjern, SINTEF Rock and Mineral Engineering, Trondheim, Norway

## 1. ABSTRACT

This study comprises pull-out testing of bolts and vibration measurements on both the rock and the bolts subjected to close proximity blasting. Laboratory testing of compressive and flexural strength of the grout and analysis of polished thin sections of specimens consisting of bolt, grout and rock are presented. There is almost no reduction in pull-out strength for bolts installed close to the blast (3.4 m), compared to more distant installed bolts (22.0 m). Furthermore, there is no difference in pull-out strength for bolts with freshest grout compared to bolts with cured grout, when subjected to blast loadings. Anchorage length of the bolts were 125 mm, which gave an average pull-out strength of 140 kN, or 93 % of the yielding load of the steel. Analysis of polished thin sections of the rock, grout and bolt do not show any variation in crack pattern or crack frequency between bolts with varying distance to the blast or varying curing of the grout when subjected to the blast loadings. Repeated pulling of previously pulled bolts show a healing effect of the grout. None of the tests carried out indicate any reduction of the bolt/grout performance. Therefore, bearing capacity of fully grouted bolts is maintained in hard rock tunnelling.

## 2. INTRODUCTION

When tunnels or other underground structures are constructed by conventional drill and blasting, the need for safe support requires the installation of rock bolts in close proximity to the blast source. In Norway, the use of fully grouted rebars has not been recommended close to the blast source because of supposed damage of the grout by blast induced vibrations, Jorstad (1967). This has led to the practice that fully grouted rebars used as secondary support are placed at least some 30 - 50 m behind the face. There has been no design criteria developed for this practice, neither on minimum distance to the face nor on the maximum level of blast induced peak particle velocity (PPV). Many have believed the practice to be unnecessary conservative.

Some practical experience has been gained in the Boliden Mines in Sweden. Where fully grouted rebars have been used in production areas without any noticeable loss of function, Krauland (1993). Stillborg (1984), showed that there was no reduction in cable bolt performance when these have been exposed to PPVs of 500 mm/s. Littlejohn et al. (1987), investigated the influence of blasting on resin anchored rock bolts. A major finding was that the safe distance to the blast source could be reduced to 1 m without detectable damage to the anchorage of the bolts.

The scope of the investigations was to find:

- The vibration influence with respect to particle peak velocity (PPV) on the performance of fully grouted rock bolts.
- The grout sensitivity to blast vibrations according to grout age.

### 3. TEST-SITE

The tests took place in the Grong copper and zinc mine located about 200 km north of Trondheim, Norway. The massive sulphide orebody is situated in greenstones of the Norwegian Calodenides. The experiments were done in an exploration drive in the upper parts of the western orebody, in an area not influenced by production blast vibrations. The cross section of the drive is 6 by 6 m. The rock mass is competent and the roof of the drive was unsupported before the tests commenced. Table 1 shows the properties of the rock at the test site. The host rock at the site was greenstone in the area of the blast charge and pyrite where the most distant bolts were installed.

*Table 1. Properties of the rock at the test site*

|                               | Pyrite                 | Greenstone                       |
|-------------------------------|------------------------|----------------------------------|
| Bulk density                  | 37.1 kN/m <sup>3</sup> | 27.0 kN/m <sup>3</sup>           |
| Uniaxial Compressive Strength | 135 MPa                | 94 MPa                           |
| Young's Modulus               | 143 GPa                | 40 GPa                           |
| Poisson's ratio               | 0.21                   | 0.10                             |
| Sound velocity                | 6320 m/s               | 2600-3330 m/s ( <i>in situ</i> ) |

The rock temperature on the site is 8.5°C.

## 4. TEST PROCEDURE

### 4.1. Test Arrangement

The 20 mm rebar bolts ( $500 \text{ N/mm}^2$ ) were placed at four different distances from the blast source in order to obtain different magnitudes of the vibrations. Different curing age was achieved by installation of bolts at seven different times preceding the blast event, ranged from 65 hours to 9 hours before the blast. Pull-out tests of installed bolts were performed to find the increase in anchorage capacity, and thereby evaluate the curing level at different times. These tests were needed since curing is a time and temperature dependent process.

Initial blasting tests were carried out at the site to find the dynamic bulk modulus of the rock mass, and the distances for each bolt group from the blast were calculated. The test was designed to get vibrations ranging from 1000 mm/s for the bolts installed closest to the charge, to about 100 mm/s for the most remote bolts. The distances from the blast to the bolt groups were 3.4 m (A), 6.8 m (B), 13.2 m (C) and 22.0 m (D) respectively (Figure 1). The bolts were installed in 51 mm vertical up-holes in groups of 4, with 3 bolts for pull-testing and 1 bolt for overcoring. In total 112 bolts were installed in the roof. For the main test, a total of 8 geophones were mounted both on the rock surface and on the adjacent bolts. This was done to compare the vibrations in the bolt shank to those in the rock. Figure 3 shows the mounted geophones on the rock surface and on an adjacent bolt. The geophones were placed in pairs at the A, B, C and D groups (Figure 1).

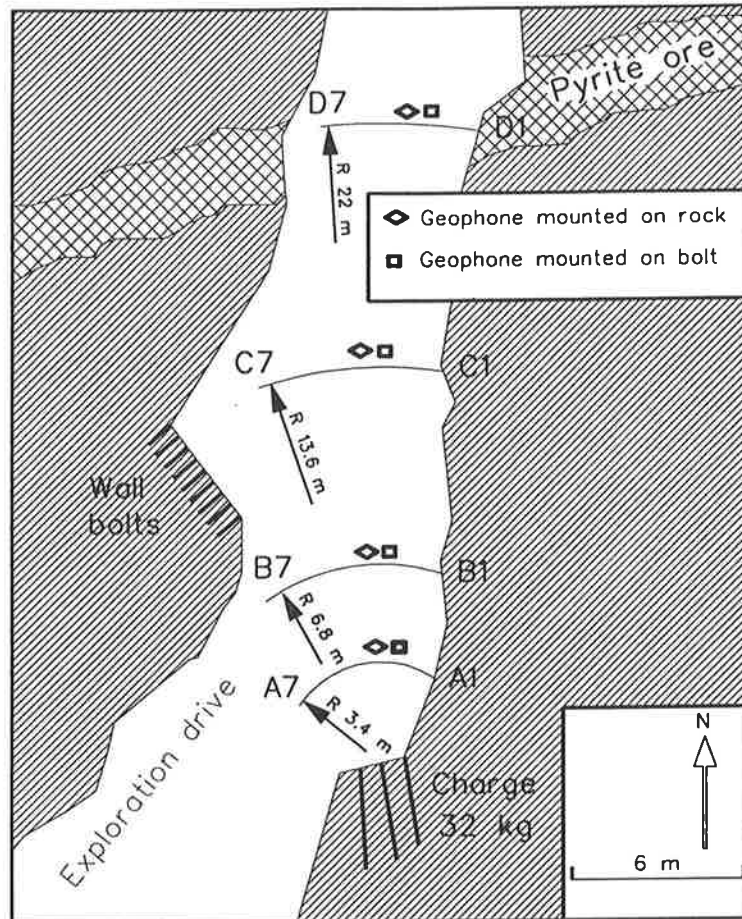


*Figure 1. Plan of the test site, showing location of bolts, geophones and blast charge. Testområde med plassering av bolter, geofoner og salve.*

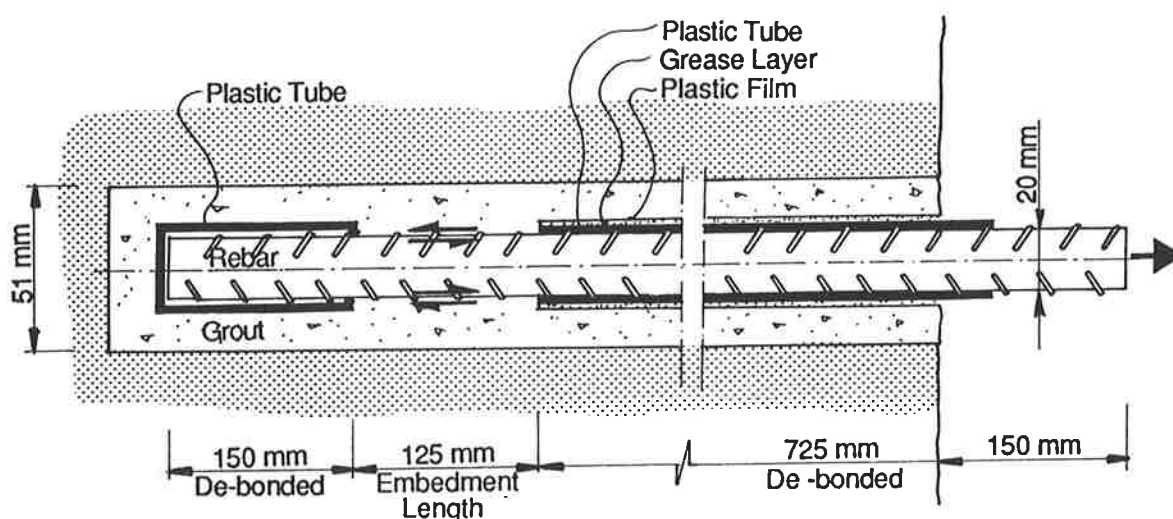
After the blast the bolts were tested by pull-out testing to find the anchorage capacity. The curing age was about 72 hours when the grout was subjected to pull-out testing. In addition one bolt in each group was drilled out by overcoring and prepared in the laboratory. Thin polished sections were made of the core samples and treated with a fluorescent liquid. These sections were illuminated by UV-light in order to map crack patterns caused by the blast vibrations.

#### 4.2. Grout properties and mixing/pumping

The grout used in the tests was a special grout mixture for rock bolts, with a cement content of 47 %. The grout was mixed by first adding water into a mixing vessel and then adding grout while mixing. The water amount was 5.7 litres per 25 kg bag of grout.

### 4.3. Bolt installation and Pull-out testing

The anchorage capacity after the blast was determined by pull-out testing of the 20 mm diameter rebars, with an ultimate tensile strength of 180 kN. The test bolt lay-out is shown in Figure 2. The grouted length of the rebar bolts was 1.0 m with an additional 150 mm of threads protruding from the rock. An anchorage length of 125 mm was chosen to keep the anchorage load below the yielding load of the bolts. The curing age of the grout was about 72 hours when subjected to pull-testing.



The pull-out testing equipment consists of a hollow hydraulic cylinder mounted to a steel cylinder with tri-pod legs. The tri-pod legs facilitate the pulling of bolts which are

not installed perpendicularly to the rock surface, while the steel cylinder enables the bolts to be pulled for a total length of 200 mm. A pressure transducer measures the pressure while the displacement is recorded by a wire potentiometer. The two gauges are connected to a portable PC where the data are stored and processed.

## 5. DETERMINATION OF THE ROCK TRANSMISSION FACTOR AND MAIN CHARGE

In order to calculate and design a charge that would induce vibrations with PPV of 1000 mm/s, test blasting was done at the site to estimate the rock transmission factor (K). Vibrations were measured at various distances from the detonation. Several blastings with two different charge weights were carried out, while vibrations were recorded. The two charge weights were 0.6 and 1.1 kg. The dynamite cartridges were loaded into 34 mm horizontal holes in the wall of the drive. All blasts were confined and no free breakage occurred.

Geophones were mounted at 4, 8 and 16 m from the detonation point, and the vertical peak particle velocity (PPV) of the vibrations was measured. The geophones were connected to a recording instrument, that shows the complete time history sequences of the vibrations. To calculate the rock transmission factor (K), Equation (1) a Norwegian guidance was used, NS 8141 (1993).

$$K = \frac{v \cdot d}{\sqrt{Q}} = 541,6 \cdot 10^{-0.0186 \cdot d} \quad (1)$$

Q = charge weight in kg

v = peak particle velocity (PPV) in mm/s

d = distance from charge point in metres

K = rock transmission factor

The rock transmission factor decreases slightly with distance from the blast point, due to presence of joints and other inhomogenities.

The test charge was calculated as (2):

$$Q = \left( \frac{v \cdot d}{K} \right)^2 \quad (2)$$

A rock transmission factor of 541 and a desired PPV of 1000 mm/s for the closest bolts gave a charge weight of 32 kg (dynamite). This agrees well with the largest charge

weight per detonator period that can be expected from a normal tunnel blast. The charge was loaded into five holes with 7 cartridges (40x400 mm) in each. Millisecond detonators with interval number zero (instantaneous detonations) were used for all holes. The holes were drilled in a pattern similar to that of stoping holes in normal blasts, but the holes were drilled with a large enough burden to avoid free breakage.

## 6. RESULTS

### 6.1. Peak particle velocity

Readings of the geophones closest to the blast failed because the horizontal vibrations exceeded the limits of the geophones. One of the geophones, 6.8 m from the blast, showed normal waveforms, but the PPV exceeded the vertical vibration limit (without damping shunt), which resulted in a flat part at 250 mm/s in the beginning of the curve. Extrapolating of the tangents on both sides of the flat part of the curve, indicated a PPV of 500 - 600 mm/s. The measured PPVs indicate that the charge was sufficient to give the expected vibration of 1000 mm/s for the closest bolts.

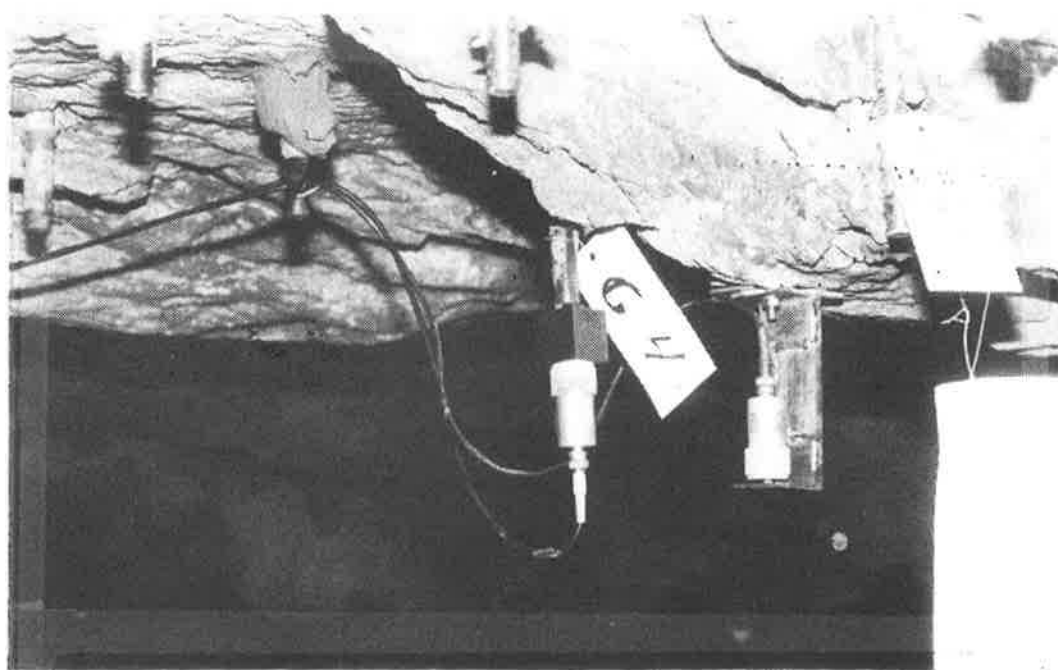


*Figure 3. Geophones mounted on the rock surface and on an adjacent bolt.  
Geofoner montert på bergflate og bergbolt.*

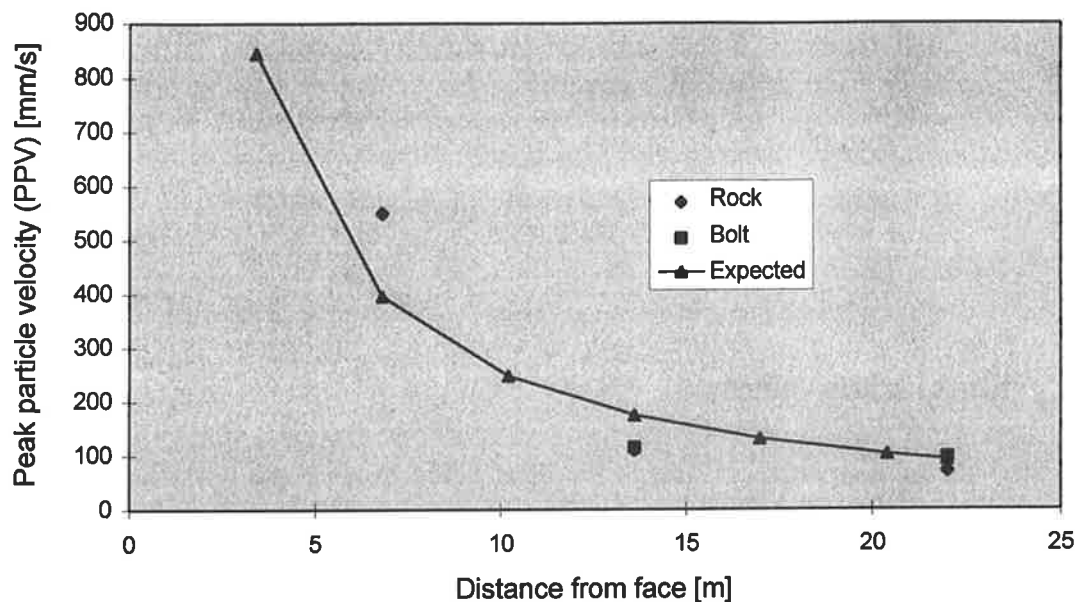


Figure 4. Calculated and measured vibrations from the main blast. Kalkulerte og målte vibrasjoner fra hovedsalve.

## 6.2. Grout testing

Three RILEM (The International Union of Testing and Research Laboratories for Materials and Structures) grout specimens (40 mm x 40 mm x 160 mm) were taken from each batch of the bolt grouting. The moulds were sealed in plastic and left at the site for 72 hours before being moved to the laboratory where they were stored for a total of 28 days. Then the specimens were tested for flexural- and compressive strength according to European standard (EN 196-1, 1992). As seen in Figure 5, the 28 days compressive strength varied between 55 MPa and 70 MPa. Normal compressive strength according to the grout mix manufacturer is 60 MPa.

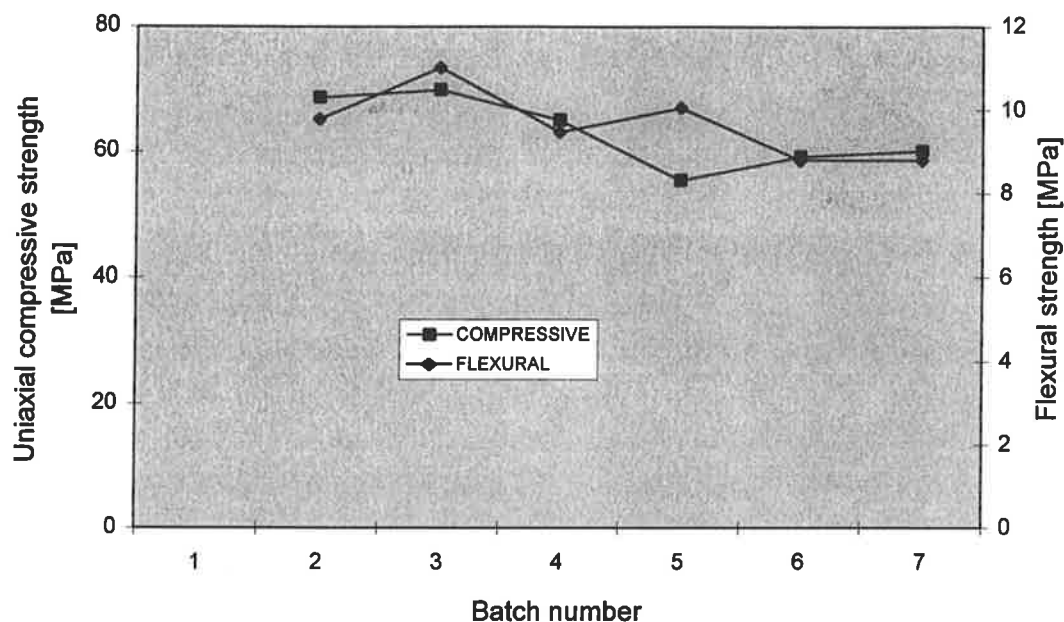


Figure 5. Grout flexural and uniaxial compressive strength. Bøystrekkfasthet og enaksiell trykkfasthet for boltemørtel.

### 6.3. Pull-out tests

40 bolts were installed in the wall before the main tests. These bolts were tested to determine the *in situ* curing of the grout with time. The results are shown in Figure 6. The installation time (time before blast) of the roof bolts was determined from these curves.

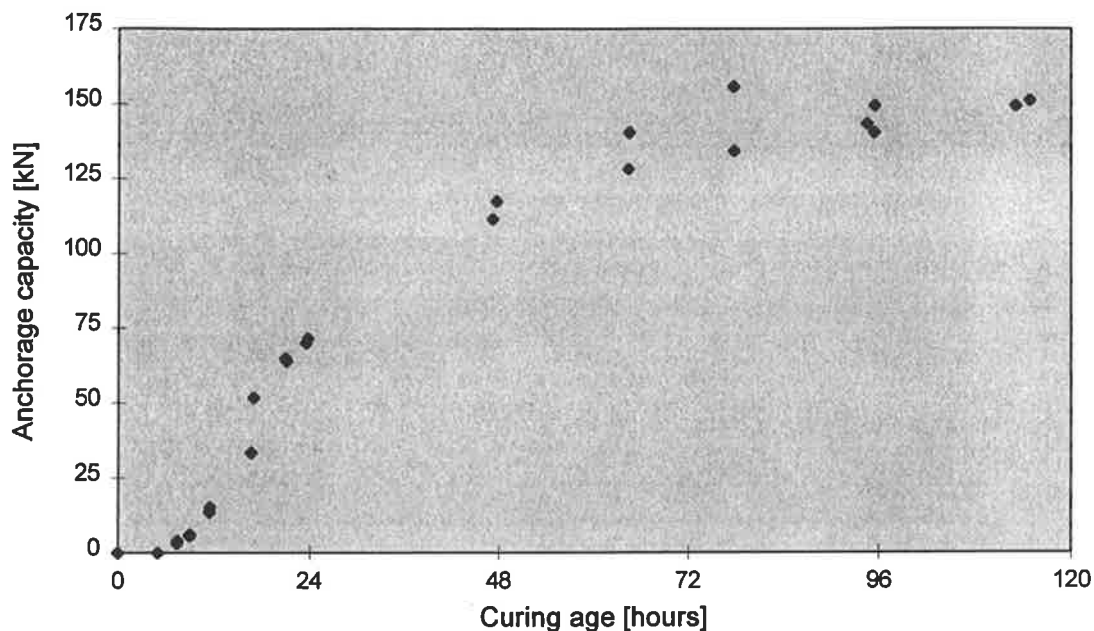


Figure 6. Anchorage capacity for bolts at different curing age. Forankringskapasitet for spesialbolter (forankringslengde 125 mm) ved ulike mørtelalder.

20 of the 80 pulled bolts were tested for both load and displacement. For the remaining 60 bolts, only maximum loads were measured

Figure 7. shows the results from the pull-testing after the blast at about 72 hours after bolt installation. The results from each location are presented as mean values with a confidence interval of  $\pm 1$  standard deviation. As seen on the figure there is virtually no reduction of anchorage capacity of the bolts close to the blast source when compared to the more distant bolts. There is a small reduction for the bolts installed 13.6 m from the face. The cause of this could be local different geological condition, giving lower confinement on the grout annulus. However, no visible changes in rock type or foliation were observed in the area. The anchorage capacity is just below the yielding load of the rebars (180 kN), even though the embedment length is only 125 mm.

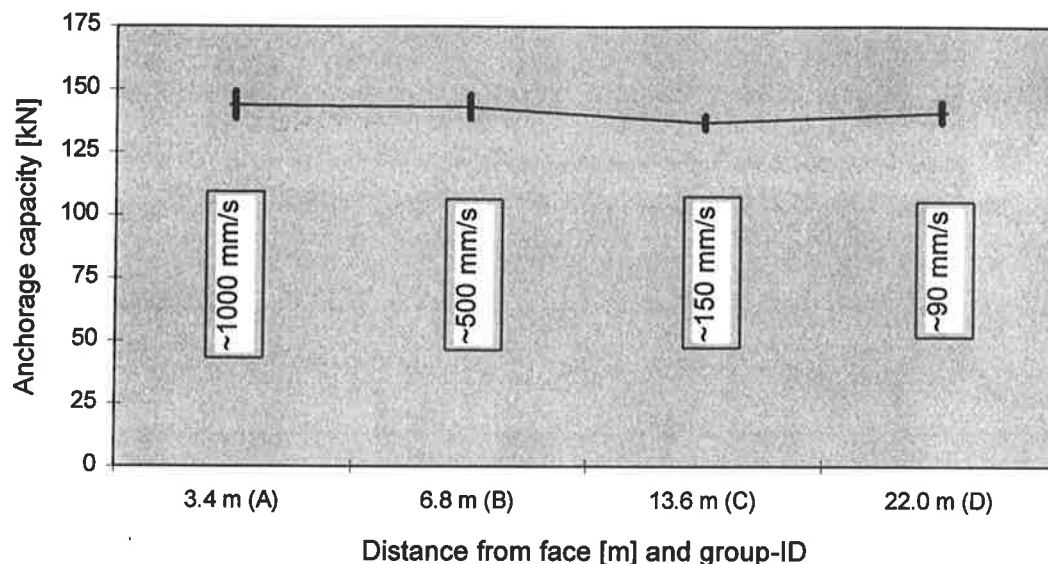


Figure 7. Anchorage capacity as a function of distance from blast source (face).  
*Forankringskapasitet som funksjon av avstand fra sprengningssted.*

Figure 8 shows the anchorage capacity versus batch number (grout age). The results are presented as the mean value for each batch regardless of the distance from the face with a confidential interval of  $\pm 1$  standard deviation. The results between the batches and also within each batch vary more than the results between each location (cf. Figure 5).

The lowest anchorage capacity is recorded for batch number 1. This can be explained by several factors unique to this experiment which could influence the final strength of the grout. Generally, when the water was allowed in the mixing drum, a small volume of water leaked through the seals and into the pumping drum. This could have resulted in different w/c-ratios between the grout batches. During the grouting of batch number 1 there were several tube breakages which delayed the installation for more than one hour. As seen in Figure 8 the lowest anchorage capacity is recorded for bolts grouted from batch number 1. Grouting delays make a stiffer grout which may promote improper steel embedment.

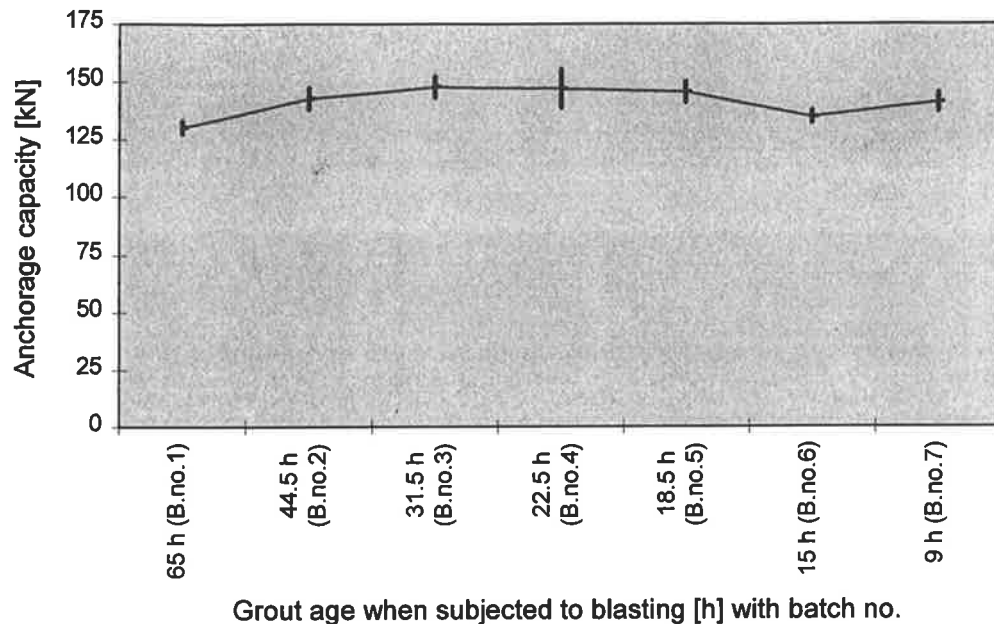


Figure 8. Anchorage capacity as a function of grout age and batch number.  
 Forankringskapasitet som funksjon av mørtelalder/batchnummer.

#### 6.4. Repeated pull-out testing

From a safety view-point it is of interest to know how fresh grout behaves after it has been subjected to large deformations (grout failure). The question is whether the grout continues the curing process and increases the anchorage capacity, or whether the grout has sustained permanent reduction in anchorage capacity. To evaluate this effect, some of the bolts from the initial pull-out tests were pull-out tested again about 110 hours after installation. Figure 9 shows the ultimate anchorage capacity when tested at the two occasions. All bolts show an increase in anchorage capacity at repeated testing. As expected, the bolts with the shortest curing time at the first test showed the greatest rate of increase in anchorage capacity. The bolts which had been allowed to cure for a longer time before being tested initially still showed a marked increase in anchorage capacity, but the rate of increase is lower. The self healing of the grout was also proved in laboratory tests performed on different grouts (Stjern, 1995).

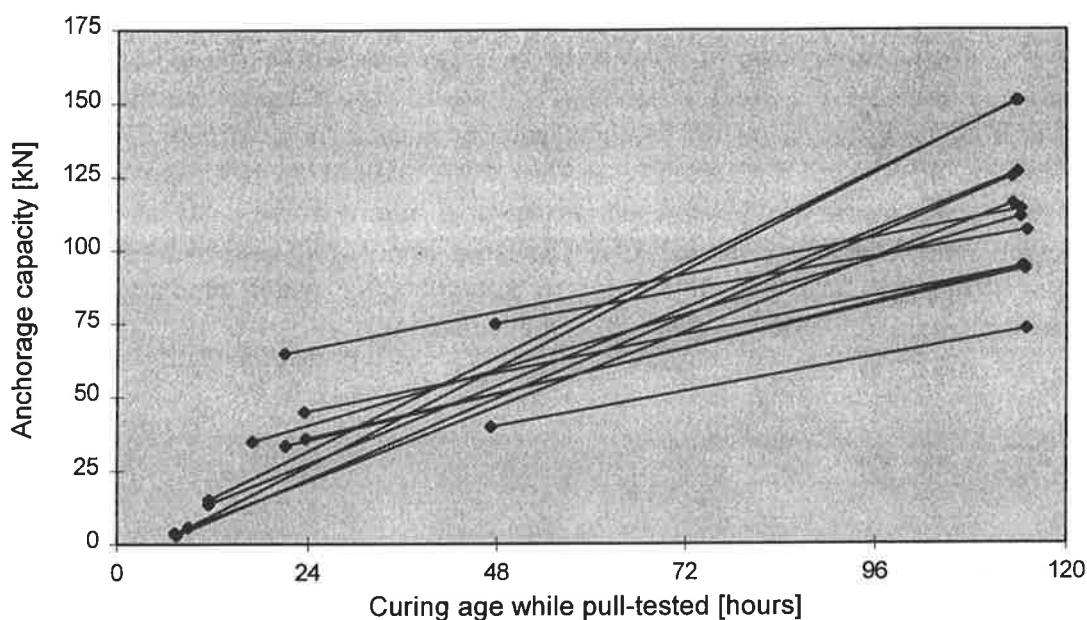


Figure 9. Results from repeated pull-out testing. Forankringskapasitet for bolter som er testet ved to ulike tidspunkt.

## 6.5. Analysis of polished thin sections of grouted bolts

In order to investigate the influence of the main blast on the grout in detail, polished sections of the core samples were prepared. All together 28 samples were gathered, one core sample from each bolt group. These bolts were standard rebar bolts not subjected to pull-out testing and embedded along the complete bolt length. The cores were prepared in the laboratory where 100 mm long samples were cut at a distance 400 - 700 mm from the rock surface. The core samples were split close to the rebar ribs prior to surface polishing. The polished section, consisting of both the grout/rock and the rebar/grout interface, was covered with a thin film of fluorescent liquid. The samples were visually inspected and photographs were taken both under normal light and UV-light, see Figure 10. Cracks and pores with a magnitude above 0.05 mm can be detected by this method.

The observation shows that the grout contains a multitude of cracks. These cracks are more or less evenly distributed, with radial cracks (mean length of 25-30 mm) radiating from the perimeter of the sample, and axial cracks along the rebar. In addition, cracks along the grout/rock interface were observed. These cracks appear to be caused by shrinkage from grout curing. It is likely that these cracks may have grown during the

preparation of the polished sections because of the continuous drying of the samples. This is not likely to occur *in situ* if water saturated condition exists in the rock. If the cracks and joints were a result of blasting, one would expect them to be more irregular and possibly oriented in a 45° inclined pattern, caused by deformations and shear failures in the grout. Furthermore, if they were caused by deformations due to vibrations, the crack would also be propagating into the rock, which never was observed. The direction of the cracks in the grout does not coincide with the foliation planes in the rock (running almost 45° to the bolt axis), which presumably are the weakest planes and thus should have guided the cracking.

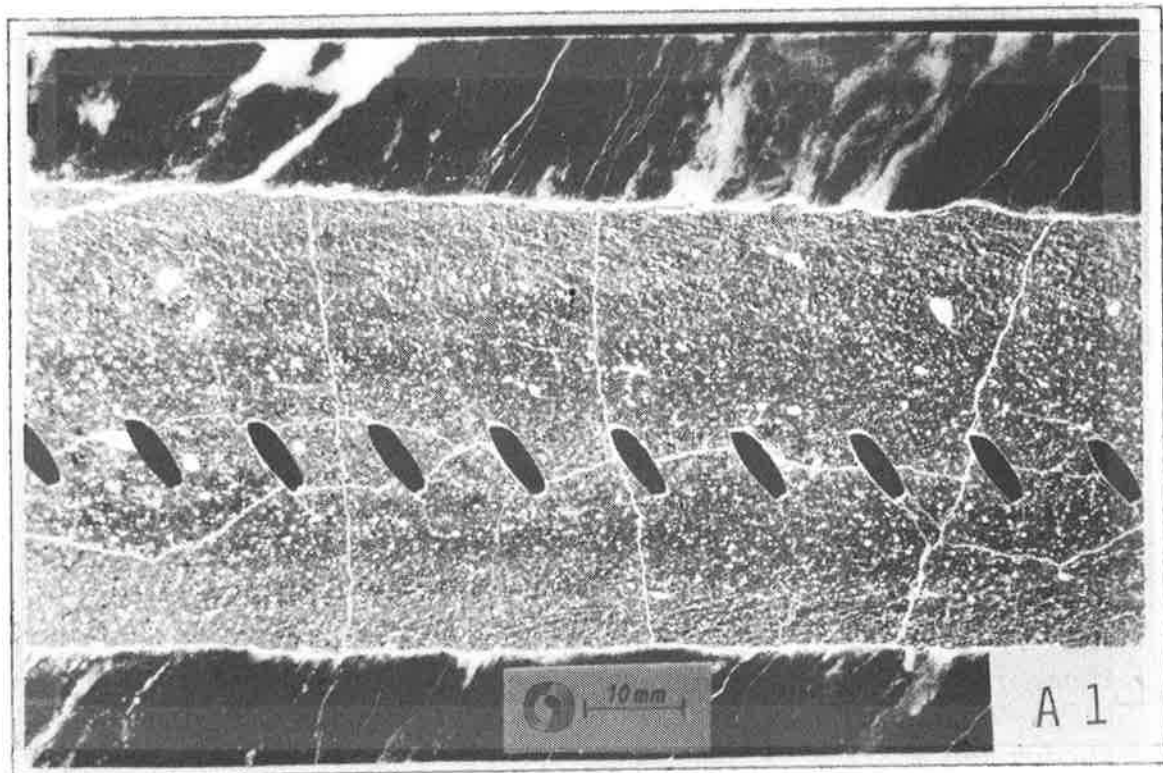


*Figure 10. Polished section of rebar bolt, grout and grout/rock interface illuminated in UV-light. Polerte planslip av kamstålbolt, mørtel og (mørtel/berg kontakt, belyst med ultraviolett lys.*

## 7. DISCUSSION

In general all tests indicate that the anchorage capacity is not detectably influenced by blast vibrations even at a PPV of 1000 mm/s. Furthermore, the age of the grout does not indicate to be sensitive to vibrations. These findings are supported by investigations

carried out by Andrieux et.al. (1994). Their study, done in the underground experimental mine in Val d'Or (Quebec, Canada), showed that induced fractures in the rock mass were generally not observed at a distance greater than 1.5 m from heavily charged (ANFO) 57 mm blast holes, corresponding to a PPV of 850-1000 mm/s.

These results are similar compared to a normal tunnelling blast charge, where the charge weight per detonator period rarely exceeds 32 kg. Furthermore, a standard tunnel blast gives lower magnitudes of vibrations, as the blast geometry will give free breakage of the rock, which not was the case for the blast described here.

Grout is often considered as a corrosion protection of rock bolts. The protection is provided by the alkaline environment that the cement creates around the steel, and also by the grout preventing free water to flow close to the bolt. However, this study of the thin sections indicate that the grout may be more permeable than previously assumed. Cracks both along the grout/rock interface and cracks running radially towards the bolt provide a mechanism for water to reach contact with the steel and thereby creating an environment more prone to corrosion damage. Previous *in situ* investigations as well as core samples taken during these experiments show that the bolt may often be placed eccentrically in the borehole. Eccentric location in the hole makes the protective layer thinner and provides a shorter flow path for the water. This gives rise to questions about the actual ability of the grout to provide corrosion protection for the installed rock bolts.

Considering the corrosion protection provided by the grout, there are several factors that must be evaluated. First, corrosion depends on the presence of water and oxygen, Svendenius (1984), which implies that there must be excess water (flow) along the bolt. Thidemann and Bruland (1991) showed that none of the protected bolts (also point anchored ones) showed minor corrosion attacks (minor pitting) when used in hydropower tunnels. Furthermore, the pH level and also the saline components of the water will play a decisive role on the corrosion rate, Andersen (1978). In addition, the alkaline conditions in the grout may dissipate over time and the corrosion protection will be reduced even further. These questions were not within the scope of this work, and it is recommended that these questions should be answered by further studies.

## 8. CONCLUSIONS

The main conclusions of this study is that the mechanical performance of fully grouted bolts subjected to close proximity blasting are not reduced by the blast vibrations. Bolts close to the blast (face) show no reduction in anchorage capacity when compared to more distantly installed bolts. This suggests that fully grouted bolts can be used close to the face for normal tunnel blasting without any damaging effect on the functioning of the rock bolt. The magnitude of the vibrations and also the charge weights per detonator

period are similar to normal tunnel blasting. However, fully grouted bolts cannot be used as primary support at the face to reinforce loose blocks or wedges, because the grouts normally need several hours for setting (cf. Figure 6).

Repulling of previously pull-out tested bolts show a healing effect of the fresh grout even when large deformations have been experienced by the bolts. This healing effect is observed for grouts reaching an age of 48 hours (temperature 8°C). Practically, this means that if grout failures have occurred because of blasts or other loadings during the initial curing period, the grout will still be maturing and regain strength.

Analyses of polished sections of bolts sampled at different distance from the blast show no fractures which were likely to have been caused by blast induced stresses. The grout was intersected by evenly distributed cracks that form a radial pattern radially and axially to the bolt shank. This cracking and the fact that the bolt shanks normally are placed somewhat eccentrically in the borehole raise some concerns about the corrosion protection ability of the grout.

## 9. SAMMENDRAG

Det er utført feltforsøk i Grong gruver for å vurdere effekten av sprengningsbelastninger på fullt innstøpte kamstålbolter. Feltforsøkene omfatter uttrekksforsøk for å vurdere forankringskapasitet på boltene samt vibrasjonsmålinger (både på boltene og på bergflaten) for å okumentere hvilke rystelsesbelastninger boltene har vært utsatt for. Videre er det foretatt kjerneboringer, hvorav det er laget planslip for å detaljstudere oppsprekkingsmønsteret i mørtelen. Mørtelen benyttet i forsøkene er laboratorietestet for i finne strekk-og trykkfasthet. Forsøkene er utført slik at både effekten av varierende vibrasjonsnivå (varierende avstand til sprengningssted) samt effekten av ulik alder på boltemørtelen kan vurderes. Det er ingen målbar reduksjon i forankringskraft for bolter montert nær stuff (3,4 m) i forhold til de som er montert lengre fra (opptil 22 m). Det er heller ingen målbar forskjell i forankringskapasitet for bolter med ulik grad av herding av mørtelen, når denne ble utsatt for sprengnings-rystelsene. Analyser av planslip viser ingen forskjell i sprekkesystem eller sprekketetthet hverken ved varierende rystelsesnivå, eller ulik mørtelalder da denne ble eksponert for sprengningsrystelser. Ettertesting av bolter, som før har vært testet for å finne forankringskapasitet, viser at mørtelen har en "legende" evne. Mørtelen fortsetter herdingen etter første brudd, noe som gir økt forankringskapasitet i forhold til første gang den ble testet. Ingen av testene har avdekket noen form for svekkelse av mørtelen og det kan dermed konkluderes med *at fullt innstøpte bolter kan benyttes på eller nært inntil stuff ved normal tunneldrift.*

## ACKNOWLEDGEMENTS

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## INGJUTNA BULTARS BESTÄNDIGHET, ERFARENHETER FRÅN VATTENFALLS ANLÄGGNINGAR INFÖR RINGEN PROJEKTET, STOCKHOLM.

*Grouted rockbolts, their resistance, experiences from Vattenfall hydropower stations with aspect to the Ringen projekt, Stockholm.*

Ulrika Hamberg, Vägverket Region Stockholm, Solna

Per Erik Söder, Vattenfall Hydropower, Ludvika

### INLEDNING

Dennisöverenskommelsen omfattar inom Vägverkets ansvarsområde vägsystemen Ringen och Yttre Tvärleden. Ringen innebär att ca 12 km väg ska förläggas i tunnel runt Stockholms innerstadsområde. Den totala tunnellängden blir drygt 40 km med hänsyn till att Ringen har dubbla tunnelrör med enkelriktad trafik och ett stort antal underjordiska trafikplatser med på- och avfartsramper. Även Yttre Tvärleden innehåller omfattande vägsträckor i tunnlar runt några av de västra och norra förorterna.

Konsekvenserna vid felbeslut gällande till exempel lösningar av konstruktionsdetaljer kan bli mycket kostsamma både i bygg- och i driftskedet till följd av de långa tunnelsträckorna. För att så långt som möjligt undvika detta har Vägverket Regions Stockholm, projekt Ringen och Yttre Tvärleden intentionen att utreda och i viss mån testa lösningar där tveksamhet föreligger för att finna goda sådana.

I Vägverkets "Allmän teknisk beskrivning - Tunnel 95" anges att det bärande huvudsystemet i tunnlar ska dimensioneras för en livslängd på 80 - 120 år.

Hur ska en bultkonstruktion se ut för att hålla i 120 år. I Sverige har berganläggningar traditionellt förstärkts med injutna, icke rostskyddsbehandlade stålbultar sedan 1940-talet. I norska anvisningar krävs normalt varmförzinkade bultar som, då tunnlar passerar under havsbotten, dessutom skall beläggas med ett lager epoxi. I Sverige ifrågasätts detta starkt med hänsyn till de erfarenheter man har av berganläggningar i landet.

Flera studier har gjorts av enskilda bultars beständighet där de studerade bultarna sällan varit monterade i mer än 15 år. Däremot finns det i stort sett inga studier gjorda på berganläggningars beständighet med avseende på bultarna.

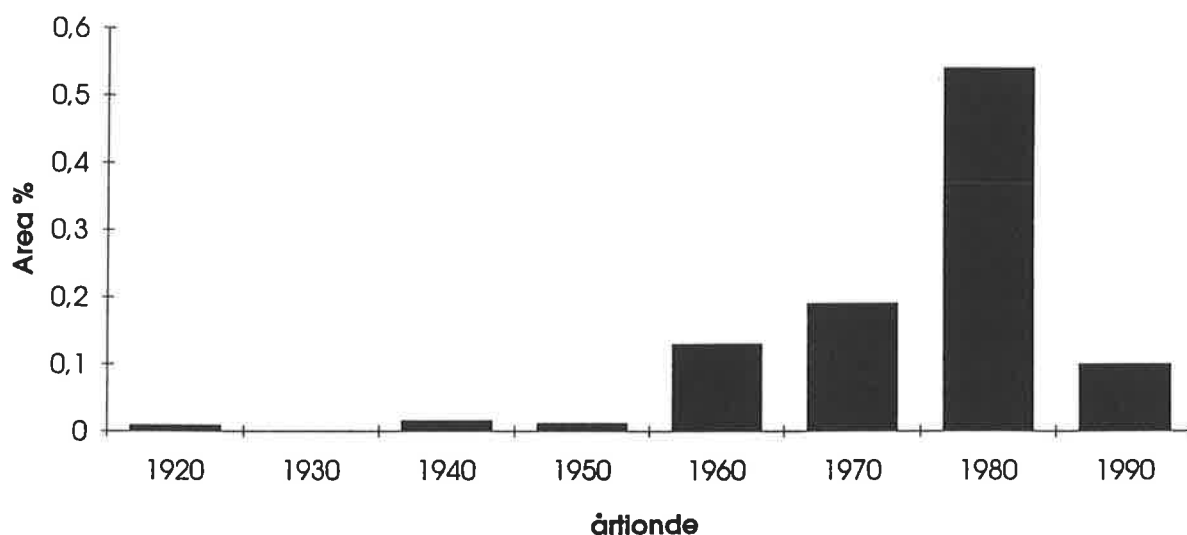
Då Vägverket endast äger ett fåtal, relativt nya anläggningar, finns ej tillräckligt eget underlag att grunda en relevant utredning på och därför har Vägverket vänt sig till en stor berganläggningsägare, Vattenfall, för att utnyttja deras erfarenheter inför valet av bultningsfilosofi.

## VATTENFALL

Vattenfall är ägare till ett stort antal tunnlar av vilka den första togs i bruk år 1907. Största utbyggnaden av stora tunnlar ligger på sjuttioalet. Stationer byggda före 1970 står för mindre än 15 % av takytan i tunnlar inom Vattenfalls anläggningar, vilket motsvarar omkring 200 000 m<sup>2</sup> takyta, totalt finns ca 1500 000 m<sup>2</sup>. I figur 1 visas åldersfördelning i 10 års intervall för takytor i vattenkraftsanläggningar.

Figur 1. Nya takytor i vattenkraftsanläggningar i 10 års intervall.

*New roofarea in hydropower plants in 10-year intervals.*



Inom Vattenfall har använts ingjutna bergbultar av armeringsjärn från i stort sett början av 40-talet. De bultar som finns installerade är: (se även figur 2)

- Injektobultar Bultar bestående av armeringsjärn som fästs i botten, med kilar alt. kilats fast vid borrhålsmynningen. Injektering har sedan skett från hålmynning. Användes fram till 1958.
- Perfobultar Bultar bestående av armeringsjärn som sätts i cementbruk som införts i hålen i perforerat cementfyllt plåtrör. Några få bultar satta från 1959, främst som installationsförankringar.
- SN - bultar Bultar bestående av armeringsjärn som införs i cementbruk som injicerats i borrhålen med slang. Relativt styvt bruk använts vct 0.28-0.30 . Bultar av denna typ har satts från 1952.  
(SN = Stornorrfors)

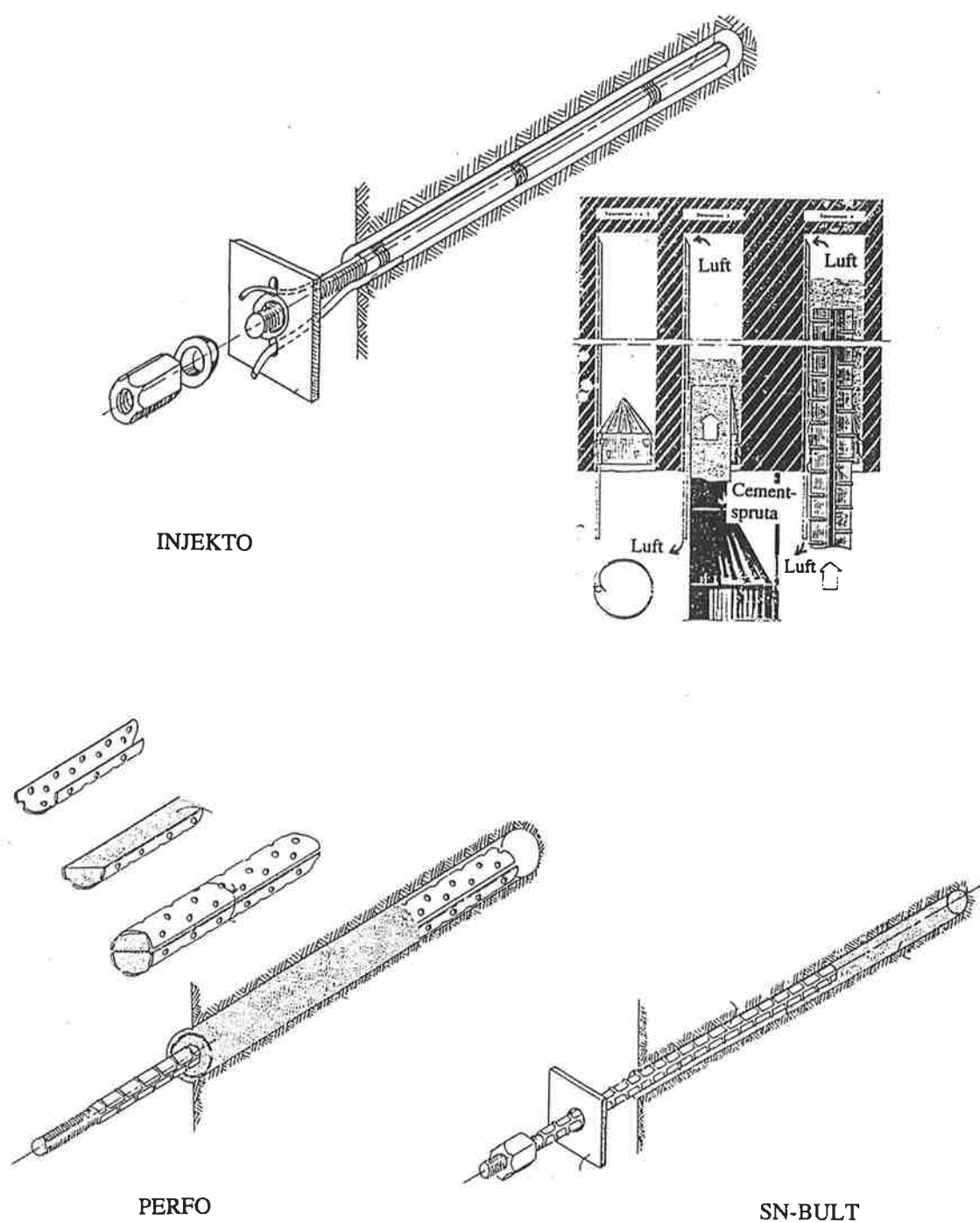
Detta innebär att de flesta bergbultar som finns som permanenta bergbultar inom Vattenfallsanläggningar är av SN-typ. I figur 3 ges takarea som förstärkts med de olika bulttyperna.

Bultsättningen har varit "normerad" genom att bultsättningsanvisningar utfärdats med ca 10 års mellanrum sedan 1956. De ingjutna bultarna har i normalfallet ej varit rostskyddsbehandlade.

Korrosionsbenägenheten hos utstickande bultdelar har beaktats i de fall då de utgör infästningar för andra konstruktionsdetaljer såsom undertak, kabelstegar och liknande. Sådana delar har ibland utförts rostskyddsbehandlade. Övriga utstickande bultändar har utförts utan rostskydd men ibland ingjutits i sprutbetong eller betong.

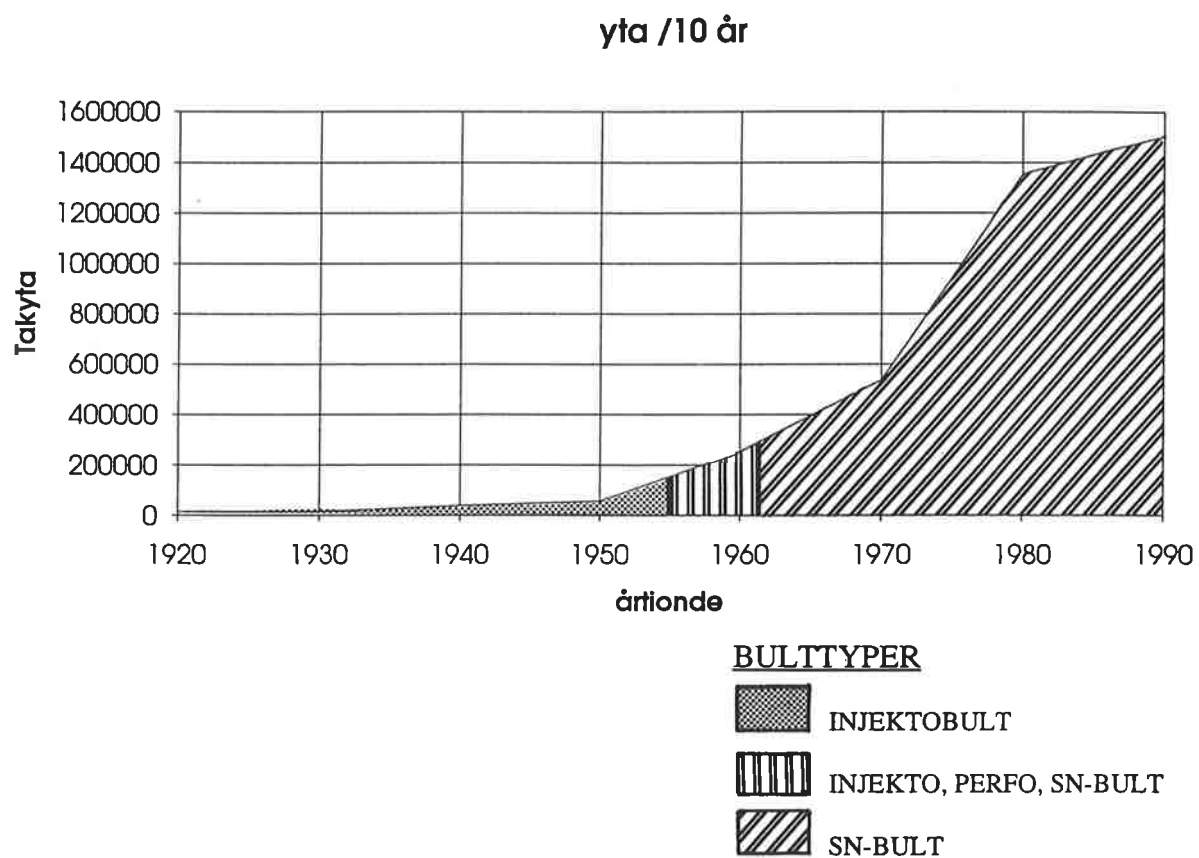
Figur 2. Olika typer av bergbultar använda för permanentsäkring inom Vattenfall anläggningar

*Various rockbolt types utilized for permanent support in Vattenfall.*



Figur 3. Typer av bergbultar och tunnelarea som funktion av produktionstidpunkt.

*Rock bolt types and tunnelarea as function of time of construction.*



## **BERGFÖRSTÄRKNINGSTYPER**

Förstärkningarna bestående av ingjutna bergbultar av olika typer som utförts från senare hälften av 40-talet. I dåligt/mycket dåligt berg har i stort sett endast platsgjutna konstruktioner - typ betongbågar, stöd, eller innertak utförts.

Tidigt sattes injektobultar och perfobultar för att efter Stornorrfors ersättas med bultar satta i tjock cementvälling injicerat i borrhål. Denna metod utvecklades från mitten av 50-talet och modifierades 1961 genom övergång till ren cementpasta istället för den tidigare använda cement/sand blandningen. I huvudsak har alltså samma metod för bultsättning använts från 1956 till fram i dag. Vilket innebär att omkring 80 - 90 % av taktyor inom Vattenfallsanläggningar förstärkts med denna bulttyp.

## **BERGFÖRSTÄRKNINGSMÄNGDER**

Bultmängden för maskinstationer är i stort sett 3 - 5 gånger större än vad Q-index scheman ger idag. Tillåtna laster dimensionerades mot en säkerhetsfaktor av 1.7, förankringslängd i fast berg anges till > 40 ggr bultdiametern och håldimension skall vara 15 mm större än bultens diameter.

Injektobultar, perfobultar och SN-bultar förordades 1956 som möjliga för permanentförstärkning. Bultsättning med SN-bultar nämns bara i vattenfallanvisningarna för bultsättning av 1961.

Cementingjutningar har provats med olika typer och blandningar av tillsatsmedel och sand. Efter 1961 så ger bultsättningsanvisningarna att bultar skall sättas i cementpasta med lågt vct 0.28-0.30.

## **BESIKTNINGAR OCH UNDERHÅLL**

Besiktningar och underhåll av bergytorna sker regelbundet för de undermarksförlagda kraftstationerna. Bergunderhållsområden finns älvvis. Underhållsområdena har upprättade underhållsplaner och underhållsscheman för varje station.

För besiktning och skrotning upprättas rapporter. I samband med skrotningsarbeten noteras nedskrotade massor för varje stationsdel för sig i separat skrotningsrapport.

För nybyggda stationer finns ett underhållsschema och driftkort med noteringar om geologi och bergförstärkning samt vad som skall kontrolleras i samband med bergbesiktning. Se exempel på driftkort och underhållsschema från Klippens kraftstation i figur 4.

Figur 4 Exempel på underhållsschema för en region och driftkort från Maskinsalen i Klippens kraftstation.

*Maintenance scheme for two rivers and sevicecard for the machinehall in Klippens hydropower plant.*

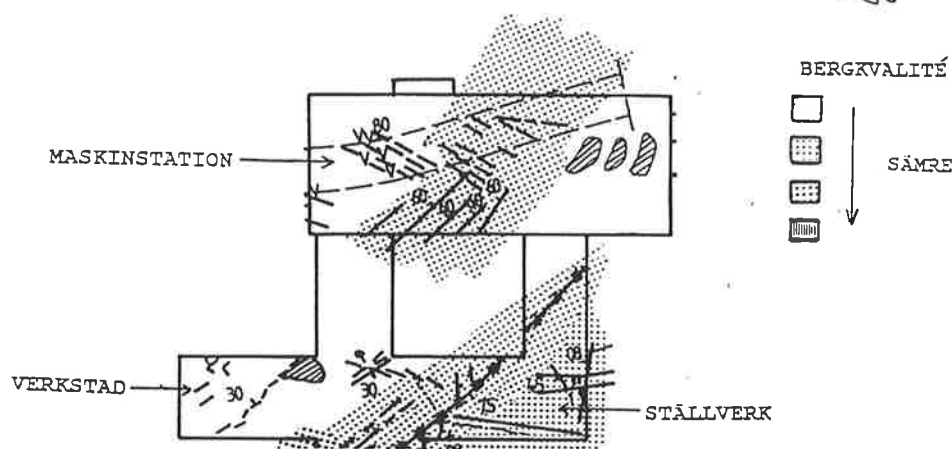
Sammanställning av planerade bergbesiktningar och bergskrotningar vid kraftstationerna efter Indals- och Ångermanälven

B = Bergbesiktning  
S = Bergskrotning

| Anläggning  | Bergytor<br>m <sup>2</sup> | 19 92 |   | 19 93 |   | 19 94 |   | 19 95 |   | 19 96 |   | 19 97 |   | 19 98 |   |
|-------------|----------------------------|-------|---|-------|---|-------|---|-------|---|-------|---|-------|---|-------|---|
|             |                            | B     | S | B     | S | B     | S | B     | S | B     | S | B     | S | B     | S |
| Stadsforsen | 1.600                      |       |   |       |   |       |   | X     |   |       | X |       |   |       |   |
| Midskog     | 7.100                      | X     |   |       | X |       |   |       |   |       |   | X     |   |       | X |
| Sillre      | 2.200                      |       | X |       |   |       |   |       |   | X     |   |       | X |       |   |
| Hölle       | 3.000                      |       |   | X     |   |       | X |       |   |       |   |       |   | X     |   |
| Näverede    | 200                        | X     |   |       | X |       |   |       |   |       |   | X     |   |       | X |
| Stugun      | 100                        | X     |   |       | X |       |   |       |   |       |   | X     |   |       | X |
| Järkvissle  | 1.400                      | X     |   |       | X |       |   |       |   |       |   | X     |   |       | X |
| Bergeforsen | 600                        | X     |   |       | X |       |   |       |   |       |   | X     |   |       | X |
| Torpshammar | 10.000                     |       |   |       |   | X     |   |       | X |       |   |       |   |       |   |
| Forsmo      | 9.200                      |       | X |       |   |       |   |       |   | X     |   |       | X |       |   |
| Nämforsen   | 1.000                      |       |   |       |   | X     |   |       | X |       |   |       |   |       |   |
| Kilforsen   | 25.100                     |       |   |       |   | X     |   |       | X |       |   |       |   |       |   |
| Lasele      | 21.500                     |       |   |       |   | X     |   |       | X |       |   |       |   |       |   |
| Långbjörn   | 5.500                      |       |   |       |   | X     |   |       | X |       |   |       |   |       |   |
| Stalon      | 9.200                      |       |   |       |   |       |   | X     |   |       | X |       |   |       |   |

4 nov 1992

Bernhard Persson



#### 121 MASKINSTATION

Bergarterna inom området utgörs i huvudsak av glimmergnejs med en del granitinslag. Berget i de centrala delarna av stationen är blockigt med två stycken sprickgrupper samt skifferighet i själva bergarten. Vattendropp främst vid infarten till stationen. Blockutfall i taket till följd av ogynnsamma sprickriktningar. Lokalt, Q-värden mitt i taket ned till ungefär 10. Särskild kontroll i område C och D (sprickzon som tvärrar stationen).

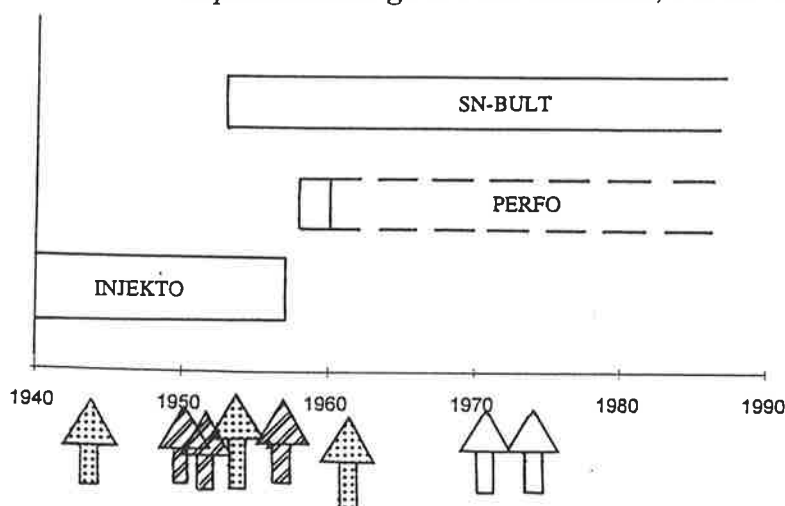
## VATTENFALLS ERFARENHETER

### Bultars och ingjutningars tillstånd i förhållande till miljö och ålder.

I de enstaka fall ras och reparationer förekommit i förstärkta områden, har dessa orsakats av bland annat försämrad bärförmåga hos ingående bultar, som i sin tur beror på att bultar rostat av, urlakning av cementingjutning, felaktigheter i samband med installation av bultar och felaktigheter i materialval. Andra orsaker kan vara geologiskt betingade såsom vittring eller ej tillräcklig förstärkningsnivå med hänsyn till geologiska förhållanden. Den vanligaste orsaken till bekymmer med bultförstärkta partier torde kunna hänföras till dålig ingjutning av bult och då främst vattendropp. I figur 5 har angivits större bergproblem och ras som inträffat kraftstationer under ca 40 år. Problemen har indelats i tre grupper: dåliga bultar, dåligt berg och tidsberoende omvandling av berg.

Figur 5. Större bergproblem och ras som inträffat i olika kraftstationer under ca 40 år, grupperade på, dåliga bultar, dåligt berg eller tidsberoende omvandling av berget, såsom uttorkning.

*Major rockproblems and failures occurring in powerplants during abt. 40 years, grouped according to bad bolts, bad rock och time dependent change in rock conditions, such as by drying.*



Bulttyp, installationstidpunkt och bergproblem de senaste åren.



## **Korrosion**

I samband med tillbyggnad av Harsprånget 1980 testades korrosionen på utstickande bultändar i den gamla utloppstunneln. Tunneln var då ca 30 år. En genomsnittlig avfrätning av 0.02-0.03 mm/år noterades, maximal gropfrätning var större än 0.1-0.2 mm/år. Korrosionen var störst vid bergytan, vid gränsen mellan det syrerikare strömmande tunnelvattnet och det syrefattigare vattnet i bulthålen.

En allmän erfarenhet är att utstickande bultändar rostar, vilket lokalt i hålmynningarna har lett till en viss "rostsprängning" av cementbruket någon cm.

## **Uppvärmda utrymmen**

Uppvärmning av bergyta i anslutning till maskinsal och transformersal till omkring +20° medför att en uttorkning av bergytan sker. I känsliga bergarter kan detta därför innebära en förändring i lasterna på bergförstärkningen, och ge krav på kompletterande förstärkningar. Detta har bla. erfarits i Gejmåns kraftstation byggd i fjällskiffer berggrund.

## **Kontroll av bultar**

Kontroll av bultar börjar redan på projekteringsstadiet genom att i stort sett endast en sorts bult, med specificerad stål och ingjutningskvalité respektive bultsättningsförfarande godkänts som permanent för bergförstärkningsändamål. Detta har reglerats i Vattenfall- anvisningar. Kontroll på byggarbetsplats har sedan utförts av kontrollant.

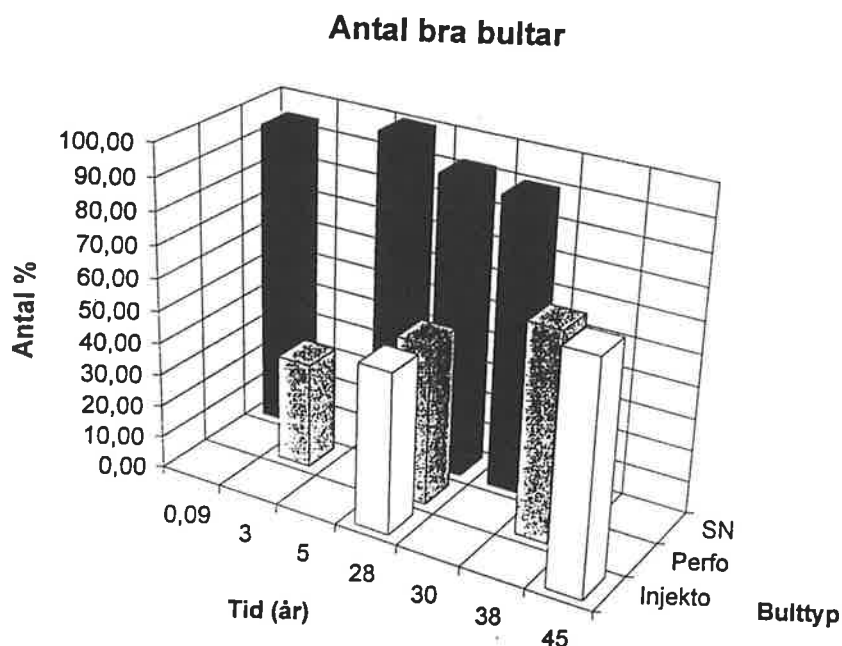
Efterkontroll av bergrum och bergförstärkning utförs genom regelbundet återkommande bergbesiktningar. Bergbultskontrollen utförs okulärt eller genom tester som Boltometer eller utdragsförsök.

Boltometer testerna är föranledda av besiktningsresultat som vattendropp ur bulthål, bomma partier som bärs av bult eller tveksamhet angående bulttypens beständighet baserat på tester av just den satta bultens typ och årtal. Som en ledning för bestämmande av detta tjänar de gamla anvisningarna och dåtidens bultmaterial och installationsförfarande.

Boltometertester har utförts på bultar av olika ålder och utförande. De bultar som testats har varit Injektobultar, Perfobultar och SN-bultar. Testningarna har i huvudsak utförts i maskinsalar och angränsande tunnelutrymmen. Testningarna har givit resultat enligt figur 6. Boltometertester gav att SN-bultar är bra till stor del, perfo- och injektobultar mer tveksamma. Inga förändringar pga olika ålder hos de testade bultarna kunde iakttas.

Figur 6. Antal bra bultar i boltometertester (A och B bultar har klassats som bra, C och D som dåliga) för tre typer av bergförankringar. Bultar i tillfartstunnlar o och maskinsalstak är de som testats.

*Amount of good bolts classed in boltometertests ( A and B bolts have been classed as good, C and D as bad) for three types of bolts. Bolts in access tunnels and machinehalls are tested.*



Inga större ras eller stennedfall har inträffat som enbart kan förklaras bero på försämring hos bergförstärkningarna. Detta beror troligen på tre faktorer

- Systematisk bergbesiktning och underhåll utförs enligt ett rullande schema med intervall ca 4 - 5 år. Vattenvägar besiktas och åtgärdas ofta i samband med större maskinellt underhåll i stationerna. Intervallet för detta är ungefär 10 - 40 år.
- Hög förstärkningsnivå i förhållande till bergkvalitén. Detta påstående gäller stationsutrymmen och ej vattenvägar.
- Val av beständig förstärkning.

En stor del av bergytorna inom Vattenfall anläggningarna har utförts utan sprutbetong vilket till viss del förklarar skrotningsbehovet.

## MILJÖFÖRHÅLLANDEN I STOCKHOLMSTRAKTEN OCH DESS BETYDELSE FÖR BERGBULTAR.

Bergarterna för de flesta Vattenfall anläggningarna är till övervägande del gnejser och graniter, huvudbergarterna inom Stockholmsområdet kan troligen förväntas ha likvärdiga egenskaper som bergarter beträffande avsaknaden av vittringsbenägenhet och vara mycket långtidsstabila. Infästning av bultar i bra berg torde därför vara fullt jämförbar.

Risk finns för kontakt med salt grundvatten i Stockholmsområdet, vilket ger en ökad korrosions- risk jämfört med Vattenfalls stationer. Troligt är att bergbults livslängd i Stockholm därför blir kortare än för Vattenfalls norrlandsstationer.

### SLUTSATSER

Värdering och jämförelse med Ringen projektet - ur förstärkningssynpunkt - görs mycket översiktligt i schemat nedan genom tilldelning av + (bättre) eller - (sämre) relativt Vattenfalls anläggningar.

|                               | Vattenfall     | Ringen                     | värdering |
|-------------------------------|----------------|----------------------------|-----------|
| <b>Geologi</b>                |                |                            |           |
| Bergtyper                     | granit o gnejs | sedimentgnejs              | ±         |
| Dåliga zoner                  | undvikits      | genomkorsas                | - -       |
| Grundvatten                   | bra kvalité    | sämre kvalité              | -         |
| <b>Driftskede</b>             |                |                            |           |
| Vibrationer                   | maskiner       | trafiklast                 | ±         |
| Uttorkning                    | uttorkning     | innertak                   | +         |
| <b>Val av bult</b>            | vanliga KS     | vanliga Ks<br>ev roskskydd | ±<br>(+)= |
| <b>Design</b>                 |                |                            |           |
| Förstärkingsmängd             | konservativ    | branchpraxis               | - -       |
| Förstärkningstyper            | bult           | bult + sprutbtg            | + + +     |
| <b>Utförande</b>              |                |                            |           |
|                               | manuellt       | manuellt                   | ±         |
|                               |                | maskinellt                 | -         |
| <b>utförande<br/>kontroll</b> | rel. god       | god                        | ±         |

Slutsatsen av ovanstående exposé är att sämre förhållanden beträffande geologi - pga riktningskrav, troligen större exponering för krosszoner och liknande för vägtunnlarna kan förväntas gentemot Vattenfalls anläggningar.

Bultsättningen blir mindre i vägtunnlar genom att numera rekommenderas mindre mängd bult samt att förstärkningen kompletteras med sprutbetong.

Ytförstärkningen i form av sprutbetong innebär troligen att underhållskraven blir lägre för Ringen än Vattenfalls underhålls krav, eftersom miljöbelastningen på bergytan blir lägre. (tex. uttorkning, vibrationsavlossning etc...)

## **DISKUSSION**

Kraftstationerna har byggts ut i etapper och det ger upphov till olika typer av tekniska lösningar var och en avspeglade sin tids tekniknivå och underhålls filosofi, ibland inom samma anläggning.

Bergbulten har utvecklats från en relativt komplex sak med injekteringsrör och avluftningsrör till ett standardiserat kamstål satt i bruksfyllt borrhål vilket minskar risken för fel i samband med bultsättning. Intensivt utvecklingsarbete bedrevs i mitten av 50-talet beträffande bultutformning. Baserat på vunna erfarenheter har sedan Vattenfall utfärdat anvisningar för bergbultning med ungefär 10-års intervall från 1956.

### **Livslängd hos förstärkningssystemen**

Det faktum att bergunderhållsgrupperna besiktas och skrotas samt utför tilläggsförstärkningar innebär att Vattenfall i stort ej har någon undermarksförlagd station där tilläggsförstärkningar inte har utförts inom en tidsperiod av 10 - 20 år. Livslängden hos ett bultförstärkningssystem bestående av berg/bult/omgivningen är trots detta okänd. I vattenvägar finns tunnlar där ingen besiktning eller tilläggsförstärkning utförts inom intervallet 10 - 50 år. Tunnlarna är trots det intakta och uppvisar mycket sällan några fallförluster som skulle bli fallet om förändringar i tunnelarean inträffar.

En högre fel andel hos gamla injekter och perfobultar förekommer. Felen är troligen fördelade mellan rena installationsfel - komplicerad metod etc.. och fel hos borrhål beroende på förekomst av vatten.

Upprättandet av installationsanvisningar för bergbult har troligen en mycket viktig roll i att bultarna är så bra som de är.

Då fel förekommer hos bultade partier beroende på att bultarna installerats slarvigt respektive av att fel material eller utförande valts( systematiskt fel) - medger vattenfalls materialet slutsatsen att bergbesiktning med skrotning, respektive kontroll av bergbult är viktigt att kunna utföra.

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Stockholm. J. Martna 1975.

### **Internt**

Vattenfall meddelanden och rapporter mellan åren 1927 - 1991 angående bergförstärkningar och bergbultar.

## **SUMMARY**

In Sweden, non-corrosive protected grouted dowels have traditionally been used since the 1940's. Based on experiences gained from a number of underground hydropower plants within Vattenfall, the long-term function of the dowels is discussed. The hydro power plants owned by the Vattenfall involve about 1.5M square metres of tunnel roof of which 15% of the total area belong to plants built before 1970. Normally, Injekto, Perfo and grouted dowel bolts have been used, and 25 mm diameter ribbed bars are the most common. Simple grouted dowels without corrosion protection have mainly been used from 1958 and onwards. Inspection of waterways and other underground structures have shown that only minor rock falls have occurred during the years due to malfunctioning bolts. The water tunnels included in the Swedish hydro schemes are without exceptions unlined and shotcrete is used only to a small extent in power houses and other underground installations. However, inspections and systematic maintenance are being made on a 4 to 5 year basis involving scaling and complementary support, such as selective bolting, replacement of malfunctioning bolts and shotcreting. Grouted dowels seem to be more reliable in comparison with the types mentioned above.



## **DIFFICULTIES RELATED TO SQUEEZING ROCK IN THE URI-PROJECT, INDIA**

Problem i samband med krypande berg i Uri-projektet, Indien

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### **ABSTRACT**

The tailrace tunnel at the Uri HPP, Kashmir, India, has been successfully excavated. The tunnel passes through the Panjal thrust which is one of the major thrusts in this part of the Himalayas and also through a major fault zone. The highly metamorphosed rock mass was weak and crushed due to the tectonic activities. The overburden of 400-500 m in combination with the soft rock caused short term stability problems as well as continuous squeezing. The method of excavation as well as that of support had to be designed for the extreme conditions within the soft rock. In order to evaluate how the ongoing squeezing would affect the cast concrete lining, two test linings were cast and instrumented for measurement of deformation. The evaluation gave a good picture of the distribution of stresses along the lining.

### **1 INTRODUCTION**

The Uri Hydroelectric Power Project is situated in the State of Jammu and Kashmir in north-western India. The power plant is supplied with water from the Jhelum River which has its springs in Kashmir and flows through the Kashmir Valley into Pakistan, where it becomes a tributary to the Indus River. The project is designed as a run-of-the-river scheme. The water is diverted from the river and lead from the head regulator and desilting basins into an 11 km long headrace tunnel and two 210 m high pressure shafts. After passing the four turbines in the machinery hall the water is lead through the draft tubes and the 2 km long tailrace tunnel back into the river.

The project is owned by the National Hydroelectric Power Corporation Ltd, NHPC. The construction of this hydro power project is undertaken on a turn-key basis by a consortium involving Skanska International Civil Engineering AB and NCC International AB for the civil works, SWECO AB as the designer, ABB Generation AB for the electrical works and Kvaerner Boving Ltd for the mechanical works. Skanska and NCC have formed a subsidiary company for their joint undertaking called Uri Civil Contractor AB. Skanska is the leader of the consortium. The work was started in 1990 and the project is scheduled to be completed in May 1997. A general description of the project has been given by Heiner et al. 1993.

Several types of tunnel instability have occurred. Instability in connection with the excavation of the machinery hall and buckling phenomena in tunnels aligned along the foliation planes have been discussed in Brantmark et al. 1995. In some reaches of the headrace tunnel the vertical rock cover exceeds 850 m. In this tunnel the same kind of stress related problems were observed as is described in the above paper.

Quite different types of instability, which are the subject of the present paper, were met with in the tailrace tunnel. These include weak or extremely weak rock under high stresses, squeezing and both short time and long time stability problems.

## 2 GEOLOGY

In the underground works at Uri a both stratigraphically and lithologically wide range of rocks has been encountered, Table 1, Fig. 2.1.

Table 1. Rock types encountered in the underground works at Uri

| Age                             | Formation  | Lithology                  |
|---------------------------------|------------|----------------------------|
| Tertiary (Eocene)               | Nummulitic | Shale, limestone layers    |
| Permo-Carboniferous             | Panjal     | Meta-volcanics             |
| Lower Paleozoic to Pre-Cambrian | Tanawal    | Quartzitic schists         |
| Pre-Cambrian                    | Salkhala   | Schists, graphitic schists |

The geology of the Uri Project is elucidated in Heiner et al. 1993, together with adapted rock mass classification and stipulated initial support.

The upstream half of the totally 10.7 km long headrace tunnel was excavated in the rocks of the Tanawal series, consisting predominantly of quartzitic schists.

The downstream half of the headrace tunnel was excavated in the so-called Panjal volcanics, comprising greenstones with a generally well developed schistosity. They are regarded to be metamorphic andesitic or basaltic lavas. Sills of a similar appearance occur also intercalated in the Tanawal schists.

The machinery hall with appurtenant rock caverns and tunnels are located in Panjal volcanics. The problems encountered during the excavation of the powerhouse complex are discussed in Brantmark et al. 1995.

In the 2027 m long tailrace tunnel a somewhat more complicated geology, rather different from the expected one, was encountered, Table 2, Fig. 2.2. It can be observed from Fig. 2.2, that the rock cover is between some 250 and 500 m. In the tunnel reaches where squeezing has been observed it is more than 300 m.

Because of adverse natural conditions - inaccessible terrain, soil cover and crushed rock - none of the attempted drillholes reached the tailrace tunnel alignment. The geological expectations were based on surface mapping and some test adits only.

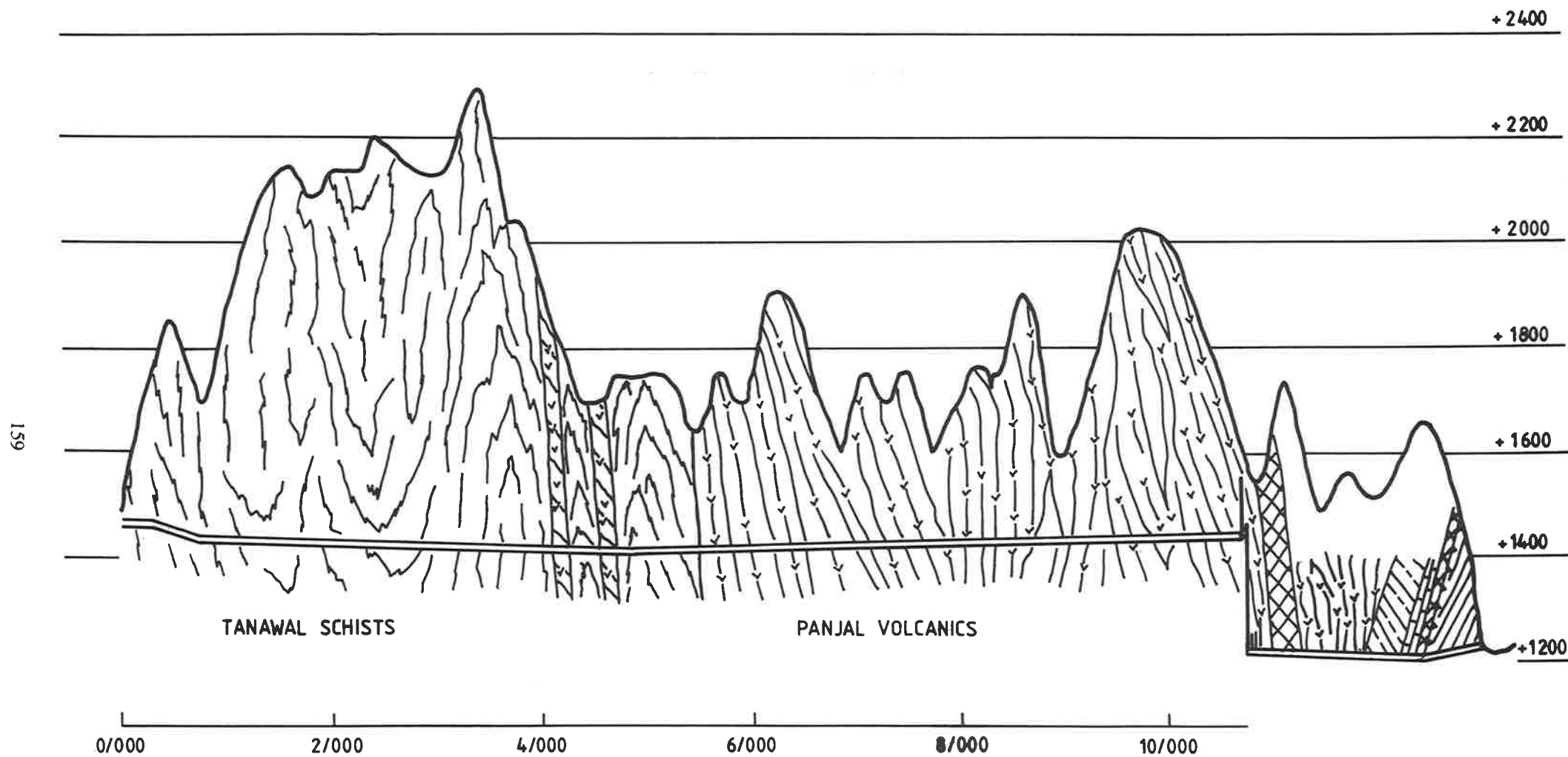


Figure 2.1 Longitudinal geological section of the Uri Project. The geology of the Tailrace tunnel is explained in Fig 2.2  
 Geologisk längdsektion av Uri-projektet. Utloppstunnelns geologi förklaras i Fig 2.2

Table 2. Geological sequence in the tailrace tunnel (from upstream).

| Lithology/stratigraphy                 | Tectonics             | Length, m |
|----------------------------------------|-----------------------|-----------|
| Panjal volcanics                       |                       | 30        |
| Sheared talc-chlorite-graphite schists | Fault zone            | 292       |
| Metavolcanics, strata of schists       | Several healed faults | 865       |
| Schist                                 |                       | 203       |
| Marble, layers of graphite schist      |                       | 51        |
| Schist, shale and graphite schist      | Panjal thrust         | 55        |
| Eocene shale with limestone layers     |                       | 531       |

Upstream of the Panjal thrust, essentially Panjal volcanics were expected to be met with in the tunnel. Because of surface evidence, these strata were expected to be interrupted by a major fault zone, containing i.a. crushed Eocene shale and a repetition of the Panjal thrust. However, this did not occur.

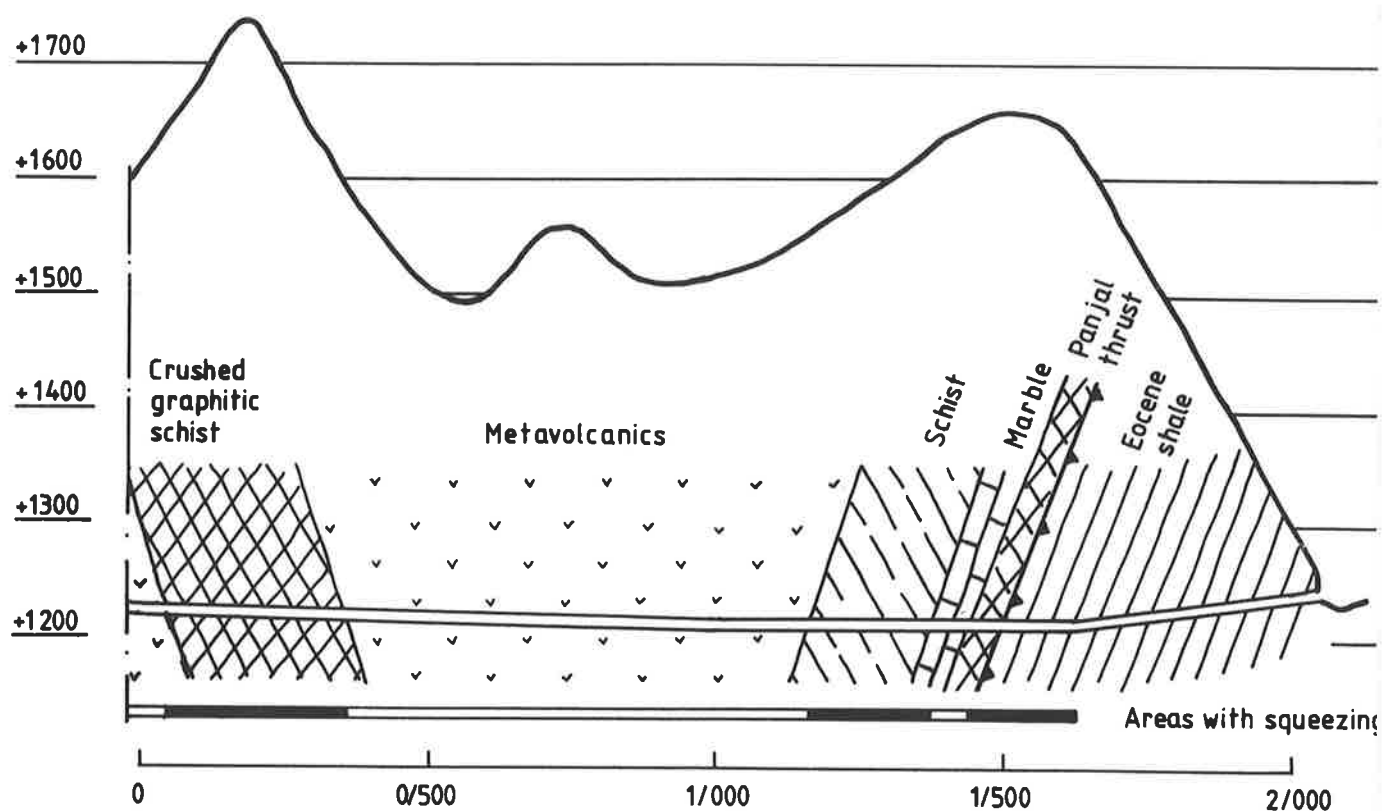


Figure 2.2 Longitudinal geological section of the tailrace tunnel.  
Geologisk längdsektion av utloppstunneln

In the upstream end of the tailrace tunnel, after about 30 m of Panjal volcanics, a some 290 m wide zone of sheared and crushed rock consisting of a blend of mica, talc, chlorite and graphite schists and phyllites with some limestone bands was encountered. A notable amount of clayey gouge was also present. The rocks in this major fault zone had a very low strength and poor engineering properties. Since the rock cover was 400 - 500 m, a troublesome squeezing of the tunnel took place.

Downstream from this fault zone, schistose meta-volcanics with strata of schists were encountered. Several minor faults, some of them quite healed, were met with. After about 860 m the rock changed to partly graphitic schists for a length of 200 m. A massive marble bank and the Panjal thrust concluded this series of strata.

The Panjal thrust causes the definite change of the tunnelling medium in the tailrace tunnel from the Palaeozoic and Pre-Cambrian meta-volcanics and schists to Eocene shale. The strike of the Panjal thrust is N60W with a north-easterly dip of some 45°. The thickness of the thrust, represented by crushed and intensely folded schist and shale, is only some 50-55 m. However, as discussed above, highly disturbed rock zones have been met with upstream of it, indicating comprehensive tectonic movements in this reach.

It has not been possible to clarify the tectonic events which in this tunnel reach led to a profound difference between the surface geology and that of the tunnel alignment.

It can be suggested, that the graphite-rich schists, unique for the Project, represent the Pre-Cambrian Salkhala formation. North of the project area, the Salkhalas are represented by such rocks.

### **3 EXCAVATION OF THE REACH CH. 0/038 - 0/322**

#### **3.1 Method of excavation**

The first 300 m of the tailrace tunnel provided challenging rock conditions. In this reach the rock was crushed and sheared with a short stand-up time. The uniaxial strength was estimated to be below 1 MPa, the overburden 400 - 500 m and the contents of graphite and chloritic clay were high. In most of this reach the rock could have been excavated by excavator only, however drill and blast was used in order to maintain the profile (Fig. 3.1).

Due to the poor condition of the rock mass, special precautions had to be taken during the excavation. The poor rock was first encountered at chainage 0/038. Here the excavation was successfully carried out with heading and bench in combination with spiling. This method of excavation worked well for the first 90 m. However, when the tunnel reached chainage 0/128, the quality of the rock deteriorated further and instability was observed in the face. In order to improve the stability, the heading had to be divided. Hence from chainage 0/128 to 0/322 the tunnel was excavated with divided heading (multiple drift) (Fig 3.2).



Figure 3.1 The rock was soft enough to be excavated by excavator. View of the bench.

Bergmassan var tillräckligt mjuk för att grävas ut. Vy mot pallen.

When entering rock conditions which require a divided, heading several considerations have to be made regarding the method of excavation.

- How many headings are needed?
- How far ahead of the side drifts should the pilot drift be?
- Length of rounds?
- Distance between face and bench?

Every additional heading requires additional time and causes additional joints in the rock support. Therefore the number of headings should be as few as possible, still providing tunnel stability. Two different designs of headings were discussed, one with a central pilot drift (3 headings) and one with a pilot drift on one side (2 headings), and the latter alternative was adapted (Fig. 3.3). The shape of the pilot drift was unfavourable from the stability point of view and, in order to achieve stability of the heading, it had to be finished as soon as possible. In order to provide adequate support to the face, a divided face had to be maintained. Thus the pilot drift had to be as short as possible yet long enough to support to the face. In the present case the length was 1 to 2 times the height of the pilot drift, corresponding to 4 to 8 m (Fig. 3.4).



Figure 3.2 The tunnel was excavated by multiple drift. View towards the bench and the divided heading.

Tunneln togs ut med delat galleri och pall. Vy mot fronten.

The rock support applied in the heading provided a good support for vertical deformation. However, the supporting action in the horizontal direction was poor as long as the supporting ring of concrete had not been closed with walls and invert. Therefore, the bench had to be excavated as soon as possible after the excavation of the heading. In the first 90 m, where the excavation was carried out with heading and bench, the bench could be left up to 50 m behind the heading. When the conditions deteriorated further the bench had to be shortened to not more than 20 m. The section could then be fully supported within 1 to 2 weeks after the excavation of the heading.

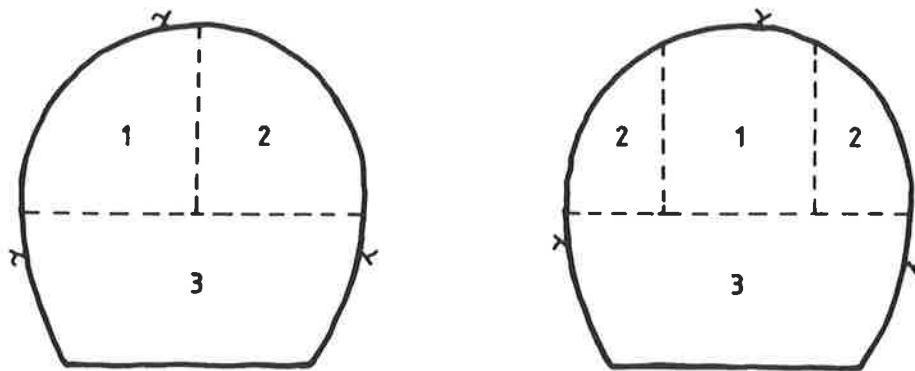


Figure 3.3 Possible designs of the heading and bench  
Möjliga utformningar av galleri och pall

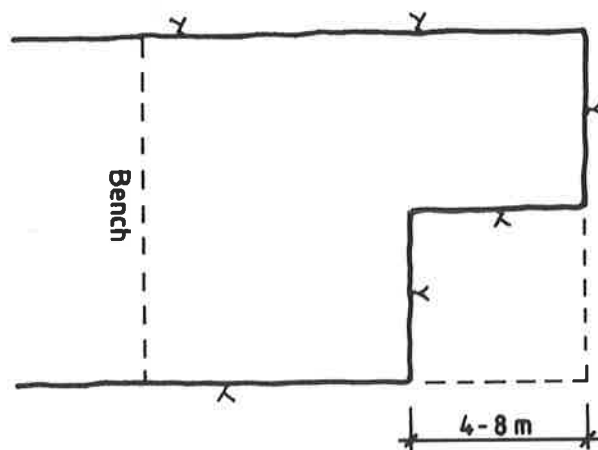


Figure 3.4 The heading was kept 4 to 8 m ahead of the right side  
Pilottunneln ligger 4 till 8 m framför strossen.

### 3.2 Pattern of deformation

#### 3.2.1 Instrumentation

When tunnelling in such poor rock conditions as encountered in this part of the tailrace tunnel, there is always a risk of long term failure. It is therefore important to monitor the movements of the rock mass in such a way that a developing failure can be taken care of in due time. The adapted system should fulfil several requirements. The measurement system must be sufficiently accurate, the results should be possible to interpret, it must be easy to take readings and it should be cheap.

The system adapted was based almost entirely on tape extensometer. Every 10 to 15 m a horizontal section was installed and every third section was a three point measurement section. A problem with the three point sections was the ventilation tube located in the centre of the roof. In many sections the tube prevented measurement and no monitoring of the vertical deformations was possible (Fig. 3.5). However, performed measurements and visual inspections indicate that the major deformation took place in the walls.

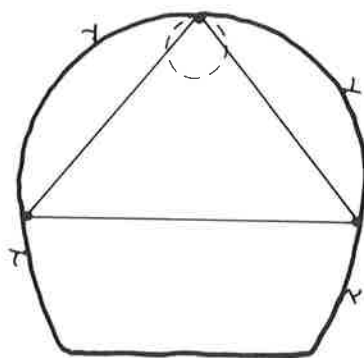


Figure 3.5 Horizontal as well as three point extensometer sections were used, the ventilation tube however prevented vertical readings in many points.  
 Horizontella och trepunkts sektioner installerades, dock hindrade lufttuben mätningar i flera sektioner.

The adapted system with tape extensometers worked well, the accuracy was high, it was a simple method and fairly cheap since all articles of consumption could be manufactured at site.

At a few places rod extensometers were installed in order to measure the extent of movement inside the rock mass. The rod extensometers were manufactured at site as one point extensometers of 5 and 10 m length. They indicated that movements extended more than 10 m into the rock mass.

### 3.2.2 Results and evaluation

Along with the excavation some considerable deformations of the tunnel took place. The total convergence was in general in the order of 0.3 m or 3 % of the diameter. The development of deformation at chainage 0/210 is shown in Figure 3.6. The graph is typical for this part of the tunnel and can be divided into three stages. It should be noted that it is practically impossible to install the measurement section until the heading is complete. Therefore the magnitude of the measured deformations is less than the actual convergence.

In the first stage (1) only the heading was excavated. The rate of deformation was then in the magnitude of 2-5 cm/week. The rock support consisted of shotcrete and dowels. The second stage was associated with the excavation of the bench. When the bench was removed (2), the tunnel walls got higher and the horizontal support was decreased. This caused a momentary convergence of 5-10 cm. The final stage (3) was reached when the temporary rock support was completed with a horizontal invert. After that the rate of deformation decreased and was less than 1 cm/week. This creeping could go on for a long time, with a gradually decreasing rate of movement.

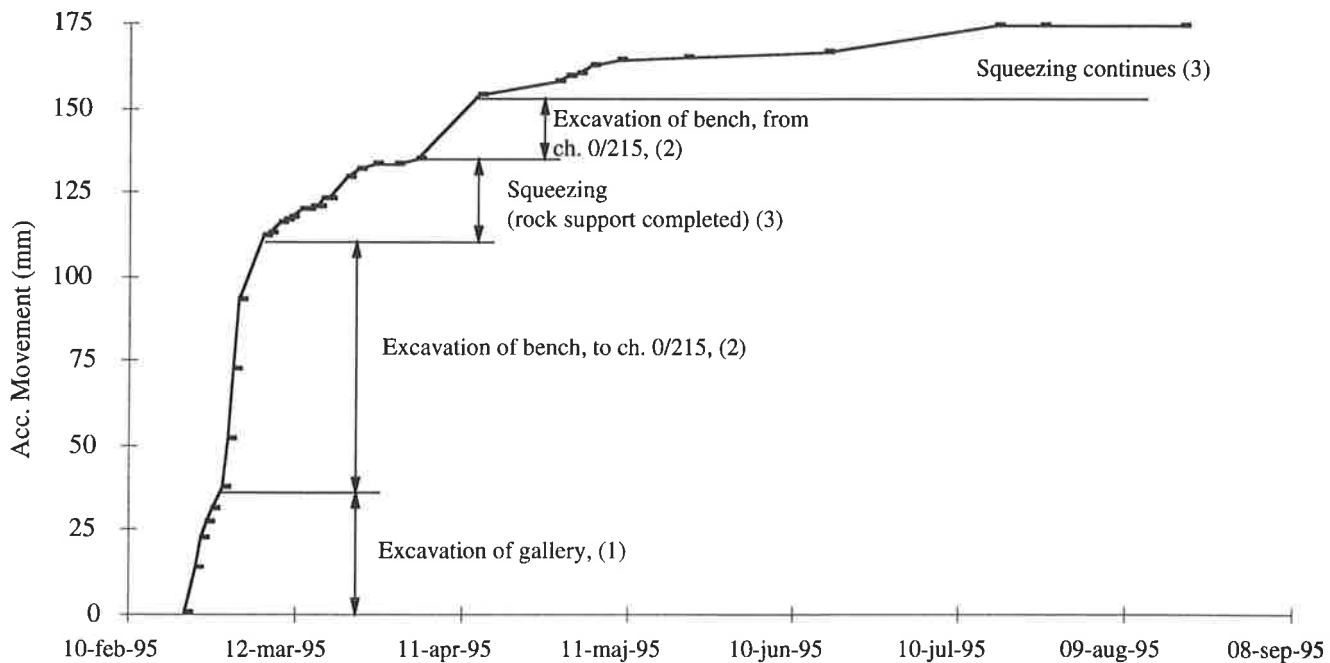


**Figure 3.6** The development of deformation can be divided into 3 parts; excavation of heading (1), removal of bench (2) and a time dependent squeezing (3). Example from chainage 0/210

Utvecklingen av deformationer kan delas in i tre delar; deformation i samband med uttaget av galleriet (1), pallen (2) och en tidsberoende krypning (3). Exempel från sektion 0/210

### 3.3 Support methods

Based on the period when active, the rock support used can be divided into three groups. Spiling was used in order to increase the stand up time and allow longer rounds. Shotcrete, dowels and lattice arches served as temporary as well as permanent supports. Finally, a cast concrete lining formed the permanent support.

Spiling was used in excavation of the entire reach between chainage 0/038 and 0/322. It was used in order to prevent loose rock in the roof and walls from raveling during the time between the blast and the application of the rock support. The spiling consisted of re-bars, installed sub-horizontally ahead of the face. In order to work properly the spiles had to be well supported in both ends, also after the blast. By extending the spiles about two metres ahead of the planned pull of the blast, this end was supported by the face. In the other end the spiles were supported by the applied rock support. Since the spiles were only supposed to take lateral load during a short period of time, there was no need to grout them (Fig. 3.7).

In the first 90 m of this zone of poor rock the temporary rock support consisted of 150 mm of fibre reinforced shotcrete, 4-6 m long grouted dowels, c/c 1.5 m and occasional wire mesh reinforced shotcrete arches. Yet as the condition of the rock mass deteriorated the amount of rock support had to be increased.

As a result of unsymmetrical deformation of the rock mass, the sprayed concrete often failed in tension. Wire mesh was therefore introduced to increase the tensional capacity of the shotcrete (Fig. 3.8). In order to avoid shadows around the mesh, unreinforced shotcrete was used to cover the wire mesh. However, in the parts where the lining was subject to high compression, the layer with wire mesh and unreinforced shotcrete turned out to be a weakness plane. Shear cracks could be initiated in the plane along the wire mesh. The design of the increased rock support was therefore a combination of wire mesh reinforced and fibre reinforced shotcrete. The support was built up of five 50 mm thick layers, two reinforced by wire mesh and three reinforced by fibre. Furthermore 6 m long dowels were installed. (Fig. 3.9).

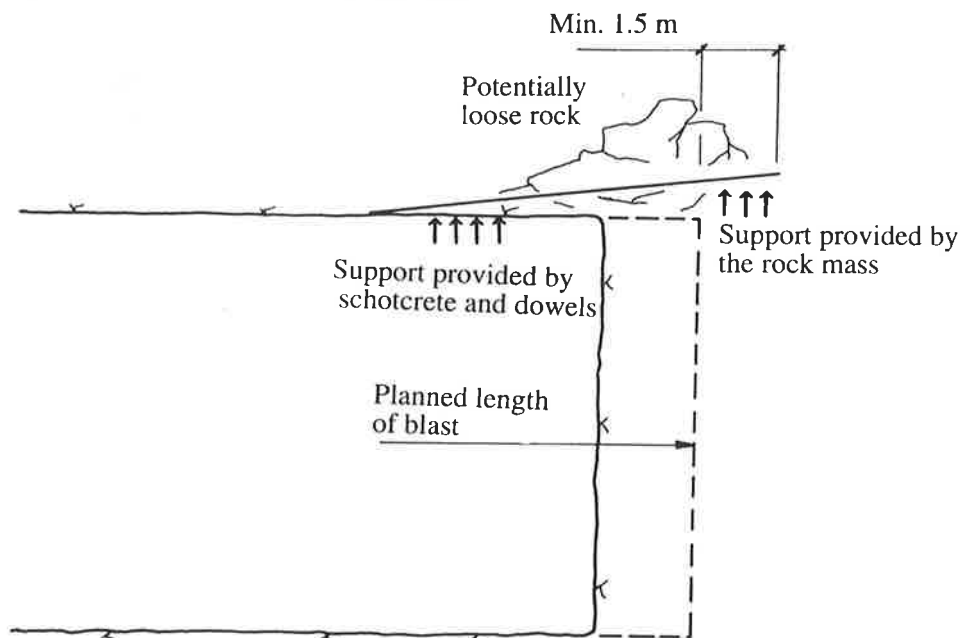


Figure 3.7 Spiling was used in order to increase the stand up time.  
Frampålning användes för att förlänga ståtiden för bergmassan.

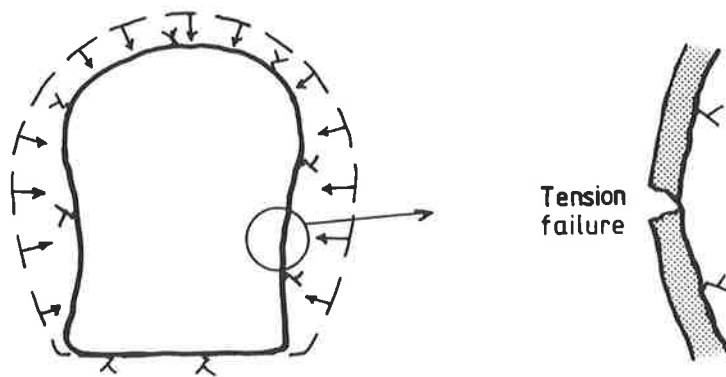


Figure 3.8 Due to inhomogeneous deformation of the tunnel, tension cracks developed.  
Då deformationerna ej var likformiga runt tunneln uppstod dragbrott.

The application of the support system had to be designed taking the squeezing of the rock mass into consideration. The aim was to expose the installed rock support to as small deformations as possible without jeopardising the over all stability of the tunnel. Since a major part of the deformations takes place close to the face, the main part of the support was applied some distance from the face. After each blast the face, roof and walls were immediately sprayed with one layer of shotcrete. This layer was extended back two rounds. Thereafter dowels, spiling and wire mesh were installed and covered by unreinforced shotcrete. The same process were repeated after each blast. In that way the final temporary support was achieved three rounds from the face (Fig.3.10).

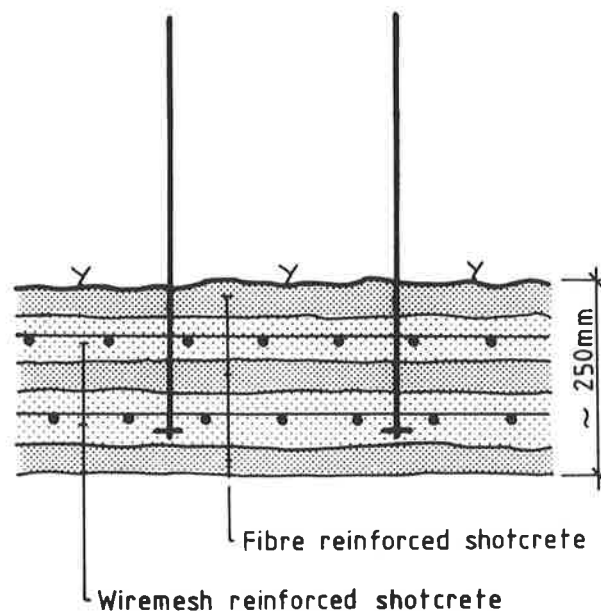


Figure 3.9 The rock support was built up by a combination of wire mesh reinforced and fibre reinforced shotcrete.  
Bergförstärkningen utgjordes av en kombination av nät- och fiberarmerad sprutbetong.

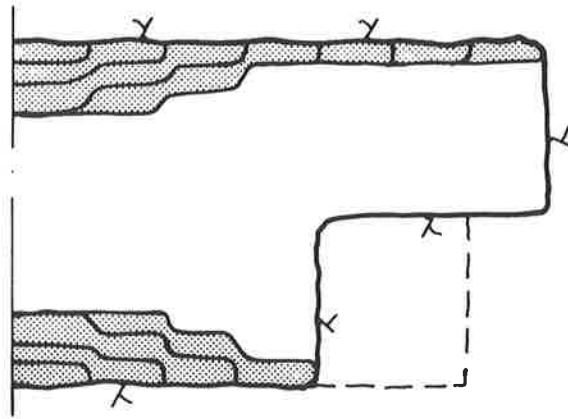


Figure 3.10 Some of the shotcrete was applied away from the face in order to restrict forced deformation of the shotcrete.  
Sprutbetongen applicerades till viss del en bit från fronten för att undvika påtvingad deformation.

Since the tunnel was excavated with a divided heading, three joints occurred in the rock support, i.e. one in the roof, and one in each wall. It was of vital importance that these joints were made as strong as possible. In the heading part of the tunnel, only the first two layer of reinforced shotcrete was applied in the pilot drift. The remaining layers were applied after excavation of the entire heading. In this way the joint in the roof was nicely covered. By the time the walls were to be supported, full treatment had already been given to the heading part. The wire mesh in the lower part of the walls of the heading was not covered with shotcrete in order to enable joining with the walls of the bench. Since no continuation can be obtained over a joint in the fibre reinforced shotcrete, it was important that the wire meshes was joined well together.

Large deformations were observed in the walls after the excavation of the bench. In order to stabilize these, a counter pressure had to be applied by the rock support. In order to enable the rock support to take horizontal loads, the ring of shotcrete was closed by casting a concrete invert.

In spite of the extensive rock support applied, the movements did not cease completely. In chainage 0/170 to 0/255 the horizontal rate of convergence was still a few mm/week in April 1995, half a year after excavation. Damages could be seen in the rock support and it was found necessary to try to arrest the movements. In order to do so, installation of lattice arches was started. These arches consisted of 20 mm reinforcement bars in a square and a triangular pattern, embedded in shotcrete (Fig 3.11). The arches were planned to be installed in two stages, in a first stage the outer layer of reinforcement bars should be installed, thereafter shotcrete should be applied, then the inner layer of reinforcement bars and finally the inner layer of shotcrete should have been placed. However, when parts of the outer layer were in place, the rate of convergence showed signs of diminishing to an acceptable level. In view of this and as construction of the arches also turned out to be complicated, the work with the arches was interrupted and the remaining bars will serve as a reinforcement in the permanent lining.

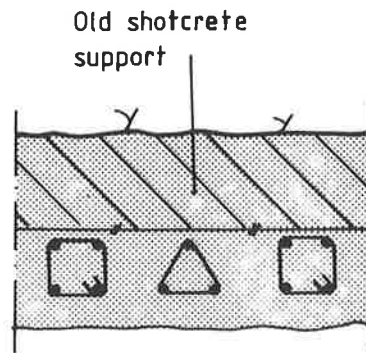


Figure 3.11 Lattice arches was constructed in a square and a triangular pattern.  
Bågar av armeringsjärn (sk. lattice arch) installerades i fyrkants- och trekantsmönster.

## 4 SQUEEZING AND LONG TERM STABILITY

### 4.1 Observed squeezing behaviour

In considerable parts of the tailrace tunnel, which were excavated in shales and schists and where the overburden was more than 300 m, an ongoing time dependent movement has been observed. The total convergence in the crushed graphitic schists in the upstream part of the tunnel was in general in the magnitude of 300 mm and the rate of squeezing was generally in the magnitude of a few mm/week. In the downstream part of the tunnel, excavated in shale, the total convergence was in general about 100 mm or less and the rate of creeping was between parts of millimetres to a few millimetres per week (Fig. 2.2).

Figure 4.1 shows the result from measurements of convergence in chainage 1/569 which was excavated in shale in the end of May 1993. The rock mass in the area is thinly bedded with a high content of clay and exhibits damp water conditions. It can be seen that initially the rate of convergence was high, but decreasing with time. The section was initially supported by 6 cm of shotcrete and 4 m long grouted dowels, c/c 2m. In the end of August 1993 severe cracking was observed in the walls and a substantial amount of shotcrete was scaled down. The walls were then supported with 10 cm of fibre reinforced shotcrete and another set of 4 m dowels, c/c 2m. In Fig 4.1 it can be seen that this treatment reduced the rate of movement. In the spring of 1994 an attempt was made to arrest the movements by installation of another set of 6 m long grouted dowels, c/c 2 m. No effect could be observed with certainty. After almost two years stable conditions had not been achieved.

In the parts of the tunnel which were excavated in the crushed graphitic schist in the upstream part of the tunnel, the rate of convergence was initially extremely high (40-100 mm/week) but after completion of the rock support, including a cast invert, the rate of movement decreased to in general 0.5-5 mm/week (Fig 3.4). This deformation rate was then continuous for a long time and not until October 1995 some completely stable sections were observed.

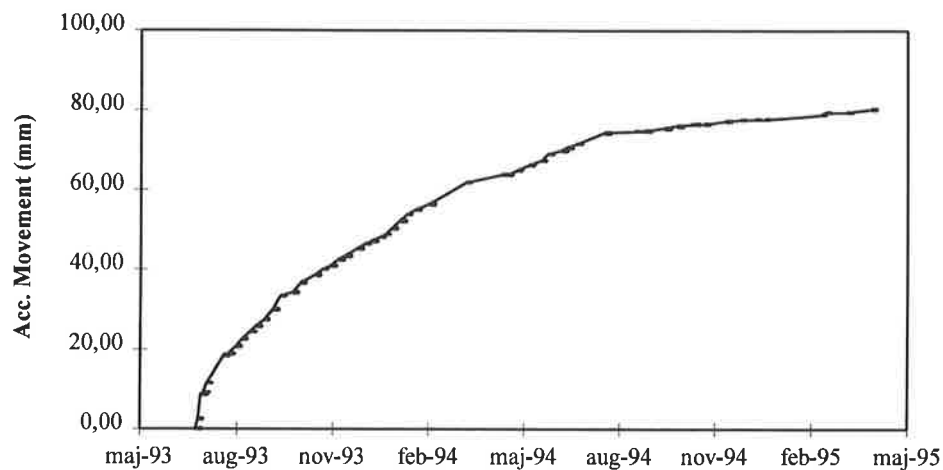


Figure 4.1 Development of deformation in shale (ch. 1/569).  
Deformationsutveckling i shale (ch. 1/569)

Common for both the schist and the shale was that the rate of deformation often was constant during a period of time and then suddenly dropped to a lower level. The graph shown in Figure 4.1 can in this way be divided into five time periods in which the rate of movement is constant (Fig. 4.2). This behaviour could be explained by adjustments in the rock mass with sudden slips along joints.

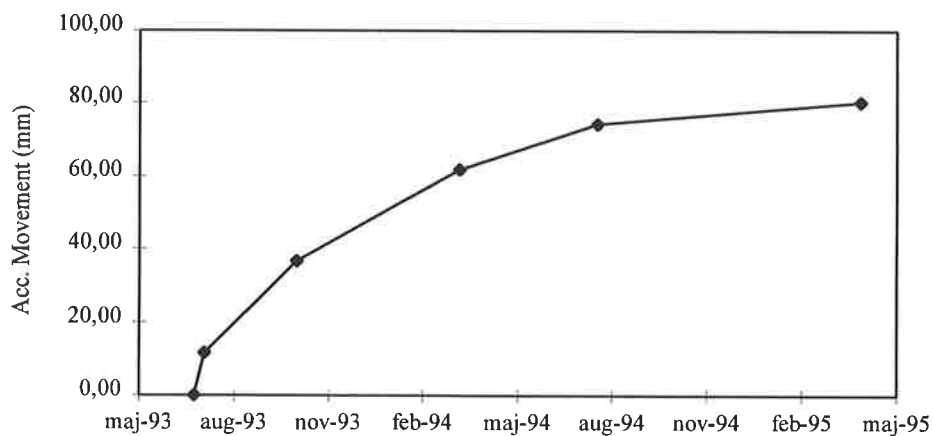


Figure 4.2 The rate of deformation is periodically constant with time. Here the graph from Fig. 4.1 is divided into 5 periods with a constant rate of movement.  
Deformationshastigheten är tidvis konstant. Här är grafen från Fig 4.1 uppdelad i 5 perioder då deformationshastigheten var konstant.

## **4.2 Considerations for the permanent lining**

### **4.2.1 General**

The permanent lining in the tailrace tunnel at the Uri Project was initially not designed to take any squeezing loads. However, when the problems with the ongoing movements became evident, they had to be taken into consideration for the design of the final lining. Following questions were asked.

- How much deformation remains?
- How does the installed rock support affect the rate of deformations?
- What load will the deformations cause in the lining?

The main problem is involved in the first question. If the future deformation can be properly estimated, then the load on the lining can be calculated and the design of the lining can be verified. In order to estimate the future deformation a valid creep model had to be established.

The initially available input was the measured convergence in the tunnel. Later on also two test linings were cast and instrumented. In the following, different available creep models are discussed in principle, and an evaluation of the recordings from the test linings is described. Based on this and the convergence measurements, a valid creep model is finally defined.

### **4.2.2 Theoretical models**

In the literature a number of different approaches have been used in order to describe time dependent behaviour of a rock mass. Available theories can be categorised in two fundamentally different groups, based on the development of deformation.

One group is based on the "ageing theory". The rock mass is characterised by an initial and a residual strength. The creep is a result of the loss of strength and ultimate deformation can be defined by the residual strength (Ladanyi 1974).

Another approach is using the Maxwell visco elastic model. The rock mass is assumed to have no long term shear strength and the deformations are therefore continuous as long as the stress field is not hydrostatic. Such a model yields loads on a lining equal to the initial in situ stress, or continuous deformations until the entire tunnel is closed. This model is mainly valid for salt mines but some shales and schists have shown a similar behaviour (Ladanyi, 1984)

A number of models has been based on a combination of these two ultimate ways to define the rock behaviour. These models are using a visco-elastic basic model, stability is however assumed when the deviatoric value becomes less than a certain limiting value.

The ageing theory yields a definite amount of squeezing and the viscoelastic approach yields an ongoing continuous deformation. When designing a lining it is important to

define which approach is appropriate. If the Maxwell model is valid for the rock mass, the final theoretical pressure on the lining will be equal to the initial in situ stress. Since it is impossible to design for such large pressures, the lining has to be designed to last for a certain period of time instead. However, if the rock mass is characterised by the ageing theory, the final expected deformation can be calculated and the lining designed accordingly.

#### **4.2.3 The effect of installed rock support and permanent lining**

The development of deformations is a combination of the behaviour of the rock mass and that of the rock support. In Figure 4.3 the squeezing behaviour of a rock mass is illustrated by the ground reaction curve concept. This figure shows that it is the movement of the *supported* rock mass which is measured. In order to show the behaviour of the rock mass only, the effect of the rock support has to be subtracted. From Fig. 4.3 it is evident that, even if the unsupported rock mass continuously approaches a state of equilibrium, the measured combined effect from the rock mass and the support can still show a continuous rate of movement.

The effect of installing the lining can be studied in principle with the ground reaction curve. At the time of casting the lining the relation between the rock support and the rock mass is represented by point A (Fig. 4.3). The maximum possible deformation the lining will be subjected to is then defined by the distance b. However, because of the stiffness of the lining, the actual deformation will be less and instead pressure will build up in the lining. The maximum possible pressure is indicated by the distance c. The actual result will involve a pressure less than c and a deformation less than b. Therefore these values can be used as a worst case estimate.

### **4.3 The selected formulation of creep**

Within the scope of a master thesis (Johansson and Stille; 1995) the available convergence measurements were analysed and some back calculations were carried out. The authors concluded that the shale could be described as an ageing rock mass, characterised by an angle of friction, a cohesion and a modulus of elasticity. According to the thesis, the shale could be characterised by a angle of friction  $\phi=25^\circ$ , modulus of elasticity,  $E=3$  GPa and a time dependent cohesion,  $C = 440 * e^{-0.008t} + 360$ .

In this connection the authors also assumed that even the graphitic, phyllitic schist could be described by the ageing theory, meaning that the rock would reach a state of equilibrium after a sufficient amount of squeezing. This assumption was made in 1994 on the basis of at that time available measurements. However, the only way the assumption could be confirmed with certainty was that the deformation ceased completely in some sections. In October 1995 stability was reached in several sections along the critical initial 300 m of the tailrace tunnel.

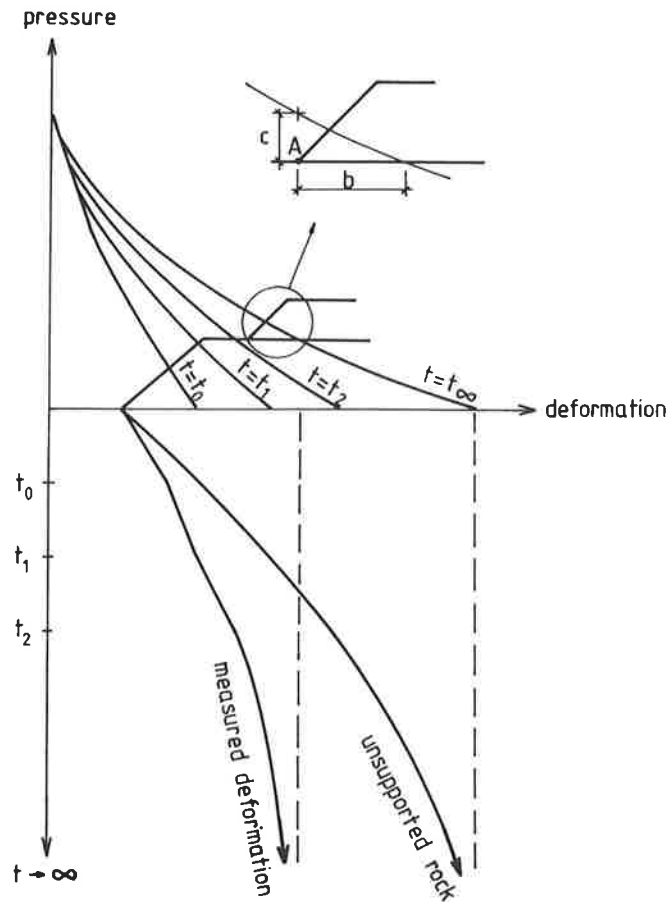


Figure 4.3 The measured deformation is the combined effect of the rock mass and the rock support. The maximum possible deformation and pressure can be estimated from this concept.

Den uppmätta deformationen beror både av bergets och bergförstärkningens egenskaper. Maximal deformation och maximal belastning på den permanenta inklädnaden kan uppskattas ur detta koncept.

## 5 TEST LININGS

### 5.1 General

In order to gain a better understanding of how the ongoing movements would affect the permanent lining of the tunnel, two test linings were cast. The first was cast at chainage 1/384-1/389 in the middle of May 1994. This reach is a transition zone between marble of good quality and graphitic-phyllitic schists of poor quality. The second lining was cast in April 1995 at chainage 1/558-1/576 where the rock consists entirely of red, thinly bedded shale with a high content of clay.

The first lining was cast only three months after excavation of the section and at the time of casting the face was only 30 m ahead. No measurements of convergence were carried out before casting of the lining. After casting a convergence of about 100 mm has been measured 6 m and 14 m m from the sections at ch. 1/370, and at ch. 1/378. At the location for the second test lining, measurements of convergence had been carried out for a long time. At the time of casting the rate of convergence was here 0,2 mm/week (Fig. 5.1).

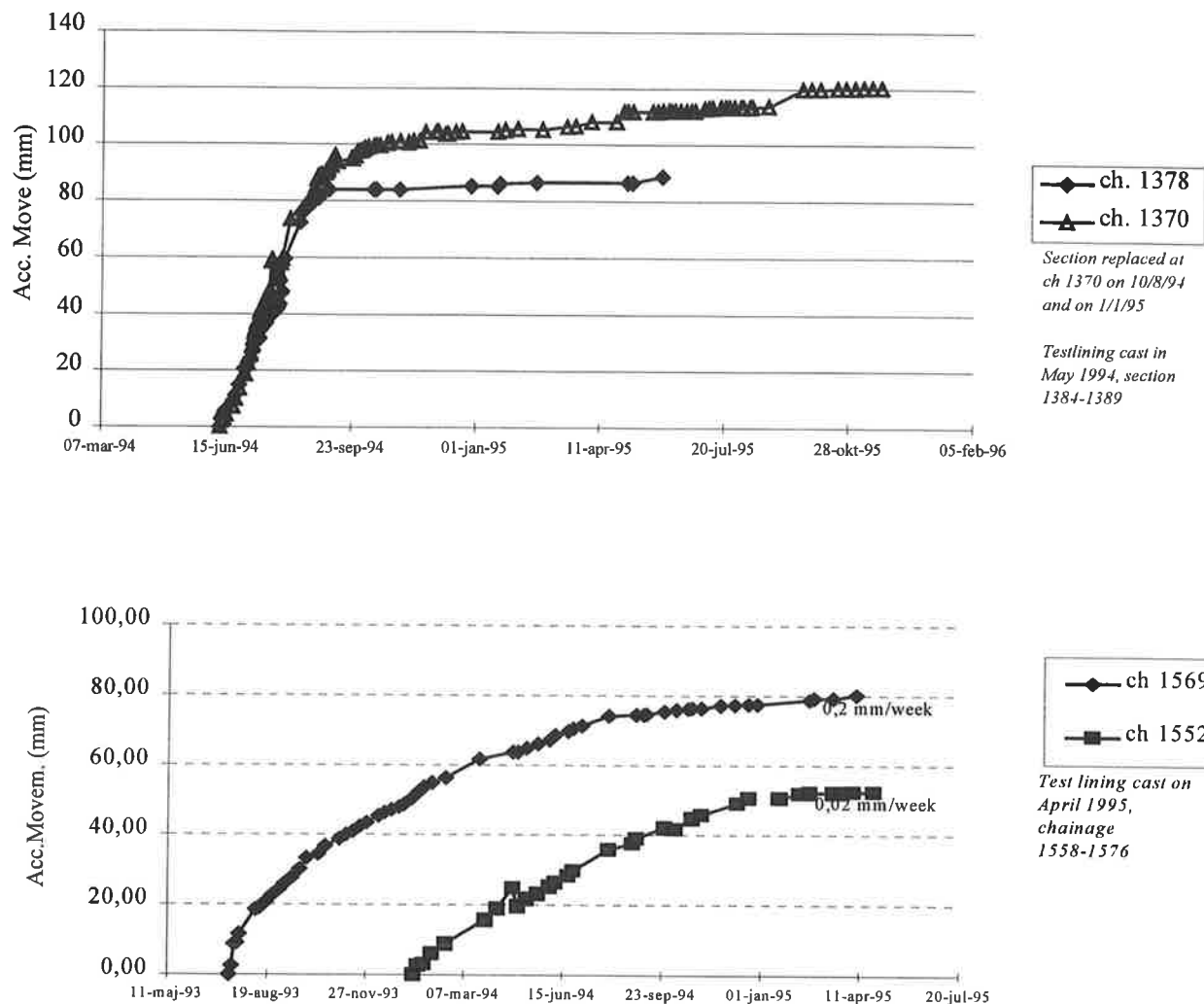


Figure 5.1 The horizontal convergence 6 and 14 m from the first test lining (upper)  
 The horizontal convergence at the location of the second test lining by  
 the time the lining was cast (lower)  
 Horisontell konvergens 6 och 14 m från den första testinklädnaden (övre)  
 Horisontell konvergens vid den andra testliningen då testinklädnaden  
 gjöts (nedre)

## 5.2 Measurement program

On each lining two types of deformation measurements were carried out according to principles lined out by Kovári et al. 1977. The convergence of the tunnel was measured in three directions, horizontally and vertically on the left and on the right side. Further, change in curvature and tangential strain was measured along the periphery of the lining on one metre long sections (Fig 5.2). The instrument used to measure the convergence was a tape extensometer and the change of curvature and tangential strain were measured by curvometer and distometer, respectively. The first test lining was provided with one measurement section and in the second lining three sections were installed. Readings were normally taken every week or every second week. The accuracy of the

tape extensometer and the curvometers was about 5/100 mm. The distometer has been somewhat more accurate.

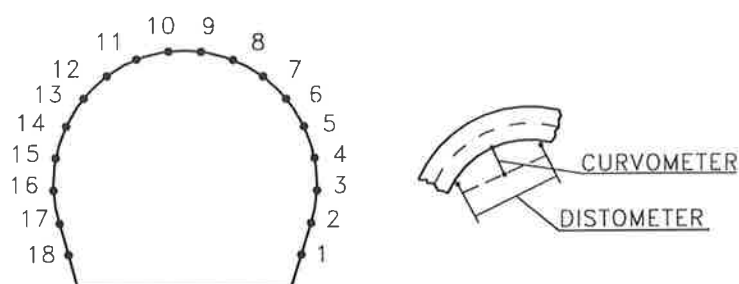


Figure 5.2 The deformation of the lining was measured in 18 sections of 1 m length. Deformationenen mättes i 18 enmeters sektioner.

### 5.3 Evaluation of the results from the measurements

The convergence measurements indicate that the tunnel at the test lining is now stable. In total a horizontal convergence of about 3 mm and a vertical deformation of 1.5 mm has been measured.

An example from measurement with distometer and curvometer is shown in Figure 5.3. The graphs have been corrected for obvious errors due to misreadings and errors caused by mechanical damage to the measurement points. Here it can be seen that the rate of deformation decreases with time.

The deformations obtained from curvometer and distometer can be converted into stresses within the concrete. In order to do so, two assumptions have to be made. Firstly, elastic conditions are assumed to apply to the concrete ring, and secondly, the moment of inertia is assumed to be known and constant for the entire lining.

Changes in curvature and length are known from the measurements. From the theory of elasticity the moment and normal force can be calculated. Based on this calculation the radial pressure which is acting on the lining is then computed. The boundary conditions are defined by the deformation of adjacent sections (Kovári et al. 1977).

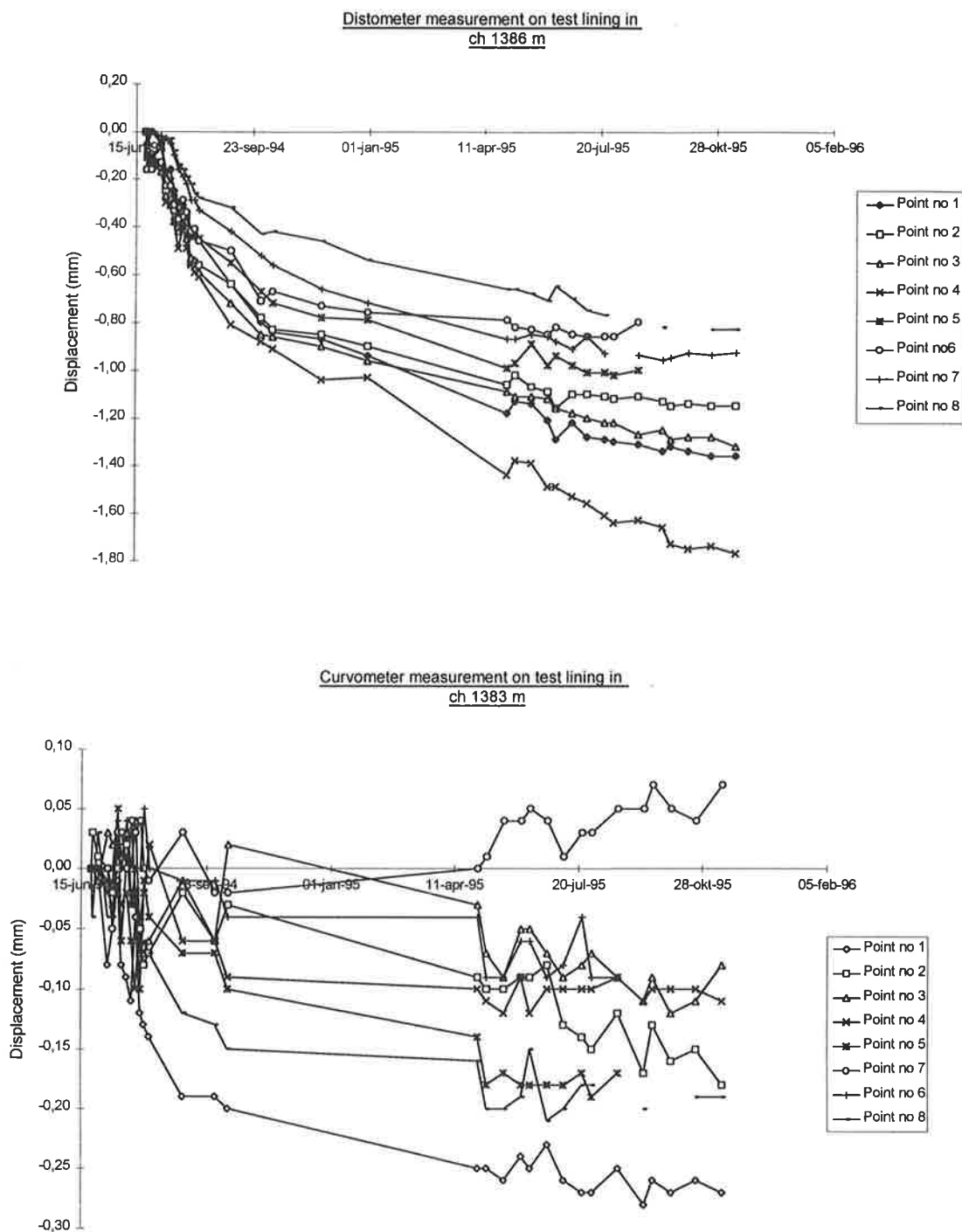


Figure 5.3 Examples of results from distometer and curvometer readings. (note that due to differences in the scale on the y-axis, the readings from the curvometer appears more scattered than those from the distometer)

Exempel på resultat från distometer och kurvometer mätning. (notera att skillnader i skalan på y-axeln gör att kurvometermätningarna ter sig mer osäkra än distometermätningarna)

In this way the radial pressure as well as the tangential stress were calculated for all four sections. The result is shown in Figure 5.4 and 5.5. From these figures it can be concluded that the radial pressure was as an average 3 MPa in the first test lining (ch. 1/383) whereas it was 0.6 MPa in the second lining (ch. 1/564-1/571). The tangential stresses in the concrete were on the average 14 MPa with a peak stress of 28 MPa in the first test lining and 6 MPa and 12 MPa, respectively, in the second lining. The tangential stresses are evaluated as mean stresses over the entire cross section, hence the maximum stress was somewhat larger. It can also be seen that the right side of the lining was in general subjected to a higher load than the left side. This is in accordance with the tape extensometer measurements performed.

It is hardly surprising that the stress level was higher in the first lining than in the second one. In the near vicinity of the first lining a convergence of 100 mm took place after casting the lining. Furthermore, the first lining is only 5 m long and it is reasonable to believe that additional load has been transferred on the lining from the boundaries. It can be concluded that the lining adds substantially to the stability of the tunnel and that the probability of overloading of the permanent lining is small.

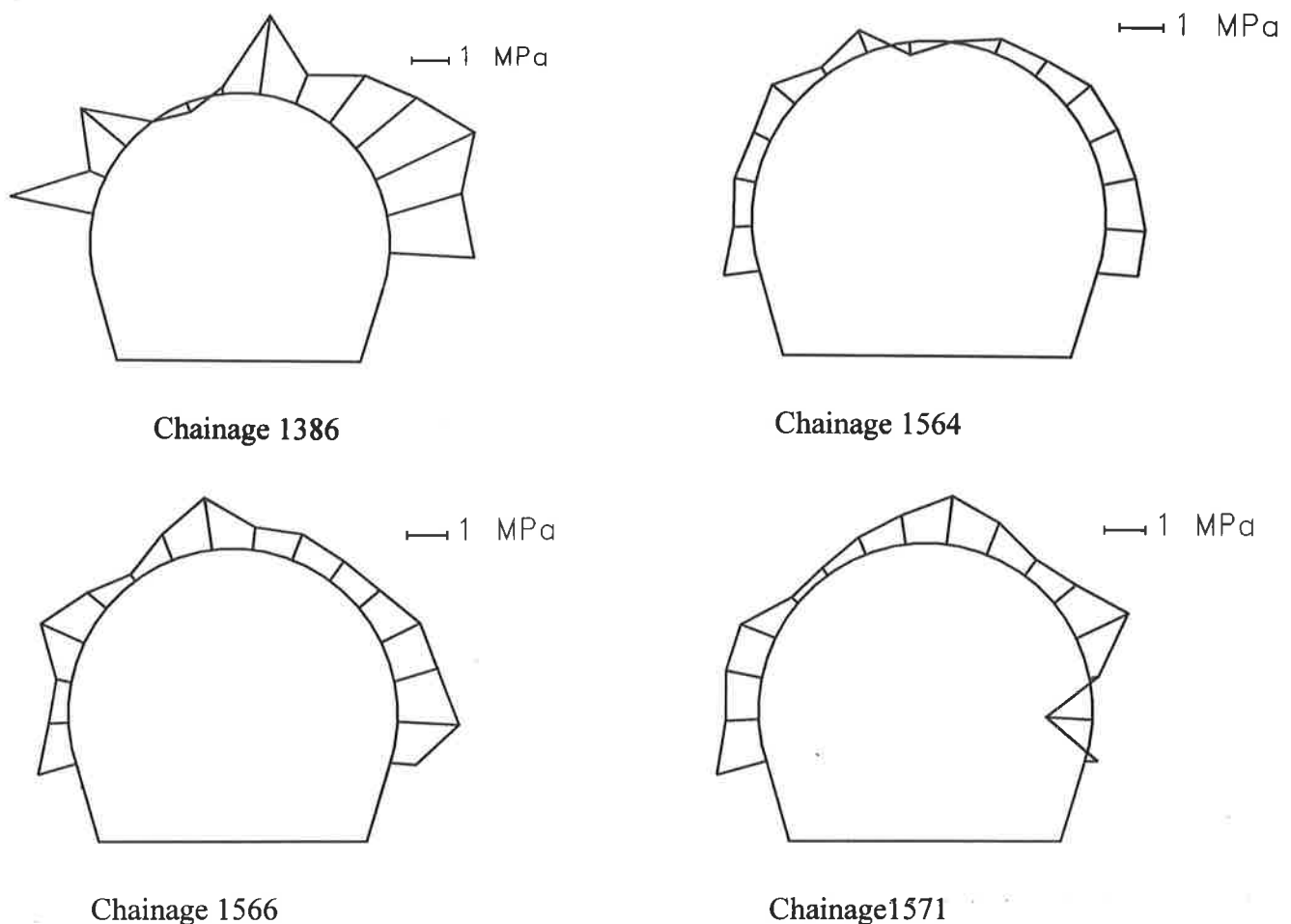


Figure 5.4 Calculated radial pressure for the different measurement sections.  
Beräknade radiella tryck verkande på testinklädningarna.

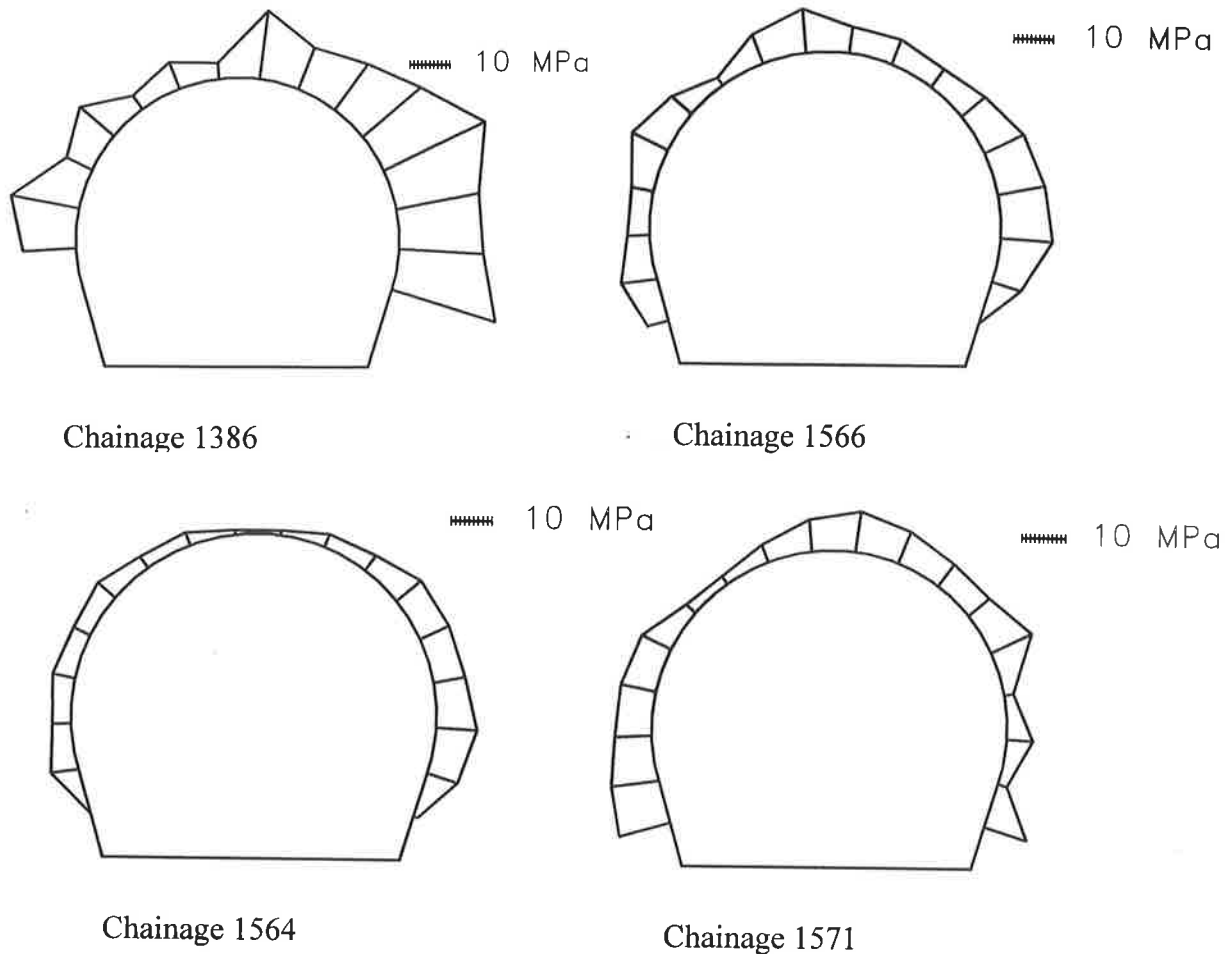


Figure 5.5 Calculated tangential stress in the lining.  
Beräknad tangentialspänning i inklädnaden.

## 6 Conclusions

The Uri Project is situated in an area with a complex geology. The tailrace tunnel intersects the Panjal thrust which is associated with large tectonic movements and extensive weakness zones. The rock consists mainly of nummulitic shales, graphitic-phyllic schists and Panjal volcanics. The shales and schists are weak but had nevertheless an ability to stand considerable deformations.

Because of the relatively high overburden and the low strength of the rock in the tailrace tunnel, problems occurred with the short term as well as the long term stability. In spite of the poor rock at chainage 0/38-0/322 the tunnelling was successfully carried out with conventional drill and blast technique. The use of spiling and multiple drift was essential in order to gain sufficient stand up time to allow a proper rock support to be installed. The adopted support system with a combination of wire mesh and steel fibre as reinforcement in the shotcrete yielded sufficient strength and stiffness to stabilize the weak rock encountered.

While tunnelling soft ground, a measurement programme has to be an integrated part of the production cycle. The continuous monitoring of the movements produced valuable information about the stability of the tunnel. The adopted system with tape extensometers fulfilled the requirements on accuracy, simplicity and cost.

The development of deformations in the graphitic phyllitic rock takes place in three stages. In the first stage the convergence is associated with the excavation of the heading. The second stage is associated with the excavation of the bench and yields a momentary convergence. This second stage of deformation is of the same magnitude as that of the initial stage. The third phase of deformation is characterised by a time dependent squeezing.

The squeezing phenomenon observed both in the shale and in the crushed graphitic schist could be described as an ageing phenomenon where the ultimate deformation can be calculated from the residual strength. Back calculation showed that the shale could be characterised by a constant angle of friction and by a cohesion decreasing exponentially with time.

The test linings yielded valuable information about the capacity of the lining to stand squeezing loads, and the instrumentation gave a good picture of the distribution of stresses around the lining. Based on the results from measurements in the test linings and measurements in the tunnel it can be concluded that the lining will be subjected to a pressure in the magnitude of 1 MPa which is about 1/3 of its bearing capacity.

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## SAMMANFATTNING

Utloppstunneln vid Uri-projektet i Kashmir, Indien, har färdigställts efter drivning undersvåra bergförhållanden. Tunneln går bl.a. genom Panjal Thrust som är en av de större överskjutningarna i denna del av Himalayas. Bergmassan består av krossade och starkt tektoniserade gimmerskiffrar, innehållande en stor mängd grafit och klorit, samt meta-vulkanit och lerskiffer (Fig. 2.2) Utloppstunneln har en upp till 400-500 meters bergtäckning vilken i kombination med den svaga lerskiffern och den krossade glimmerskiffern leder till tidsberoende krypningar i bergmassan.

Den krossade skiffern som uppträder främst i början av tunneln (sektion 0/020-0/320, Fig. 2.2) var svag med kort ståtid vilken ställde stora krav på valda drivnings- och förstärknings-metoder. Den första delen av denna sträcka kunde tas ut genom drivning med galleri och pall i kombination med frampålning, men efter 90 meter försämrades bergkvaliteten ytterligare vilket ledde till instabilitet i tunnelfronten. Drivningen fick då fortsättas med ett delat galleri (multiple drift) och mängden förstärkning fick ökas från 150 mm fibersprutbetong med 4 m gjutbult, c/c 1,5 m till 250 mm av en kombination av nätarmerad och fiberarmerad sprutbetong och 6 m gjutbult c/c 1,2 m. Under drivningen av hela denna sträcka har frampålning använts för att öka ståtiden och därigenom möjliggöra längre salvor. Frampålningen har utgjorts av armeringsjärn som borrarats in c/c 30 cm runt om profilen, riktade utåt ca 10°. Järnen har i sin yttre ända sprutats in bakom nätarmerad sprutbetong, och de sträckte sig in ca 2 m framför den planerade salvan. På detta vis hindras eventuell löst berg i tak och vägg från att falla ned. Därigenom kan tunnelprofilen behållas, vilket är viktigt ur stabilitetssynpunkt.

Vid uttag av denna sträcka med mycket svagt berg har deformationer varit ett stort problem. I samband med tunneluttaget skedde en konvergens av ca 25 cm, därefter följde en långsam krypning. Den kraftiga rörelsen i samband med drivningen gjorde det önskvärt att installera huvuddelen av förstärkningen en bit från stuffen. På det viset utsattes den för en så liten tvångsdeformation som möjligt.

I Fig. 2.2 indikeras att problem med tidsberoende deformationer uppträdde längs knappt halva tunnelsträckningen. Kryphastigheter på delar av millimetrar upp till runt en cm per vecka har uppmätts längs tunnel. Då tunneln slutligen skulle kläs med en betonginklädnad var det väsentligt att undersöka hur inklädnaden kommer att påverkas av bergrörelserna. För att avgöra detta gjöts två provinklädnader som instrumenterades för deformationsmätning i enlighet med en metod som har beskrivits av Kovári et al. 1977. Den första inklädnaden gjöts i en övergångszon mellan marmor och glimmerskiffer, medan den andra gjöts i lerskiffern. Den första inklädnaden gjöts endast 30 meter från tunnelfronten. Efter gjutningen har en konvergens på ca 100 mm uppmätts som en följd av tunnelsprängningen och en vidare krypning (Fig. 5.1). Vid läget för den andra inklädnaden hade deformationerna redan avtagit till mindre än 0,5 mm/vecka. Ändringar i krökning och längd mättes i 1 m långa bitar längs tunnelns periferi (Fig 5.2).

Utvärderingen av provinklädnaden visade att betonginklädnaden har en god stabiliserande effekt. Ett radiellt tryck på runt 3 MPa har uppmätts vid den första inklädnaden medan motsvarande tryck i den andra var 0,6 MPa (Fig. 5.4, 5.5). 3 MPa är i högsta laget men kan förklaras med de stora deformationer nära inklädnaden som orsakats av sprängning vid den närliggande fronten och av det tidiga stadiet av krypning.



# **INJEKTERING OCH FLÖDEN I BERG**

## **Grouting and Water Flow in Rock**

Introduktion: Lars Lundström, J & W



## FLÖDESTEST OCH SPRICKVIDDSBESTÄMNING AV NATURLIGA BERGSPRICKOR

Flow experiments and aperture measurements on natural rock fractures

*Eva Hakami, Itasca*

*Erik Larsson, CTH*

### INLEDNING

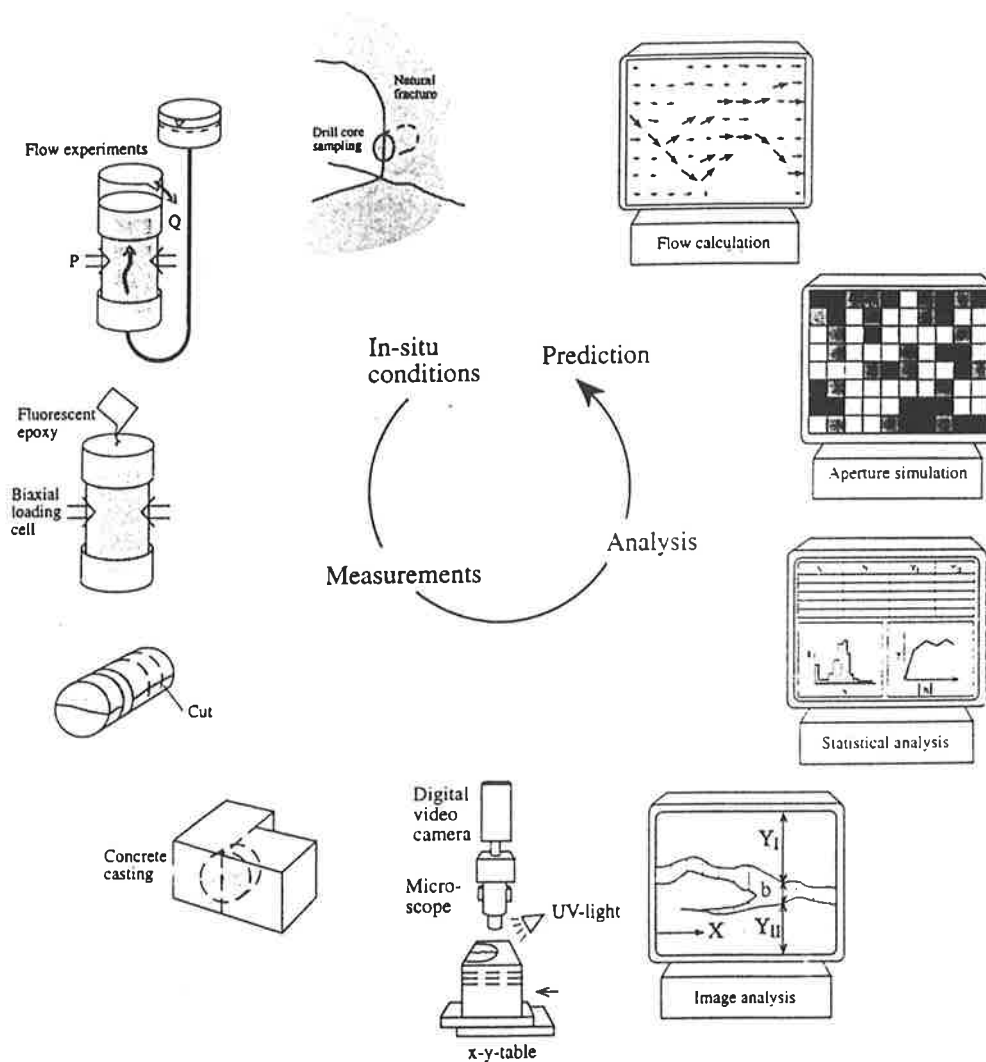
Vattenströmning i bergsprickor är ett ämnesområde som har tagits upp ett flertal gånger på BM-dagen de senaste åren. Man konstaterade till exempel vid studier av injekterings styrmekanismer (BM-dagen 1991 [1]) att bergsprickornas öppningsgeometri styr injekteringsresultatet och vilken metod som bör väljas. De hydrauliska tester som används för att undersöka bergmassans egenskaper, och vars resultat ligger till grund för design av tätningsåtgärder, kräver också att man gör antaganden om strömningens (dvs spricköppningens) geometri (BM-dagen 1994 [2]).

Forskningen i samband med frågan om slutförvaring av radioaktivt avfall har ytterligare fokuserat intresset kring de hydromekaniska egenskaperna hos bergsprickor. Forskningsresultat från Stripa-projektet och Äspö-laboratoriet har presenterats här på BM-dagen 1992 och 1995 [3-5]. En viktig del av denna forskning är ämnat just åt att visa hur, och i vilken grad, grundvattentransport av radioaktiva partiklar skulle kunna ske från ett framtida förvar i berg. Transporttiden för partiklar i grundvatten beror till stor del på hastigheten hos vattenflödet. Flödes hastigheten varierar från punkt till punkt i bergmassan på grund av variationen i spricköppning, dels mellan olika sprickor och dels över ett enskilt sprickplan.

Vid förra årets bergmekanikdag presenterades de hittills vunna erfarenheterna från DECOVALEX-projektet [6]. I detta pågående internationella projekt söker man utveckla beräkningsmetoder för att studera termo-hydro-mekaniska processer i bergmassan. En av erfarenheterna som fördes fram var den att största svårigheten vid utvärderingen av beräkningsmodellerna låg i bristande kännedom om de bakomliggande fysikaliska mekanismerna för sprickor.

Dessa ovan nämnda arbeten visar, tillsammans med forskningsresultat från många andra länder, på behovet av bättre kunskap om vattenströmning i enskilda sprickor. För att knyta samman de bergmekaniska och geohydrologiska erfarenheterna har ett samarbetsprojekt pågått under 1992 - 95 mellan avdelningen för teknisk geologi, KTH och geologiska institutionen CTH med SKB som uppdragsgivare och finansär.

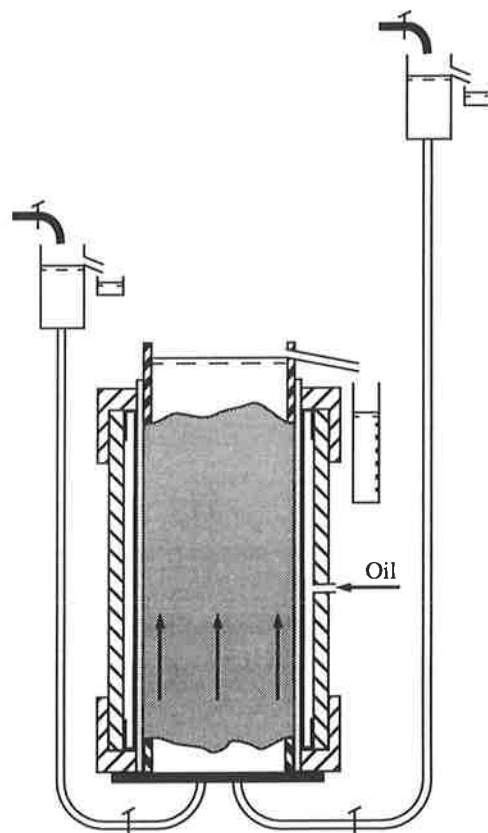
Projektets övergripande mål var att prediktera grundvattenströmning och transport i bergsprickor. Figur 1 illustrerar och sammanfattar de olika stegen i projektet. Först bestämdes ett experimentellt värde på flödet genom provet med ett flödestest. Sedan mättes sprickviddsfördelningen och med hjälp av beräkningsmodeller baserade på fördelningen kunde en prediktion av flödet göras. Genom att jämföra de experimentella resultaten med prediktionen kunde beräkningsmodellerna utvärderas. De olika mätmetoderna beskrivs närmare i det följande. För närmare redovisning om analys och beräkningsdelen refereras till Hakami [7] och Larsson [8].



Figur 1 De olika momenten i projektet.  
The different steps taken in the project.

## FLÖDESEXPERIMENT

En borrhärna med diameter 190 mm och en längd på ca 415 mm borrades ut vid Äspö-laboratoriet. Orienteringen valdes på sådant sätt att kärnans längdaxel var parallell med skärningslinjen mellan två sprickplan. Det var dock endast ett av de två sprickplanen som var hydrauliskt aktiv. Kärnan placerades i en biaxialcell med vilken ett omgivande tryck på kärnan kunde varieras. Till cellens nedre ända anslöts en tätslutande gavel där vatten kunde ledas in, figur 2. Genom detta fick vi ett vattenflöde genom sprickan

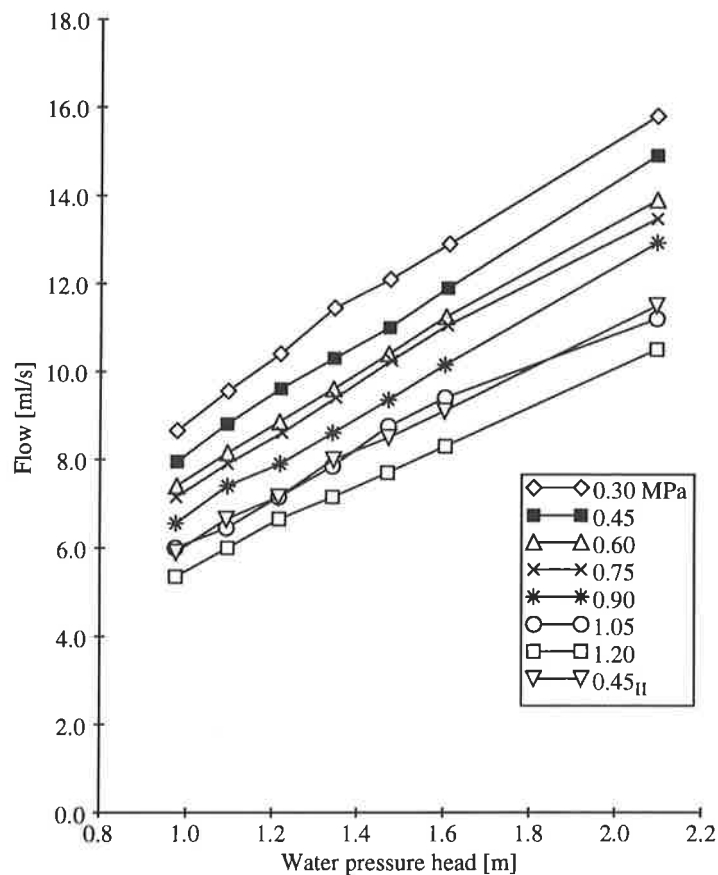


Figur 2 Den principiella uppställningen vid flödesförsöken med borrhärnan i biaxialcellen.

*The arrangements of the flow experiments with the core in the biaxial cell.*

nerifrån och upp. På detta sätt reducerades risken för att luft skulle fastna i sprickan. Vid kärnans övre ända placerades ett utloppsrör så att en konstant vattennivå kunde bibehållas och vattenflödet genom sprickan mätas upp. Flödesmätningarna genomfördes med olika hydrauliska gradienter och normaltryck över sprickplanet.

I figur 3 ser vi det uppmätta flödet som funktion av skillnaden i vattentryck. Normaltrycket har ökats stegvis för varje lastcykel. Punkterna som är benämnda 0.45<sub>II</sub> är mätta i nästa lastcykel med ett tryck på 0.45 MPa. Tiden mellan de två lastcyklerna var



Figur 3 Flödet plottat mot vattengradienten för olika normaltryck.

*The water flow plotted against the water gradient for various compression pressures.*

cirka två och ett halvt dygn. Vi kan alltså konstatera att vi har skillnad i flödet även efter flera lastcykler. Flödet är direkt proportionellt mot gradienten vilket innebär att vi har laminärt flöde i sprickan.

I tabell 1 är transmissiviteten,  $T$ , och den hydrauliska sprickvidden,  $e$ , beräknade utgående från flödet. Den hydrauliska sprickvidden definieras som sprickvidden hos en spricka med konstant spricköppning som skulle gett samma flöde som den naturliga sprickan gav. Den hydrauliska sprickvidden minskade med  $40 \mu\text{m}$  när trycket över sprickan ökade från  $0.3$  till  $1.2 \text{ MPa}$ . Detta motsvarar en kompressibilitet  $\kappa = 2 \cdot 10^{-6} [\text{Pa}^{-1}]$ . Den mekaniska deformationen av sprickan registrerad av lägesgivare över sprickan vid kärnans båda kortsidor var för det aktuella tryckintervallet  $31 \mu\text{m}$  respektive  $27 \mu\text{m}$ .

Tabell 1 Beräknad Transmisivitet och hydraulisk sprickvidd för olika tryck över sprickan.

*Calculated transmissivity and hydraulic aperture at different pressure over the fracture.*

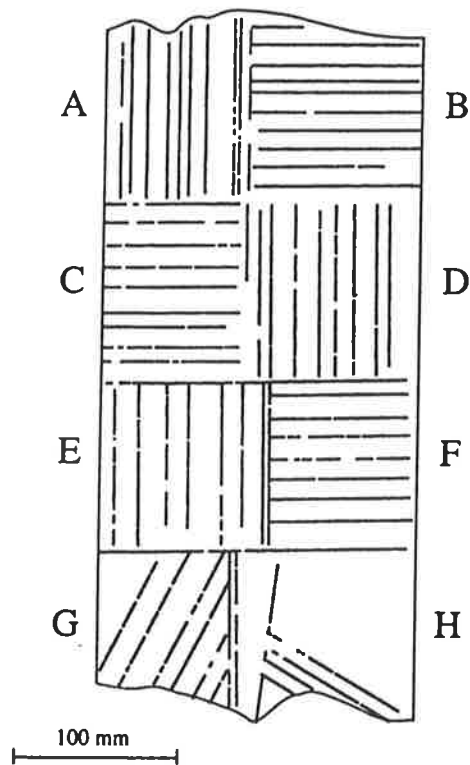
| $P$ [MPa]          | $T$ [ $m^2/s$ ]      | $e$ [ $\mu m$ ] |
|--------------------|----------------------|-----------------|
| 0.30               | $1.81 \cdot 10^{-5}$ | 285             |
| 0.45               | $1.67 \cdot 10^{-5}$ | 277             |
| 0.60               | $1.56 \cdot 10^{-5}$ | 271             |
| 0.75               | $1.51 \cdot 10^{-5}$ | 268             |
| 0.90               | $1.40 \cdot 10^{-5}$ | 261             |
| 1.05               | $1.27 \cdot 10^{-5}$ | 253             |
| 1.20               | $1.15 \cdot 10^{-5}$ | 245             |
| 0.45 <sub>II</sub> | $1.19 \cdot 10^{-5}$ | 248             |

## SPRICKVIDDSBESTÄMNING

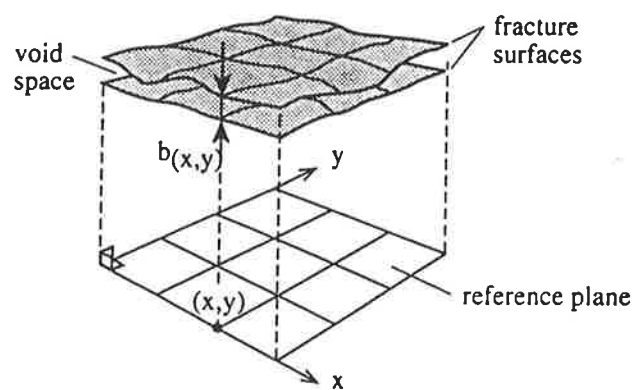
Efter avslutade flödesförsök injekterades sprickan med flourescerande epoxy som fick härda medan kärnan fortfarande satt kvar i lastcellen. Epoxyskiktets tjocklek motsvarar därmed sprickvidden vid det aktuella trycket. Skiktets tjocklek kunde mätas genom att kärnan först sågades upp i fyra segment. Dessa delar göts in i betong och kapades allteftersom längs mätprofilerna (se figur 4). Profilerna studerades i ett stereomikroskop som var direkt anslutet till ett bildbehandlingssystem. Med hjälp av lämpliga filter i mikroskopet var det möjligt att få en bra kontrast mellan berget och epoxyn. Bildbehandlingssystemet programmerades att mäta sprickvidden vid punkter på ett specificerat avstånd längs profilen. Sextiofem profiler mättes med 200  $\mu m$  mellan mätpunkterna, fyra profiler med 100  $\mu m$  och en med 50  $\mu m$ . Totalt registerades cirka 30000 mätvärden.

Parametern sprickvidd,  $b$ , är här definierad enligt figur 5. Sprickplanet antas ligga parallellt med ett referensplan. Sprickvidden är det punktvisa avståndet mellan sprickytorna vinkelrätt mot referensplanet.

I tabell 2 redovisas statistiska parametrar för sprickvidden i de åtta olika delområdena på sprickytan (se figur 4). Medelvärde för sprickvidden på hela sprickytan var cirka 360  $\mu m$ . Det kan jämföras med den hydrauliska sprickvidden på 248  $\mu m$ . Medelsprickvidden är alltså 1.4 gånger så stor som den hydrauliska sprickvidden. Den hydrauliska sprickvidden är oftast mindre än medelsprickvidden därför att man inte får ett rätlinjigt flöde i en naturlig spricka och att trånga sektioner kontrollerar totalflödet.



Figur 4 Mätprofilernas lägen på sprickan med delområden A-H.  
*Location of measurement profiles on the fracture surface with subareas A-H.*



Figur 5 Definition av sprickvidd.  
*Definition of fracture aperture.*

Tabell 2 Statistik över sprickvidder fördelat på åtta delområden  
*Statistics of apertures in eight subareas of the fracture*

| Delyta                   | a    | b    | c    | d    | e    | f    | g    | h    |
|--------------------------|------|------|------|------|------|------|------|------|
| Antal datapunkter        | 5836 | 5825 | 3525 | 3840 | 3220 | 3468 | 2423 | 1679 |
| medelvärde               | 365  | 307  | 380  | 349  | 384  | 358  | 379  | 360  |
| standardavvikelse        | 133  | 133  | 151  | 145  | 157  | 150  | 207  | 231  |
| median                   | 366  | 317  | 373  | 338  | 358  | 331  | 336  | 321  |
| $b < 50 \mu\text{m}$ [%] | 5.5  | 7.0  | 3.0  | 3.5  | 0.5  | 0.7  | 0.7  | 3.0  |

Datan från sprickviddsmätningen kan även analyseras med avseende på den spatiella korrelationen. Med hjälp av beräknade variogram kan det så kallade korrelationsavståndet bestämmas. Denna parameter talar om inom vilken radie runt en punkt på sprickplanet som sprickvidden är statistisk korrelerad med sprickvidden i punkten. Korrelationsavståndet kan användas för att ytterligare karaktärisera spricköppningsgeometrin (se Hakami 1995 [7]), och var i detta fall 5 - 20 mm för samtliga delområden.

## SLUTSATSER

Den metod och utrustning som utvecklats för strömningsförsök och sprickviddsbestämning på naturliga bergsprickor har visat sig fungera tillfredsställande. Det nya med metoden är att den kan ge såväl hydrauliska som mekaniska sprickparametrar och dessutom detaljerad information om spricköppningens geometri för ett och samma sprickprov.

Metoden har tillämpats på en spricka i granit. Medelsprickvidden för denna spricka var 360  $\mu\text{m}$  med en standardavvikelse på 150  $\mu\text{m}$  vid 0.45 MPa normalspänning. Mindre än 5% av sprickytan hade en sprickvidd mindre än 50  $\mu\text{m}$ . Korrelationsavståndet för den spatiella fördelningen av sprickvidden var ca 1 cm. Medelsprickvidden var 1.4 gånger större än den hydrauliska sprickvidden beräknad från resultaten från flödestestet, vilket stämmer överens med resultat som erhållits i andra studier av bergsprickor.

## SUMMARY

The aim of the experimental work presented in the paper has been to carry out flow experiments and aperture measurements on the same specimen of a single natural fracture, in order to compare measured flow with predicted flow based on geometrical description of the fracture void space.

A core was drilled parallel to a natural fracture in crystalline rock. The diameter of the core was 190 mm and it was about 415 mm long. The sample was placed in a biaxial cell and a constant water head was supplied to the lower end of the cell. The flow through the fracture was measured from the overflow at the top for different confining pressures and water heads. The hydraulic aperture,  $e$ , decreased from 285  $\mu\text{m}$  to 245  $\mu\text{m}$  when the confining pressure was increased from 0.3 MPa to 1.2 MPa. In terms of compressibility this corresponds to a  $\kappa = 2 \cdot 10^{-6} [\text{Pa}^{-1}]$ .

A technique to measure the aperture was developed utilizing injection of fluorescent epoxy in the fracture specimen. An image analysis system was used to take measurements along sections across the fracture surface. The method was successfully applied to a fracture in granite having a mean aperture of 360  $\mu\text{m}$  at 0.45 MPa normal stress. The spatial correlation of the aperture was about one centimeter. The mean of the measured aperture was 1.4 times larger than the hydraulic aperture. This is a ratio of the same order as in other studies.

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## **BERGINJEKTERING VID SVÅRA FÖRHÅLLANDEN MED TVÅKOMPLEMENTSPOLYURETAN.**

**Rock grouting in difficult conditions with two component polyurethane.**

*Peter Jonsson, Stabilator AB*

*Martin Brantberger, Stabilator AB*

### **BAKGRUND**

Vid underjordsbyggande kan dåliga bergförhållanden, t ex stora vattenflöden och höga vattentryck ge problem under drift samt i den permanenta konstruktionen. En av åtgärderna för att minska problemen är normalt en systematisk förinjektering. Denna injektering kan vid svåra förhållanden bli tidsödande med många ominjekteringar och / eller efterinjekteringar som följd.

En del av Stabilators utvecklingsarbete har inriktat sig på att ta fram metoder för att på ett effektivt sätt kunna täta bergpartier med svåra förhållanden och därmed minska behovet av ominjekteringar och efterinjekteringar. Ett system, som för närvarande studeras, är baserat på ett tvåkomponentspolyuretan, Bevedan / Bevedol. Produkten tillverkas av det tyska företaget Carbo Tech. Företaget Carbo Tech har sitt ursprung i den tyska kolindustrin och dess produkter har utvecklats genom att svåra stabilitets - och tätningsproblem ofta uppträder i tyska kolgruvor. Polyuretanet har provats av Stabilator på ett tunnelbygge på Island och Hallandsåsen i Sverige.

### **SYFTE**

Syftet med rapporten är att beskriva polyuretansystemets funktion samt återge erfarenheter från några utförda injekteringar.

## **INJEKTERINGSSYSTEMET**

### **Funktion**

Bevedan / Bevedol är ett tvåkomponentspolyuretan som under en längre tid har använts framför allt i gruvor men även i tunnelarbeten i bl a Tyskland och Schweiz. Vid tunnelarbeten används produkten inom följande områden.

- För - och efterinjektering
- Stabilisering av berg
- Injektering av bultar

### **Injekteringsmedel**

Injekteringsmedlet består av två komponenter, Bevedan och Bevedol. Bevedan är grundkomponenten och Bevedol är en komponent, som finns i olika typer. Beroende på valet av Bevedol erhålles ett polyuretan med olika egenskaper t ex viskositet, härdningstid, volymökning, hållfasthet mm.

När de två komponenterna blandas reagerar de med eller utan vatten. Vid reaktionen med vatten sker en uppskumning och volymen ökar, varvid en fast, skumliknande struktur bildas. Den vattenoberoende reaktionen ger en hård och tät slutprodukt. För tätning av berg med stora vattenflöden och höga vattentryck är Bevedol WFA eller Bevedol WF lämpliga. Reaktionen startar efter ca 0,75 respektive 1,5 minuter och efter ca 5 respektive 15 minuter är produkten genomhärdad. Vid reaktionen mellan de två komponenterna frigörs koldioxid, vilket ger ett gastryck, som kan förbättra inträngningsförmågan.

Urlakningstester i avjoniserat vatten, utförda av CBI, visar att mängden urlakade ämnen understiger svenska gränsvärden för dricksvatten. Polyuretanet är också resistent mot sulfater, syror och baser enligt tyska undersökningar .

### **Utrustning**

Injekteringsutrustningen består av en luftdriven tvåkomponentspump. De två pumpenheterna drivs av en gemensam cylinder, vilket säkerställer en exakt volymproportionering ( 1:1 ) oavsett slagfrekvens. Komponenterna sugas separat upp från dunkar eller fat och pumpas åtskilda till ett rör, i vilket de blandas med en statisk mixer, som är monterad i röret. Röret i sin tur är kopplat till en engångsmanschett, vilken blåses upp med injekteringstrycket. När injekteringen i ett hål är klart, kopplas röret bort från

manschetten och används till nästa hål. För att få in injekteringsmedlet så nära läget för vattenläckage som möjligt kan manschetten förlängas med plaströr.

## **ERFARENHETER FRÅN UTFÖRDA TUNNELPROJEKT**

### **Botnsdalur - och Breidadalurtunnlarna, Island**

#### **Bakgrund**

Botnsdalur - och Breidadalurtunnlarna ingår i ett vägtunnelprojekt på Nordvästra Island. Tunnlarna, vars totala längd är 8,7 km, är belägna i basaltisk lava. Basalten var normalt av god kvalitet men ofta kraftigt uppsprucken med kraftiga vattenflöden från lokala akvifärer eller ytan.

#### **Utförande**

##### **Botnsdalur**

I Botnsdalurtunneln var bergmassan av god kvalitet, emellertid var den ofta kraftigt uppsprucken med stora vattenflöden. I en del av tunneln var vatteninflödena så kraftiga, att fortsatt tunneldrivning inte kunde ske. Vattenflödena var upp till 150 - 180 liter / sekund vid ett tryck på 10 - 12 bar. Den stora uppgiften blev att skapa kontrollerade förhållanden avseende vatteninflödena och att tätat området för att möjliggöra fortsatt tunneldrivning med systematisk förinjektering med cement. Försök att tätat berget med enbart cement misslyckades på grund av att injekteringsbruket spolades ut av det kraftiga vattenflödet.

Tättningsarbetena utfördes som en omfattande förinjektering uppdelad i nio steg med både polyuretan och cement. Vid polyuretaninjekteringarna användes Bevedol WFA, vilket ger en mycket snabbreagerande produkt, vilket var lämpligt vid de mycket kraftiga vattenflödena. Den preliminära planen att utföra systematisk borrarning och injektering övergavs. Istället utfördes selektiv borrarning med ett förutbestämt längsta håldjup eller tills dess att vatten påträffades. Detta gjordes för att minimera antalet borrhål och på så sätt få bättre kontroll över injekteringen. Vid polyuretaninjekteringen var borrhålen mellan 8 - 10 meter långa och trycket som applicerades ca 200 bar. Vid cementinjekteringarna förlängdes vissa av de redan injekterade hålen genom omborrarning i tremetersintervaller upp till 24 meter.

Efter att ha avslutat injekteringsinsatsen och borrade sonderingshål visade på små vattenmängder, drevs tunneln vidare två salvor med mellanliggande sonderingsborrning. Tunneln var under dessa salvor i det närmaste torr. En ny sondering gav emellertid stora vatteninflöden i den vänstra väggen trots att man befann sig i redan injekterat område. Detta indikerade att stora vattenflöden även kunde förekomma lokalt, vilket satte stora krav på borrhålsorienteringen. En ny injekteringsomgång med cement utfördes på liknande sätt som i den första omgången. Efter denna omgång kunde tunneln drivas vidare enligt den ursprungliga metoden.

### Breidadalur

I Breidadalurtunneln påträffades en förkastningszon med dålig bergkvalitet. Vattenflödena var inte så stora som i Botnsdalur, men i kombination med den dåliga bergkvaliteten var risken för kollaps stor.

Innan injekteringsarbetena startade hade området runt tunnelfronten förstärkts med fiberarmerad sprutbetong och bultar. Injekteringen utfördes sedan på samma sätt som i Botnsdalurtunneln med omväxlande polyuretan och cement. Trycket vid polyuretaninjekteringen minskades till ca 150 bar på grund av den dåliga bergkvaliteten. Efter nio dagars injektering var området stabiliserat, så att fortsatt tunneldrivning kunde ske.

### Hallandsåsen, Sverige

#### **Bakgrund**

Tunneln genom Hallandsåsen blir vid färdigställandet en ca 8 km lång järnvägspassage med två enkelspårstunnlar. Bergmassan består till stor del av gnejs med inslag av amfibolit och diabasgångar. Amfiboliten, som är av mer eller mindre skiffrig karaktär, uppträder vanligtvis som gångar, linser eller oregelbundna sliror i gnejsmassan. Ofta är sprickorna i bergmassan fyllda på grund av ler - och kloritomvandling.

De ordinarie tätningsarbetena har varit omfattande på grund av svåra bergförhållanden. Många gånger har vatteninflödena varit stora, men vid förinjekteringarna var det svårt att få en tillräcklig utbredning på injekteringsmedlen och därmed uppnå en god tätningseffekt. I praktiken har detta inneburit många ominjekteringar innan en ny salva kunde skjutas. Många olika typer av injekteringsmedel har provats utan någon större effekt.

Stabilator har därför av Banverket fått möjlighet att prova en annan injekteringsmetod med ambitionen att minska ominjekteringsbehovet. Det injekteringsmedel som provades var Bevedan / Bevedol. Provinjekteringarna utfördes i tre sektioner, en på södra sidan och två på norra sidan.

## Utförande

Vid injekteringarna i Hallandsåsen användes Bevedol WF. Bevedol WF ger ett snabbreagerande polyuretan, som är lämpligt vid tätning i jord och berg med stora vattenflöden. Polyuretanet blir dock inte lika snabbt som vid användning av Bevedol WFA. Alla injekteringarna utfördes enligt samma metod, dock med varierande sluttryck. Antalet hål, hållängden och hålplaceringen var den samma som vid de ordinarie injekteringarna. I alla hål placerades manschetten 1 - 1,5 m innanför hålmynningen. På grund av blandningens snabba reaktionstid krävdes en stor pumpkapacitet med höga injekteringstryck. Pumptrycket som applicerades varierade vid de olika injekteringarna mellan 90 och 150 bar. Detta tryck minskar emellertid med ca 50 bar på grund av motstånd i pumpsystemet, vilket ger ett effektivt tryck i hålet på ca 40 - 100 bar.

### Södra sidan, östra tunneln 198 / 380

Injekteringarna i de två föregående sektionerna hade utförts i två omgångar med cementbaserat injekteringsmedel. Vattenflödena i dessa sektioner var mellan 40 och 90 liter / minut, stuff och koncentrerade till botten av stuffen.

Polyuretaninjekteringen i den nya sektionen utfördes i 18 hål, 14 m långa och med borrhålsdiameter 64 mm. Vattenläckaget ur injekteringshålen var koncentrerade till det nedre vänstra hörnet och vänster vägg. Totalt vattenflöde från stuffen var ca 150 liter / minut, av vilket 130 liter / minut kom ur två stycken hål. Injekteringarna utfördes med ca 90 bars sluttryck på pumpen.

Totalt pumpades 1150 liter in i berget, av vilka 720 liter i zonen med vattenflöden ur injekteringshålen. En viss spridning av injekteringsmedel till stuffen upptäcktes under injekteringen.

Vid injekteringen noterades problem med att få packern att sitta kvar i hålet, vilket framför allt berodde på dåligt berg i manschettläget.

Kontrollhål borrade ett i botten och ett i respektive vägg gav endast ett vattenflöde på totalt ca 2 liter / minut. Stuffen behövde inte ominjekteras och en ny salva togs ut direkt.

#### Norra sidan, västra tunneln 191 / 086

Polyuretaninjekteringen utfördes i en sektion en salva efter den föregående injekteringsomgången. Den förra injekteringen bestod av fem stycken injekteringsskärmar utförda med cement. Vattenflödet var från början totalt 472 liter / minut och ganska jämnt fördelat över stuffen. Vid den första ominjekteringen minskade flödet till 124 liter / minut men därefter gjordes två injekteringsskärmar utan något mätbart tätningsresultat. Vid den femte injekteringen sänktes vattenflödet till 20 liter / minut och en ny salva fick skjutas.

Polyuretaninjekteringen i den nya sektionen utfördes i 18 hål, 15 m långa och med borrhålsdiameter 51 mm. Vattenläckagen ur injekteringshålen var mellan 5 och 30 liter / minut, vilka var ganska jämnt fördelade över stuffen. Totalt vattenflöde var ca 220 liter / minut. Det läckte också ganska mycket vatten ur sprickor över hela stuffen. I denna sektion noterades en tydlig uppsprickning i decimetertjocka skivor, som övertvärade tunneln. Sluttrycket sattes till ca 100 bar och totalt pumpades 1330 liter in i berget.

Vattenläckaget ur kontrollhålen var ca 20 liter / min, hål. En ny skärm borrades upp, vilket gav ett sammanlagt vattenflöde ur injekteringshålen på ca 110 liter / minut. Vatteninflödet hade därmed minskat till hälften. De nya injekteringshålen injekterades med cement av huvudentreprenören.

#### Norra sidan, västra tunneln 191 / 121

Vid denna polyuretaninjektering beställdes en injektering inklusive eventuell erforderad ominjektering. Injekteringen utfördes i en oinjekterad zon i 18 plus 18 hål, 15 m långa och med borrhålsdiameter 51 mm. I den andra omgången injekterades också 4 stuffhål.

Vattenflödet ur injekteringshålen i den första omgången var 879 liter / minut av vilket ca 600 liter / minut kom ur takhålen ( ca 150 liter / minut, hål ). Även stuffläckage i den nedre vänstra delen noterades. Sprickstrukturen hade samma utseende som i sektion 191 / 086. Stopptrycket ökades vid dessa injekteringar till ca 150 bar för att om möjligt få en bättre inträngning.

Totalt pumpades 1320 liter in i berget i den första omgången och 1585 liter i den andra omgången. Materialåtgången var ganska jämnt fördelad över stuffen dock med något större mängder i tak - och stuffhål.

Resultatet av kontrollhålen efter första omgången medförde att en ny skärm var tvungen att borrar upp. De kraftiga vattenläckagen från takhålen hade då endast minskat marginellt. Efter ytterligare en injekteringsomgång med polyuretan borrades kontrollhål

på nytt. Även denna gång erhöles ett resultat som krävde att en ny skärm borrades. Tätningseffekten var nu något större i den vänstra delen av taket, där flödet hade minskat till ca 50 - 100 liter / min, hål. I den högra sidan av taket noterades ingen mätbar tätningseffekt. Den nedre vänstra delen av stoffen torkade emellertid upp märkbart. De nya injekteringshålarna injekterades med cement av huvudentreprenören.

## **Kommentarer**

En god tätningseffekt uppnåddes i det vattenförande partiet i sektionen på den södra sidan. Att en sämre tätningseffekt uppnåddes på den norra sidan, kan bero på en eller flera av följande faktorer.

- Spricksystemet: Sprickfyllnadsgraden var något större på den norra sidan, vilket kan innebära att det var svårt att få förbindelse mellan olika delar av spricksystemet. Detta indikerades även vid cementinjekteringarna i de polyuretaninjekterade områdena. På den norra sidan krävdes fler ominjekteringar än på den södra sidan.

- Reaktionstiden: En snabb reaktionstid ger en begränsad inträngning, speciellt vid användning av långa hål på grund av en längre hålfyllnadstid.

- Hålavstånden: En snabb reaktionstid i kombination med långa hål och långa hålavstånd ger en begränsad tätningseffekt.

## **SLUTSATSER**

Vid injektering i svåra bergförhållanden, t ex stora vattenflöden och höga vattentryck, är tvåkomponentspolyuretaner av typen Bevedan / Bevedol ett alternativ eller komplement till andra typer av injekteringsmedel. Vid polyuretaninjektering bör emellertid följande parametrar beaktas.

Hållängden: Långa hål tar relativt lång tid att fylla. Vid injektering med snabbreagerande polyuretaner, Bevedan / Bevedol WF och Bevedan / Bevedol WFA, kan polyuretanet ha hunnit för långt i härdningsprocessen innan den egentliga inträngningen i bergmassan, vilket begränsar spridningen i spricksystemet. Kortare borrhål är ur denna aspekt att föredra. Vid mycket stora vattenflöden bör korta hål kombineras med användande av Bevedol WFA. Om förutsättningarna i övrigt tillåter, kan man med rätt kombination Bevedan / Bevedol injektera även längre hål vid en förinjektering. Långa hål är normalt att föredra för att erhålla en effektiv drivningscykel.

Håldiametern: Borrhålsdiameter 51 mm är att föredra eftersom detta ger kortare hålfyllnadstider, mindre materialåtgång och en bättre anliggning mellan packer och berg.

**Hålavståndet:** Vid problem med inträngningen kan ett kortare hålavstånd vara lämpligt för att möjliggöra en god tätningseffekt.

**Hålorienteringen:** Hålorienteringen bör anpassas för att få kontakt med de vattenförande zonerna.

**Bevedoltyp:** Rätt kombination av Bevedan och Bevedoltyp måste tillämpas för att erhålla de rätta egenskaperna hos polyuretanet vid olika applikationer.

För att effektivt kunna utnyttja produktens positiva egenskaper krävs att man får möjlighet att genom förprov göra nödvändiga justeringar för optimering av reaktionstid, hållängd, hålplacering, pumptryck mm.

## **REFERENSER**

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## **ABSTRACT**

In tunnel constructions difficult rock conditions, such as heavy inflow of water and high water pressures, can cause severe problems during excavation and in the permanent construction. In order to solve these problems, normally a systematic pregrouting campaign is executed. In difficult conditions these groutings can be time-consuming with many regroutings as a result.

One method, which at present is studied by Stabilator in order to decrease the number of regroutings, is the two component polyurethane system Bevedan / Bevedol, manufactured by the German company Carbo Tech. The system has been applied by Stabilator at tunnel projects in Iceland and in Hallandsåsen, Sweden.

The polyurethane is mostly used for rock stabilization, bolting, sealing against gas and water and rehabilitating of tunnel structures. The system consists of the two components Bevedan, which is the basematerial and different types of Bevedol, depending on the required setting time and foaming factor. The components react with each other after blending in the static mixer of a feedpipe just before passing the packer. The reaction starts with or without water. When reacting with water the material foams and CO<sub>2</sub> is liberated. The reaction without water results in a dense and hard product. For sealing

works in tunnels with heavy inflow of water and high water pressures the Bevedan / Bevedol WFA or the Bevedan / Bevedol WF system is preferable. These polyurethanes have a reaction time of approximately 0,75 and 1,5 minutes respectively. The equipment needed for the system is a pneumatically driven two component pump, which delivers the two components at the volumetric ratio of 1:1.

Experiences from tunnel projects show that a good result can be achieved in difficult rock conditions. To reach this effect it is however important to carefully choose the right combination of Bevedan and Bevedol, and if necessary combine it with other types of grout materials. Depending on the choice of grout material it is also important to adjust the length and the orientation of the boreholes.



## **DRÄNERS FUNKTION VID OLIKA DRIFTSFÖRHÅLLANDEN - RESULTAT FRÅN TEST AV DRÄNER I TUNNEL VID HORNSBERG, STOCKHOLM**

The behaviour of drains under different working conditions. Results from a field test in a tunnel at Hornsberg, Stockholm

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Ulrika Hamberg, Vägverket Region Stockholm  
Jan Johansson, Naturgasteknik AB

### **INLEDNING**

Dennisöverenskommelsen omfattar inom Vägverkets ansvarsområde vägsystemen Ringen och Yttre Tvärleden. Ringen innebär att ca 12 km väg skall förläggas i tunnel runt Stockholms innerstadsområde. Den totala tunnellängden blir drygt 40 km med hänsyn till att Ringen har dubbla tunnelrör med enkelriktad trafik och ett stort antal underjordiska trafikplatser med på- och avfartsramper. Även Yttre Tvärleden innehåller omfattande vägsträckor i tunnlar runt några av de västra och norra förorterna.

Konsekvenserna vid felbeslut gällande till exempel lösningar av konstruktionsdetaljer kan bli mycket kostsamma både i bygg- och i driftskedet till följd av de långa tunnelsträckorna. För att så långt som möjligt undvika felbeslut har Vägverket region Stockholm, projekt Ringen och Yttre Tvärleden intentionen att utreda och i viss mån testa konstruktionslösningar i de fall tveksamhet föreligger för deras funktion.

Trots omsorgsfull tätning vid tunneldrivning i berg letar sig små mängder vatten alltid in i tunneln. Detta vatten kan under vintern frysa till is och orsaka skador. För att undvika detta appliceras i de flesta fall isolerade dräner på bergtak och väggar för att samla upp vattnet och frostfritt leda bort det.

Att dräner inte fungerar är ett känt och inte ovanligt problem i bergtunnlar. Dränerna läcker, sätter igen, vibrerar loss etc. Eftersom det är ett uttalat krav att Ringens tunnlar ska ha en hög tillgänglighet för trafikanterna kan man ej acceptera återkommande lappning och lagning av dränerna.

Det finns nära nog inget skrivet eller undersökt om dräners funktion eller drifterfarenheter. Tillfrågade anläggningsägare konstaterar dock att dräner i kalla tunnlar nästan alltid innebär problem.

För att i möjligaste mån undvika problem med dräner i Ringens tunnlar installeras innertak över körbanorna och dränsättningen begränsas till tunnlarnas väggar. Osäkerheten om rätt utformning av väggdräner har utretts i ett omfattande försök under sommar och höst 1995 samt vintern 1996.

## FÖRSÖKENS SYFTE

Det övergripande syftet för dränagesförsöken har varit att jämföra fem olika dräntyper (typ A - E) med avseende på:

- byggbarhet och ekonomi
- dräneringsfunktion
- driftsäkerhet (m.h.t. störningar i form av frost, igensättning, dynamisk last)
- underhåll (möjligheter och behov)

Försöken har utförts under så realistiska former som möjligt. Dränerna installerades under juni 1995 av BESAB AB. Försöket kommer att avslutas under vintern 1996 och resultatet kommer att utgöra en viktig del i beslutsunderlaget vid val av dränagesystem för RYT-tunnlarna. I denna artikel redovisas några delresultat av den hitintills utförda provningen.

## STUDERADE DRÄNTYPER

De fem testade dräntyperna är egentligen ganska lika. De innehåller samma primära funktioner:

- Skapande av spalt
- Vattenavskärmning
- Frostskydd
- Lastupptagning

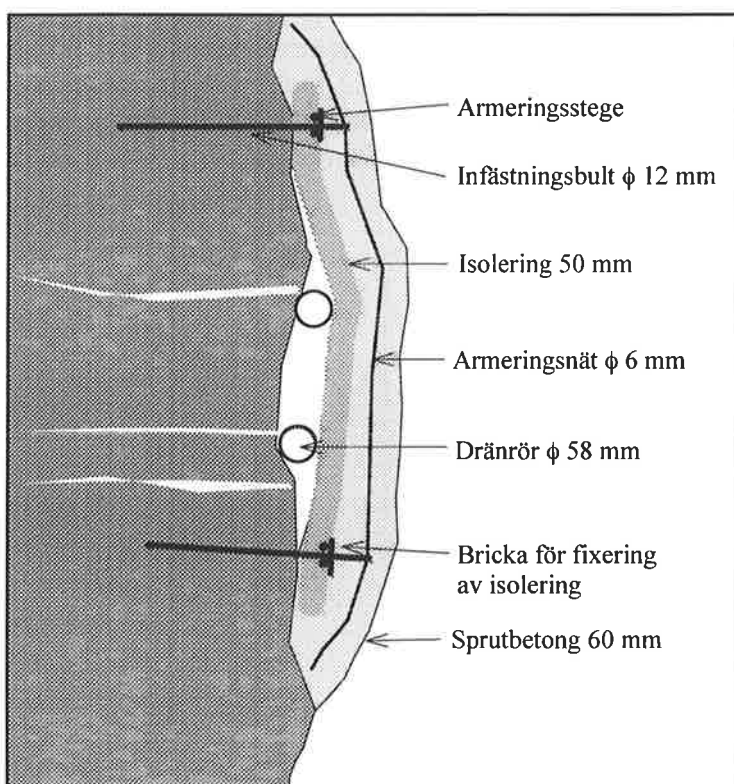
Spalten skapas med hjälp av rör (typ A,B,E), nabbar (typ C) eller urfrästa spår (typ D). Vattenavskärmningen skapas med hjälp av expanderad polyetylenisolering i samtliga fall utom typ C, där funktionen är dubblerad med en platonmatta. Frostskyddet baseras på polyetylenens isolerande förmåga. Typ B kompletteras med värmekabel. Lastupptagningen sker för dräntyp A-D med hjälp av nätarmerad sprutbetong infäst i kraftiga bergbultar. För typ E används fiberarmerad sprutbetong och vidhäftning mot berget.

Det är redan här värt att notera att det med den aktuella konstruktionen finns skillnader mellan dräntyperna avseende till vilken maximal bredd dränen kan byggas. Dräntyperna A, C och D kan med utnyttjande av det komponentsystem som använts göras oändligt bred. Dräntyp B är med sin tunna isolering inte lämplig för större spännvidder mellan infästningpunkterna (syftet med denna dräntyp är att ta hand om enstaka punktläckage). Dräntyp E måste vid större bredder kompletteras med lastbärande förankringar i berget då lastupptagningen genom vidhäftning utmed ränderna blir otillräcklig. Dräntyp E närmar sig alltså vid större bredder utformningsmässigt typ D fast med fiberarmerad sprutbetong i stället för nätarmerad.

## Typ A, Insprutad isolerad drän med dränagerör

Utformningen av dräntyp A "Insprutad isolerad drän med dränagerör" illustreras i Figur 1. Dränen kan utföras med varierande bredd, i försöksanläggningen monteras dränen i det smalaste utförandet 1500 mm.

Dränen består av ett eller flera distansbildande dränagerör,  $\varnothing 58/50$  mm, som monteras vertikalt mot berget över det läckande avsnittet. Som kombinerad vattentätning och isolering används en 50 mm tjock matta av expanderad polyetylen. Isoleringsmattan har slutna celler och låg densitet vilket innebär låg vattenabsorption och god isoleringsförmåga. Dränen kläs med ett 60 mm tjockt skikt av sprutbetong, centriskt armerad med ett armeringsnät ( $\varnothing 6$  #150). Armeringsnätet förses med distanser för att hålla rätt avstånd till mattan.



Figur 1      Illustration av dräntyp A. Horisontalsnitt.  
*Illustration of drain type A. Horizontal section.*

Infästningen av hela dränen till berget sker med  $\varnothing 12/400$  kamstålsbult c/c minimum 1200 mm som förankras med polyesterpatroner i borrhålet. Bultarna är gängade i ytterändan och förses med speciella frontbrickor som fixerar isoleringen mot berget. Även sprutbetongens armering fixeras till bultarna med hjälp av ett speciellt fästbeslag. Vid dränens vertikala ränder sammanbinds bultraden med en armeringsstege  $2 \cdot \varnothing 10$  mm som

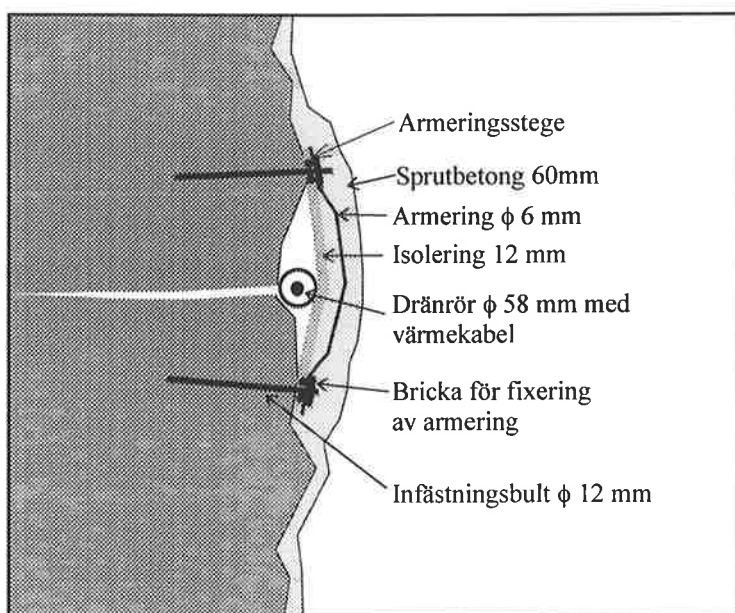
har till uppgift att pressa isoleringsmattan mot berget. Alla infästningsdetaljer är standardkomponenter.

Dränen har gjorts åtkomlig för inspektion och rensning genom att dränrören försetts med rensningsanordningar i över- och underkant. I provanläggningen har dränen försetts med några extra inspektionsanordningar, bland annat för inspektion av utrymmet vid sidan om och mellan dränrören.

### Typ B, Insprutad isolerad drän med dränagerör och värmekabel

Utformningen av dräntyp B "Insprutad isolerad drän med dränagerör och värmekabel" illustreras i Figur 2). Dränen är främst tänkt för punktläckage med begränsad bredd, i försöksanläggningen utförs dränen med bredden 600 mm.

Dränelternativet består av ett distansbildande dränagerör  $\varnothing 58/50$  mm som monteras vertikalt mot berget över det läckande avsnittet. I dränagerörret placeras en självbegränsande värmekabel. Som kombinerad vattentätning och isolering används en 12 mm matta av expanderad polyetylen försett med reveteringsnät. Dränen kläs med ett 60 mm tjockt skikt av sprutbetong, centriskt armerad med ett armeringsnät ( $\varnothing 6$  #150). Armeringsnätet förses med distanser för att hålla rätt avstånd till mattan.



Figur 2 Illustration av dräntyp B. Horisontalsnitt.  
*Illustration of drain type B. Horizontal section.*

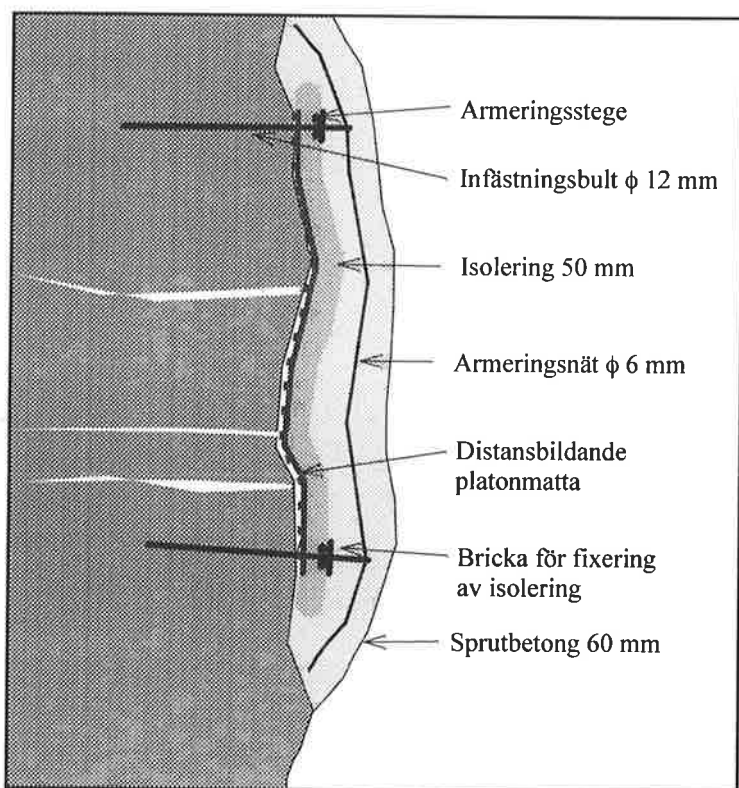
Infästningen av hela dränen till berget sker på samma sätt som drän A.

Dränen har gjorts åtkomlig för inspektion och rensning genom att dränrören försetts med rensningsanordningar i över- och underkant. I provanläggningen har dränen försetts

med några extra inspektionsanordningar, bland annat för inspektion av utrymmet vid sidan om och mellan dränrören.

### Typ C, Insprutad isolerad drän med platonmatta

Utformningen av dräntyp C "Insprutad isolerad drän med platonmatta" illustreras i Figur 3. Dränen kan utföras med varierande bredd, i försöksanläggningen monteras dränen i det smalaste utförandet 1500 mm. Dränalternativet består av en distansbildande plastmatta typ Platon Tunnel-matta. Mattan monteras med "nabbarna" vända mot berget över det läckande avsnittet. Utanför plastmattan monteras en 50 mm tjock isoleringsmatta av expanderad polyetylen. Dränen kläs med ett 60 mm tjockt skikt av sprutbetong, centriskt armerad med ett armeringsnät (ø6 #150). Armeringsnätet förses med distanser för att hålla rätt avstånd till mattan.



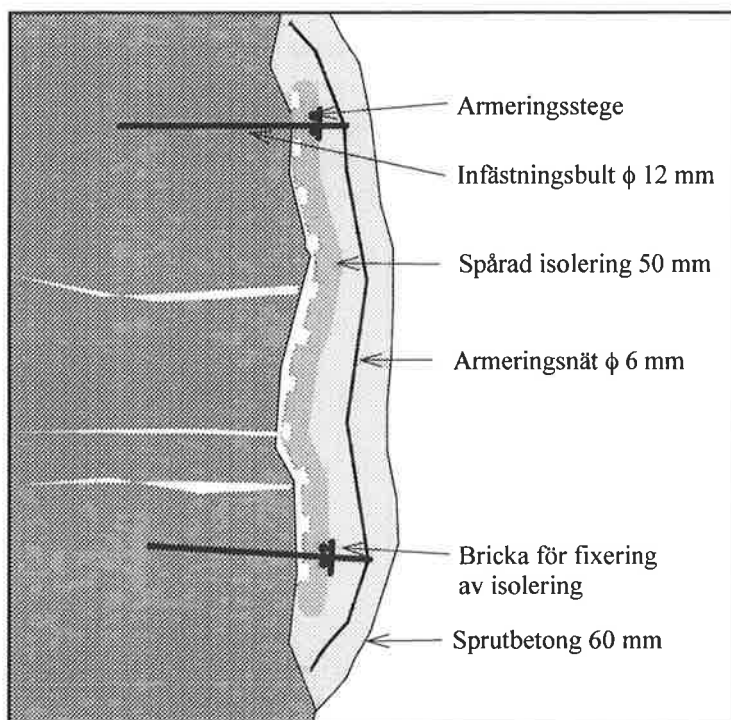
Figur 3 Illustration av dräntyp C. Horisontalsnitt.  
*Illustration of drain type C. Horizontal section.*

Infästningen av hela dränen till berget sker på samma sätt som drän typ A.

Dränen har gjorts åtkomlig för inspektion och rensning genom att dränen har försetts med inspektionsanordningar.

### Typ D, Insprutad isolerad drän med spårad isolering

Utformningen av dräntyp D ”Insprutad isolerad drän med spårad isolering” illustreras i Figur 4. Dränen kan utföras med varierande bredd, i försöksanläggningen monteras dränen i det smalaste utförandet 1500 mm.



Figur 4 Illustration av dräntyp D. Horisontalsnitt.  
*Illustration of drain type D. Horizontal section.*

Dränalternativet innehåller en kombinerad vattentätning, isolering och distansbildare i form av en 50 mm tjock matta av expanderad polyetylen med urfrästa spår. Isoleringsmattan har slutna celler och låg densitet vilket innebär låg vattenabsorption och god isoleringsförmåga. Mattan monteras med den spårade ytan mot berget (vertikala spår) över det läckande avsnittet. Dränen kläs med ett 60 mm tjockt skikt av sprutbetong, centriskt armerad med ett armeringsnät ( $\phi$  6 #150). Armeringsnätet förses med distanser för att hålla rätt avstånd till mattan.

Infästningen av hela dränen till berget sker på samma sätt som drän typ A.

Dränen har gjorts åtkomlig för inspektion och rensning genom att dränen har försetts med inspektionsanordningar.

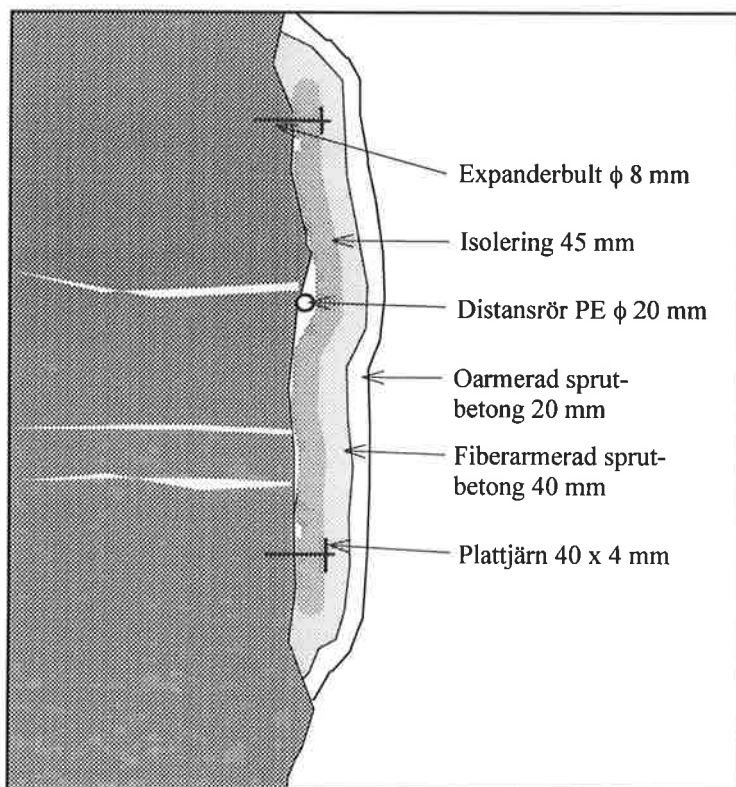
### Typ E, Insprutad isolerad drän, Banverksdrän

Utformningen av dräntyp E "Insprutad isolerad drän, Banverksdrän" illustreras i Figur 5. Dränen har använts av Banverket för bland annat Svealandsbanan, därav namnet. Dränen har maximalt utförts i en bredd av 2000 mm, i försöksanläggningen monteras dränen i en bredd av 1000 mm.

Dränalternativet består av en 45 mm tjock expanderad polyetylenisolering med två vertikala frästa spår 150 mm från kanterna vända mot berget. För att bilda distans mellan berget och isoleringen monteras en  $\varnothing 20$  mm slang av polyetylen över det läckande partiet. Dränen sprutas in med 40 mm fiberarmerad sprutbetong och 20 mm oarmerad sprutbetong.

Infästningen av isoleringen till berget sker med plattjärn 40•4 mm utefter de vertikala sidorna (även den horisontella överkanten). Plattjärnen fästs i berget med expanderbult  $\varnothing 8/165$  c/c minimum 400 mm. Sug- och tryckkrafterna på sprutbetongen överförs till sidoberget genom vidhäftning mellan sprutbetong och berg längs dränens sidor.

Dränen kan göras åtkomlig för inspektion och rensning genom att den förses med rensningsanordningar i över- och underkant. I försöksanläggningen förses dränen med några extra rensningsanordningar.



Figur 5 Illustration av dräntyp E. Horizontalsnitt.  
*Illustration of drain type E. Horizontal section.*

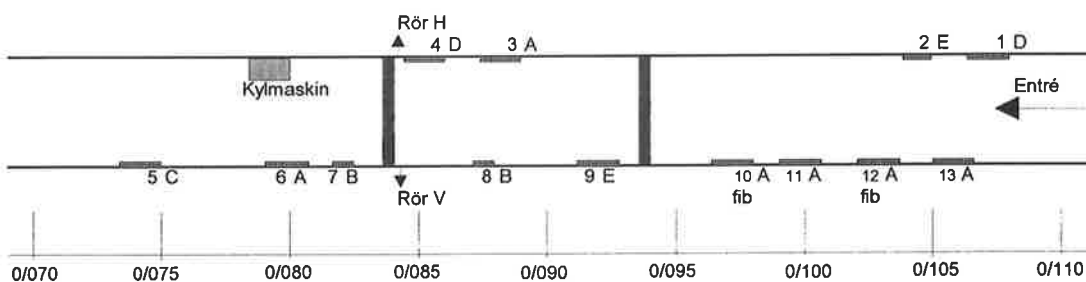
## PROVPLATS

Ett flertal tunnlar i Stockholmsområdet har under planeringsfasen granskats av Vägverket med avseende på lämpligheten att utföra fälttester på dräner. Dessutom har en viss inventering av kända problem med utfällningar utförts.

Det slutliga valet föll på en tunneln vid Hornsbergsstrand. Tunneln ägs av Stockholm Energi. Det främsta skälet till att förlägga försöken till denna tunnel var att tunneln erbjuder möjlighet till tester både med avseende på isbildning vid kyla (det finns gott om naturliga vatteninläckage) och med avseende på igensättningsfenomen (kraftiga utfällningar av bakterieslam förekommer). Även övriga förutsättningar är gynnsamma, bland annat tillgänglighet och försörjning med el och kommunalvatten. En del av tunneln har avgränsats så den kan kylas med ett kylaggregat, se figur 6. Placeringen av de olika dränera framgår av figur 6. Drän nr 10 - 13 har monterats för att utföra dynamiska tester på.

Vattenflödet genom varje drän har samlats upp i en låda monterad i dräns underkant. Lådan är försedd med ett utlopp där vattenflödet kan mätas med ett mätglas.

Bergtäckningen vid provplatsen är cirka 15 m.



### LEGEND

- A Dräntyp A, Insprutad isolerad drän med dränagerör
- B Dräntyp B, Insprutad isolerad drän med dränagerör och värmekabel
- C Dräntyp C, Insprutad isolerad drän med Platonmatta
- D Dräntyp D, Insprutad isolerad drän med spårad isolering
- E Dräntyp E, Insprutad isolerad drän, Banverksdrän
- Rör H Höger stuprör från taket
- Rör V Vänster stuprör från taket

Figur 6 Placering av dräner i tunnel vid Hornsberg  
*The location of the drains in the tunnel at Hornsberg.*

## PRODUKTIONSUPPFÖLJNING

Produktionsuppföljningen har syftat till att värdera de studerade dräntyperna med hänsyn till byggbarhet och kostnader.

Utvärderingen av dräntypernas byggbarhet har gett följande rangordning. (Det bör påpekas att rangordningen enbart baserar sig på montagemässiga bedömningar och alltså inte tar någon hänsyn till skillnader i funktion. En sådan värdering kan göras först när hela testprogrammet genomförts.)

1. Drän E Enklast att montera främst på grund av smidigare infästningsdetaljer och utförandet med fiberarmerad sprutbetong.
2. Drän D Enklast av de dräner som har ingjutna bultar och nätarmerad sprutbetong. Inga onödiga arbetsmoment.
3. Drän A Lika som D, men dränrören utgör extra arbetsmoment.
4. Drän C Lika som D, men platonmattan utgör extra arbetsmoment och har oklar funktion.
5. Drän B Den tunna isoleringen fungerar montagemässigt dåligt. Dränrör med värmekabel utgör extra arbetsmoment

Kostnadsjämförelsen mellan dräntyperna A-E har visat att dräntyp B har den lägsta kostnaden räknat per längdmeter drän (beror främst på att typ B har den minsta monterade bredden). Om priset i stället beräknas per kvadratmeter drän blir dräntyp D billigast.

## DYNAMISK PROVNING

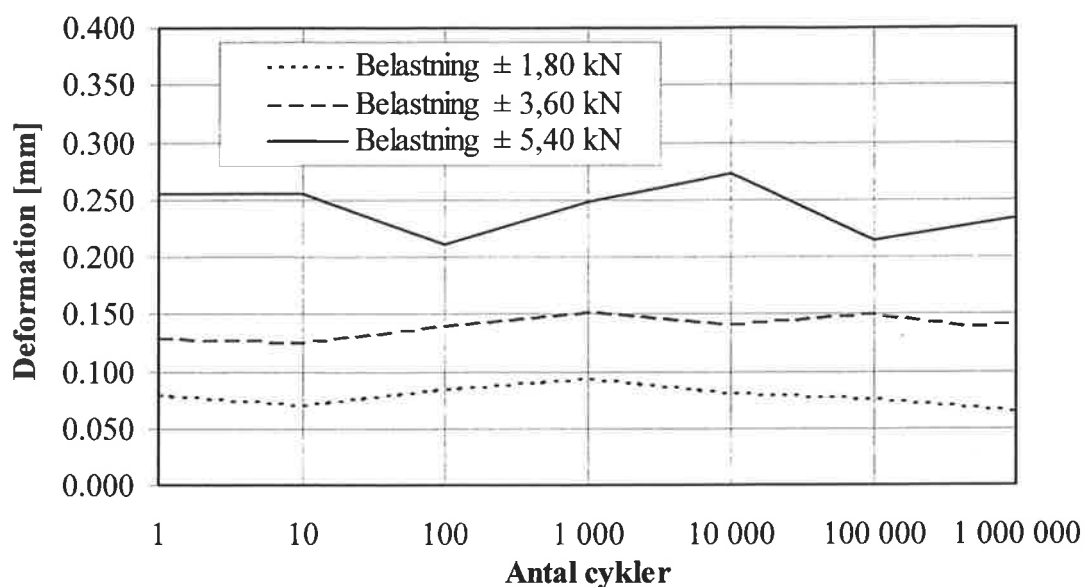
Lastöverföringen från sprutbetongen till berget sker för de studerade dräntyperna på två alternativa sätt. För drän typ A-D sker överföringen till berget med bultar till vilken sprutbetongen är förankrad medan för drän typ E sker överföringen till berget via vidhäftning mellan sprutbetong och berg längs dränens kanter. Sprutbetongen kan utformas på två sätt dels som nätarmerad sprutbetong, typ A-D, och dels som fiberarmerad sprutbetong, typ E.

Syftet med den dynamiska provningen har varit att se om det är någon skillnad i utmatningstålighet mellan nätarmerad sprutbetong och fiberarmerad sprutbetong samt mellan de två sätten att överföra lasten till berget.

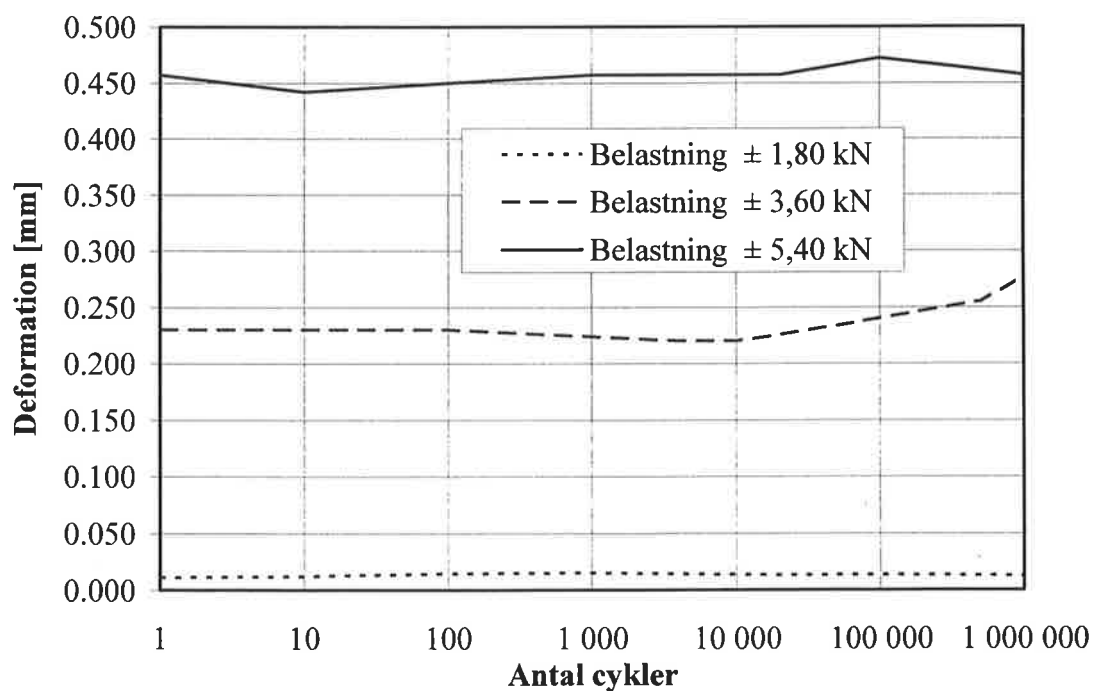
Den dynamiska belastningen på sprutbetongen har åstadkommits med en hydraulisk kolv som var infäst i en mothållsbalk. Mothållsbalken förankrades i berget utanför dräner. Mothållsbalken och hydraulkolven hängdes upp i en rörställning.

Mellan kolven och infästningsanordningen i sprutbetongen placerades en kraftgivare. Den hydrauliska kolven styrdes via en styrenhet och en hydraulpump. Belastningsamplituden hölls konstant med hjälp av styrenheten. Vid provningen har en belastningsfrekvens av 13 Hz använts. Efter ett antal förutbestämda belastningscykler mättes deformationsamplituden. Genom att plotta deformationsamplitudens storlek mot antalet belastningscykler kan ev utmatning av konstruktionen studeras.

Den dynamiska belastningen har alltså utgjorts av en punktlast i dränens mitt. Storleken på lasten har satts lika med arean mellan infästningsbultarna multiplicerat med 0,8 kPa



Figur 7 Deformationsamplitud som funktion av antalet belastningscykler vid belastning av drän med förankrad nätarmerad sprutbetong.  
*The amplitude of the deformation plotted against the number of loading cycles for a drain with shotcrete reinforced with a steelnet.*



Figur 8 Deformationsamplitud som funktion av antalet belastningscykler vid belastning av drän med fiberarmerad sprutbetong där lasten överförs via vidhäftning till berget..  
*The amplitude of the deformation plotted against the number of loading cycles for a drain with shotcrete reinforced with steel fibres.*

som är den dimensionerande belastningen som sug eller tryck. Lasten har varierats mellan tryck och drag. I en första belastningsomgång har dränen belastats med 1 miljon lastväxlingar med denna last därefter har lasten ökat i två steg till en fördubbling respektive tredubbling. Vid varje lastnivå har 1 miljon lastväxlingar utförts.

Resultatet av dessa belastningar på en drän med nätarmerad, förankrad sprutbetong redovisas i figur 7 och i figur 8 på en fiberarmerad sprutbetong där lasten överförs via vidhäftning till berget.

Det framgår av resultatet att det vid de studerade lastnivåerna inte är någon skillnad mellan konstruktionsalternativen fränsett att deformationerna är något större för dränen med den fiberarmerade sprutbetongen jämfört med den nätarmerade. Man kan inte spåra någon tendens till utmattning även vid en lastnivå som motsvarar tre gånger dimensionerande last.

## **DRÄNFUNKTION VID BAKTERIELLT BETINGAD IGENSÄTTNING**

Mikrobiell omvandling av järn mellan olika oxidationstal spelar en avgörande roll i naturen. Vanligen äger processerna rum i gränsområdet mellan syrerika och syrefattiga miljöer. Speciellt den syrefria delen är viktig eftersom det är där järnreduktion och järnmobilisering sker. Järnreducerande bakterier kan leva med eller utan syre och livnära sig på organiska föreningar. Dessa bakterier ger inga direkta utfällningsproblem i sig men skapar förutsättningar för de järnoxiderande bakterierna genom att öka mängden järn (II) i grundvattnet.

De järnoxiderande bakterierna verkar i gränsområdet mellan aerob och anaerob miljö där de söker en optimal kombination av järn (II) och syre. Järnoxidationen följs av en omedelbar utfällning av järn (III) i form av hydroxider och oxider, ofta tillsammans med bakterierna, stälkar, slem eller höljen. Man får då en voluminös massa som efterhand hårdnar till tämligen svåravlägsnade beläggningar. Precis som för järn finns det även ett stort antal bakterier som reducerar eller oxiderar manganföreningar. Processerna är likartade och kan även de leda till igensättningsproblem.

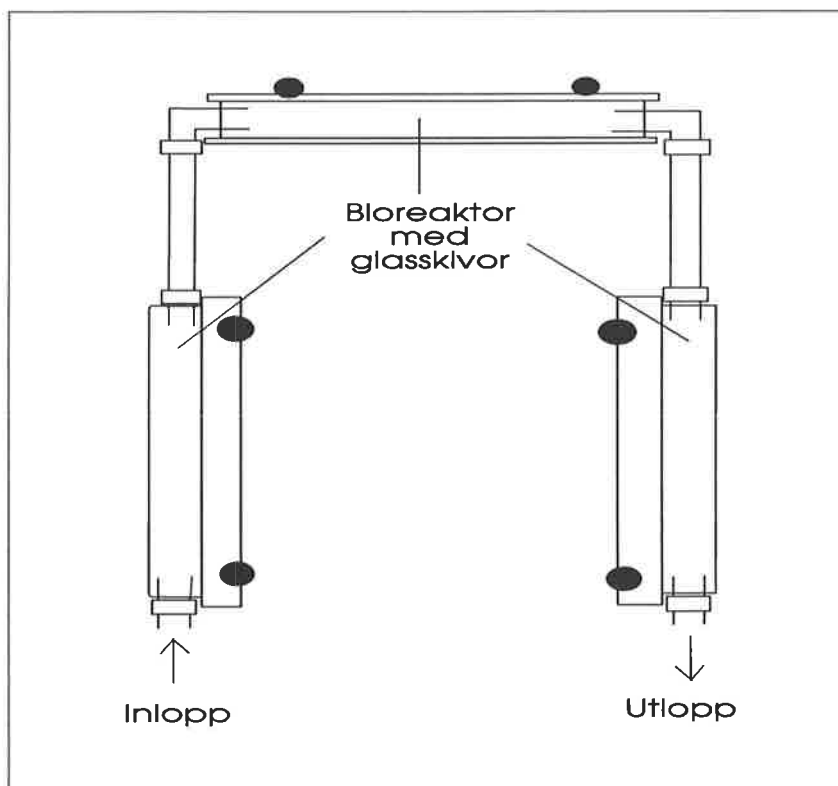
Avsikten med delförsöket var att studera bakteriellt betingad igensättning av dräner. Följande moment ingick i delstudien:

- Kemisk analys av grundvatten
- Provtagning av befintliga utfällningar i tunneln för att bestämma typ av bakterier
- Analys av biologisk tillväxthastighet med hjälp av monterade biofilmsreaktorer
- Observation och dokumentation av igensättningsförlopp i dräner
- Rensningsförsök

Grundvattenanalysen visade på relativt höga järnhalter, 22 - 46 mg/l. Även manganhalten och alkaniteten var högre än normalt. Proverna kan ha varit påverkade av de befintliga bakterieutfällningarna på bergväggarna samt av injekteringsbruk. Mikrosko-

peringen av prover från utfällningarna visade att dessa var starkt oxiderade och därför troligen gamla. Den biologiska aktiviteten var låg, endast på ett fåtal ställen (vid rinnande eller droppande vatten) hittades bakteriellstruktur, främst stjälgar från den järnoxiderande bakterien *Gallionella ferruginea*.

För att kunna bestämma tillväxthastighet och för att erhålla ostörda prover monterades tre paneler med biofilmsreaktorer kopplade till manschettförsedda borrhål. Vattnet från borrhålet leddes för varje panel genom de tre reaktorerna (se figur 9), som simulerar tre olika flödes- och syresättningstillstånd. Från reaktorerna togs efter en viss tids exponering prover ut för vidare analys på laboratorium. Tyvärr sinade vattnet vid panel 2 varför den uteslöts från analysen. Resultaten från analysen av den bakteriella tillväxthastigheten i biofilmsreaktorerna visas i Tabell 1. Docent Karsten Pedersen vid Göteborgs universitet, Avd. för allmän och marin mikrobiologi, har svarat för tillverkning av reaktorerna och för analys av proverna.



Figur 9 Testpanel med tre biofilmsreaktorer.  
*Testpanel with three biofilm reactors.*

Resultaten visar bland annat att flödes hastigheten inte är den mest avgörande faktorn för utfällnings hastigheten. Storleken på ytan som vattnet rinner över har däremot stor betydelse för mängden utfällning. Vattnets järninnehåll avgör längs hur lång sträcka som utfällningen sker. Viktiga miljöfaktorer (förutom järnhalt) är pH-värde, syretillgång och temperatur.

Tabell 1 Resultat från analys av bakteriell tillväxthastighet (viktangivelser avser torrsvikt).

|                    | Drifttid<br>(Dagar) | Flöde<br>(ml/min) | Flödeshastighet<br>(mm/s) | Utfälld mängd<br>(g/m <sup>2</sup> ) | Tillväxt<br>(g/m <sup>2</sup> ·vecka) | Tillväxt<br>(g/m <sup>2</sup> ·m <sup>3</sup> ) | Järnhalt<br>(%) |
|--------------------|---------------------|-------------------|---------------------------|--------------------------------------|---------------------------------------|-------------------------------------------------|-----------------|
| Panel 1, Reaktor 1 | 40                  | 26-77             | 1,7-4,9                   | 12                                   | 2,1                                   | 4,0                                             | 27,5            |
| Panel 3, Reaktor 1 | 21                  | 1-3               | 0,06-0,18                 | 2,2                                  | 0,73                                  | 36,7                                            | 22,7            |

Under hela försöksperioden i Hornsbergstunneln har igensättningsförloppet följts och dokumenterats genom inspektion med hjälp av fiberskop och videokamera. Dessutom har vattenflödena genom dränerna mätts regelbundet för att kontrollera att igensättningen inte gått så långt att flödet avstannar. Utfällningar av brunt bakterieslam kunde konstateras i dränerna redan efter kort tid, ca en månad. Tillväxthastigheten är mycket olika i de monterade dränerna. Dräner med kraftig utfällning återfinns i de områden där stora mängder utfällningsrester observerades på bergytan innan försöken påbörjades.

Utfällningar av betydelse (centimetertjocklekar) har konstaterats bland annat i dränerna 1, 4 och 7. Speciellt kraftiga är utfällningarna i drän 7. Dränen är av typ B och är försedd med dränagerör  $\phi 58$  mm. Utfällningarna har byggts upp inne i dränen så att det för ett horisontalsnitt så småningom bara var dränröret kvar för vattnet att rinna i. Senare växte utfällningarna till även inne i dränröret så att ca 30% av arean fylldes.

I detta läge beslöts att genomföra rensningsförsök i drän 1 och drän 7 med hjälp av högttrycksspolning. Spolning av drän 1 var svår eftersom dräntypen saknar spolrör. Det går därmed inte att styra spolmunstycket så att hela dränens längd kan genomspolas. Däremot var det betydligt lättare att spola dränröret i drän 7. Alla utfällningar inne i röret kunde enkelt avlägsnas och spolslangen kunde föras genom hela dränens längd. Spolningen hade däremot liten märkbar effekt på utfällningen utanför själva dränröret.

Slutsatserna från studien av bakteriellt betingad igensättning är bland annat att den biologiska aktiviteten, och därmed utfällningshastigheten, kan variera avsevärt mellan närliggande läckagepunkter. För områden och sprickor med "rätt" förutsättningar kan tillväxten av igensättningsprodukter ske snabbt och täta en drän inom loppet av sex månader. Vidare är det en förutsättning att ha dräner med spolrör för att rensning och spolning ska vara möjligt. Om allt järn inte fallit ut i dränen fortsätter utfällningarna i väggkroppen och dränageledningarna. I problemområden bör därför dränagesystemet utformas som ett slutet system för att möjliggöra rensning och för att undvika igensättning av väggkroppen.

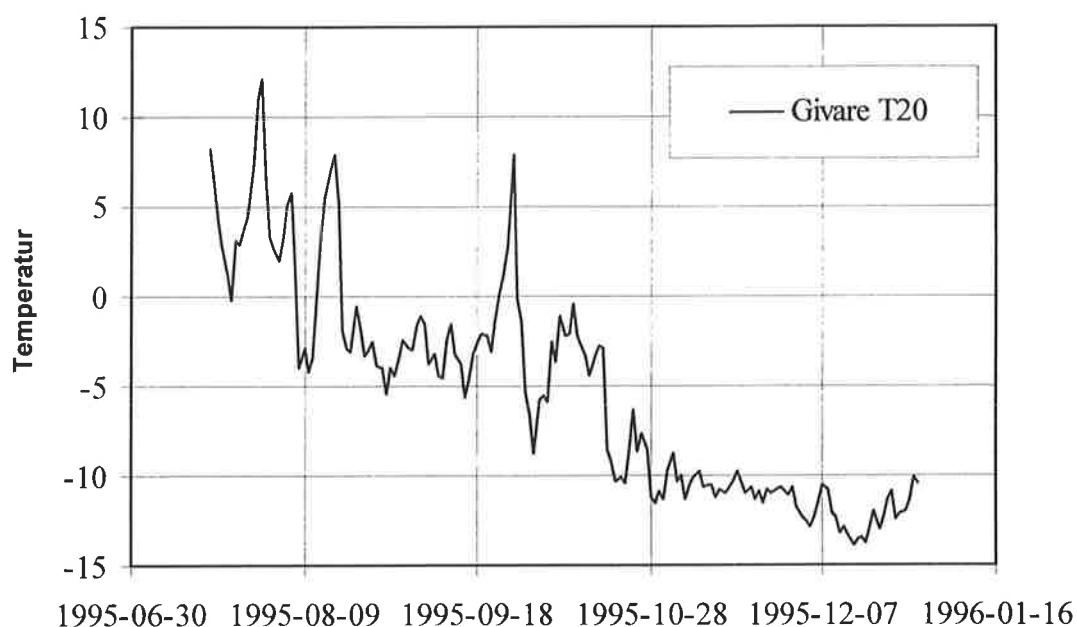
## DRÄNFUNKTION VID KYLA

I det kylda avsnittet har fyra olika dräntyper monterats (då typ C och typ D bedömdes vara likvärdiga till funktionen monterades endast typ D). Vidare finns ett isolerat inner-tak med hängrännor och två stuprör installerats i den kylda tunneldelen.

Provningsprogrammet för kylförsöket baserades på förhandsutgåva av TUNNEL 94 där dimensionerande köldmängd anges till 900 dygnsgrader för Stockholmsområdet och den lägsta temperaturen till  $-15^{\circ}\text{C}$ . Enligt TUNNEL 95 är dimensionerande maximiköldmängd ca 1100 dygnsgrader varför kylförsöket har förlängts mot de ursprungliga programmet.

Under successiv nedkylning av tunneldelen har temperaturutvecklingen i dräner, ovanför innertak och i bergmassan studerats. Istillväxten i dräner har studerats genom okulärbesiktning och videofilmning. I dränerna har besiktningen skett med hjälp av fiberskop som förs in genom rensnings- och inspektionsanordningarna. Mätning av vattenflöden från de enskilda dränerna har utförts med mätglas och klocka.

I figur 10 visas hur temperaturen har varierat i tunnelluften framför drän nr 8.



Figur 10 Temperaturvariation i tunnelluften framför drän nr 8.  
*The temperature variation in the tunnel air in front of drain 8.*

I figur 11 - 14 redovisas temperaturförhållandena i bergmassan bakom dränerna vid tre olika tidpunkter. Isotermerna har konstruerats med lednings av de sex temperaturgivare som finns i bergmassan bakom dränerna.

Figur 11, typ A, visar hur kylan tränger in bakom isoleringen från den oisolerade bergmassan på sidan om dränen. Själva dränens isoleringsförmåga är tillräcklig att förhindra köldinträngning via sprutbetong och isolering. På grund av dränens ringa utsträckning i

sidled kommer bergmassan efter hand att frysa bakom isoleringen på grund av köldin-  
trängningen från sidan.

Drän 8, typ B, uppvisar samma beteende, se figur 12. I denna drän är själva dränröret  
uppvärmt med en värmekabel. Det tillförda värmet är emellertid inte tillräckligt för att  
förhindra frysning av bergmassan bakom dränen.

Drän 9, typ E, uppvisar ett liknande beteende som de två föregående dränerna, se figur  
13, men man kan se att kyla även tränger in genom själva dränen. Isoleringstjockleken  
är något mindre i denna drän jämfört med drän typ A. Dessutom är luftspalten mellan  
isolering och berg betydligt mindre än i dränerna typ A och typ B.

Bakom drän nr 4, typ D, har bergmassan inte frusit, se figur 14. Detta beror troligen på  
att den befinner sig för nära den tvärgående väggen som avskiljer den kylda delen av  
tunneln från den del av tunneln där kylaggregatet står. Denna del av tunneln håller en  
temperatur av ca 20 °C. Värmeströmningen bakom denna vägg är tydligen tillräcklig för  
att värma upp bergmassan bakom dränen. Dessutom är vattenflödet i denna drän betyd-  
ligt större än i de övriga dränerna inom det kylda tunnelavsnittet.

I figur 15 - 17 visas vattenflödet ur drän 3, 8 och 9 samt temperaturen i bergmassan 0,2  
m innanför bergytan mitt bakom dränen som funktion av tiden. Det framgår av figurerna  
att när temperaturen i bergmassan blir 0 °C minskar vattenflödet och när temperaturen  
understiger ca -1 - -2 °C avstannar vattenflödet helt.

Okulärbesiktning med fiberskop av utrymmet bakom isoleringen i dränerna visar att i  
drän 9 finns fortfarande ej isfyllda utrymmen efter det att vattenflödet ut ur dränen av-  
stannat.

I drän 8 är utrymmet mellan isolering och berg delvis fyllt med is medan själva dränröret  
där värmekabeln finns är öppet.

Drän 3 är helt fylld med is när vattenflödet ut ur dränen avstannat. Inläckagepunkten i  
drän 3 finns under det isolerade innertaket varför läckagepunkten inte har frusit till när  
övriga delar av dränen är frusna så att vattnet inte kan passera genom dränen.

De inläckagepunkter som har funnits i dränerna inom det kylda tunnelavsnittet har blivit  
täta när temperaturen i bergmassan blivit lägre än ca -2°C. Hitintills har inga för funk-  
tionen avgörande skador uppkommit på de dräner som har frusit. Flödesmätningarna  
tyder på att inläckaget i övriga ej igenfrusna läckagepunkter ökar när en läckagepunkt  
fryser igen.

## SAMMANFATTNING

För att få ett underlag till beslut om dränagesystem för Ringen och Yttre Tvärledens-tunnlar har ett fältförsök genomförts där fem olika dräntyper har testats med avseende på:

- byggbarhet och ekonomi
- dräneringsfunktion
- driftsäkerhet (m.h.t. störningar i form av frost, igensättning, dynamisk last)
- underhåll (möjligheter och behov)

Ur montage och kostnadsbedömningar har följande slutsatser dragits:

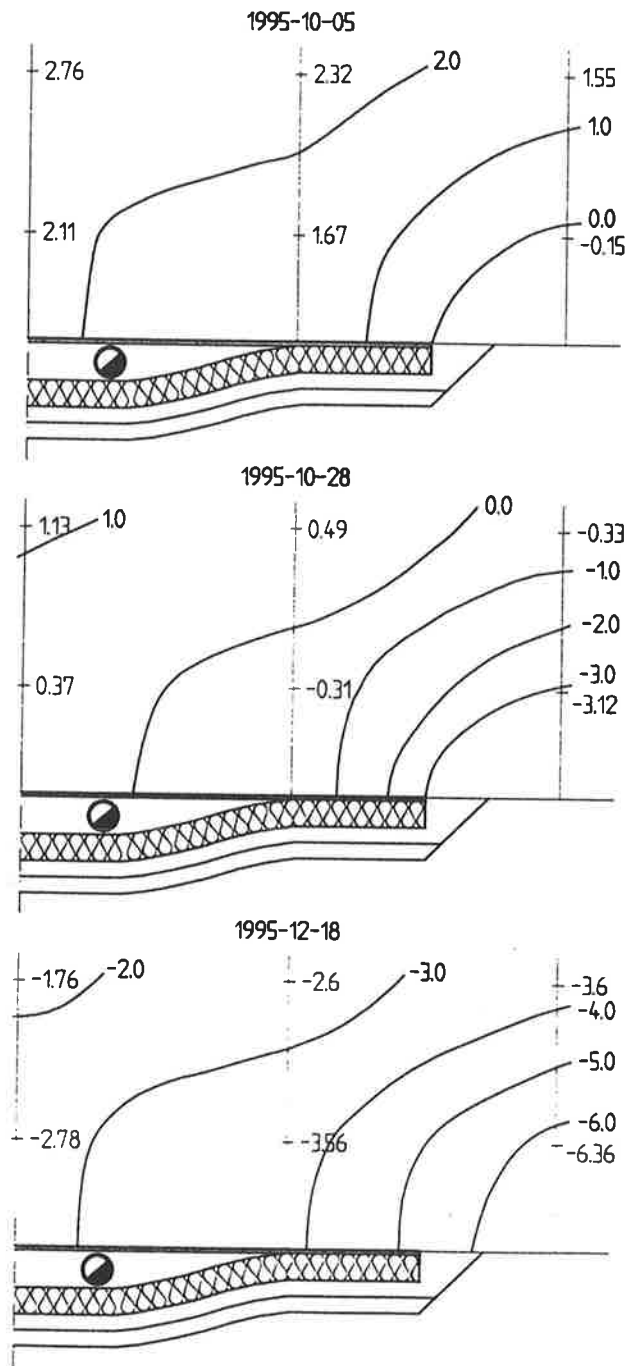
- De alternativ som är utförandemässigt enkla och som innehåller få komponenter ger också de lägsta kostnaderna.
- Billigaste utförandet för både en smal och en bred drän är med fiberarmerad sprutbetong och ingjutna bultar men utan dränrör och värmekabel.

Den utförda dynamiska provningen uppvisar ingen skillnad med avseende på utmattningshållfasthet mellan en drän som kläs med nätarmerad sprutbetong eller en som kläs med fiberarmerad sprutbetong. Man kan inte spåra någon tendens till utmattning även vid en lastnivå som motsvarar tre gånger dimensionerande last.

Slutsatserna från studien av bakteriellt betingad igensättning är bland annat att den biologiska aktiviteten, och därmed utfällningshastigheten, kan variera avsevärt mellan närliggande läckagepunkter. För områden och sprickor med "rätt" förutsättningar kan tillväxten av igensättningsprodukter ske snabbt och täta en drän inom loppet av sex månader. Vidare är det en förutsättning att ha dräner med spolrör för att rensning och spolning ska vara möjligt. Om allt järn inte fallit ut i dränen fortsätter utfällningen i väggkroppen och i dränageledningarna. I problemområden bör därför dränagesystemet utformas som ett slutet system för att möjliggöra rensning och för att undvika igensättning av väggkroppen.

Test av dräner i en nedkyld tunneldel visar att ingen av dränerna kan förhindra frysning av bergmassan bakom dränen. När temperaturen i bergmassan bakom dränen understiger ca -2 °C avstannar vattenflödet in i dränen vid de läckagemängder som uppmätts under försöksperioden ca 2 - 70 ml/min och läckagepunkt. Hitintills har inga funktionen avgörande skador uppkommit på de dräner som har frusit.

# DRÄN 3 TYP A

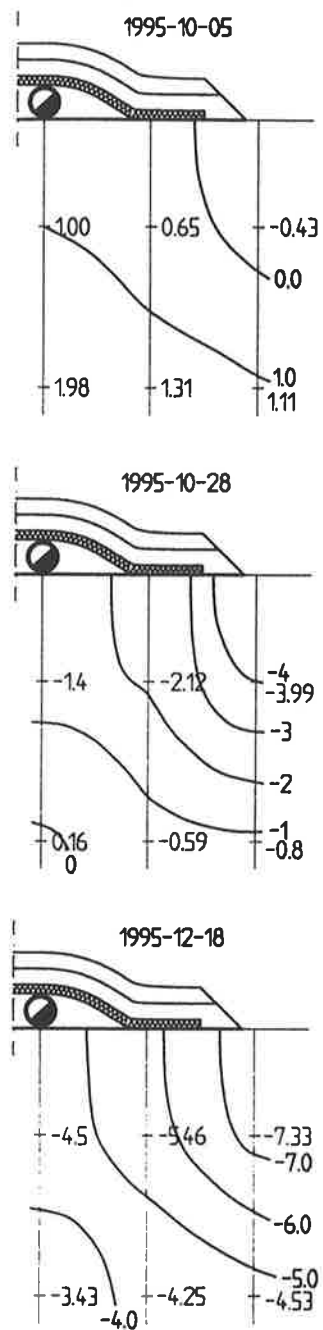


Figur 11

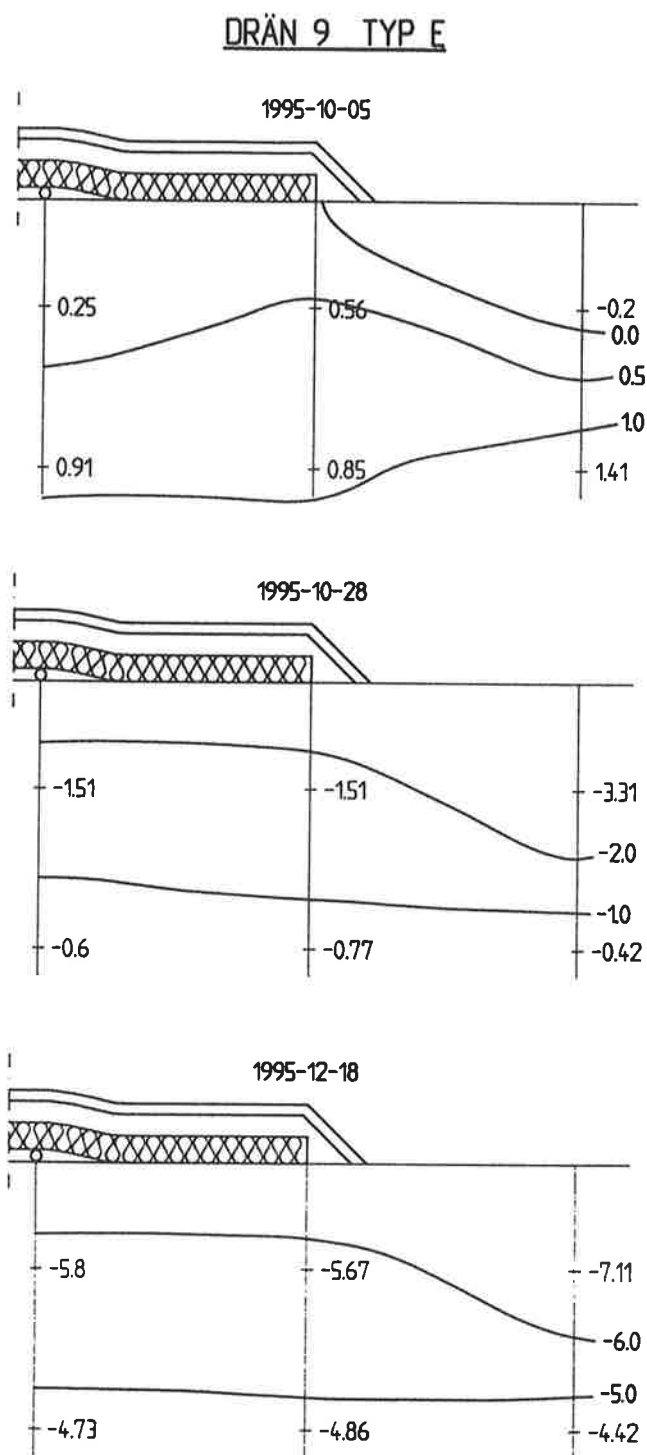
Isotermier över temperaturförhållandena i bergmassan bakom drän 3, typ A, vid tre olika tidpunkter.

*Isotermis in the rockmass behind drain 3, type A, at three separate times.*

### DRÄN 8 TYP B



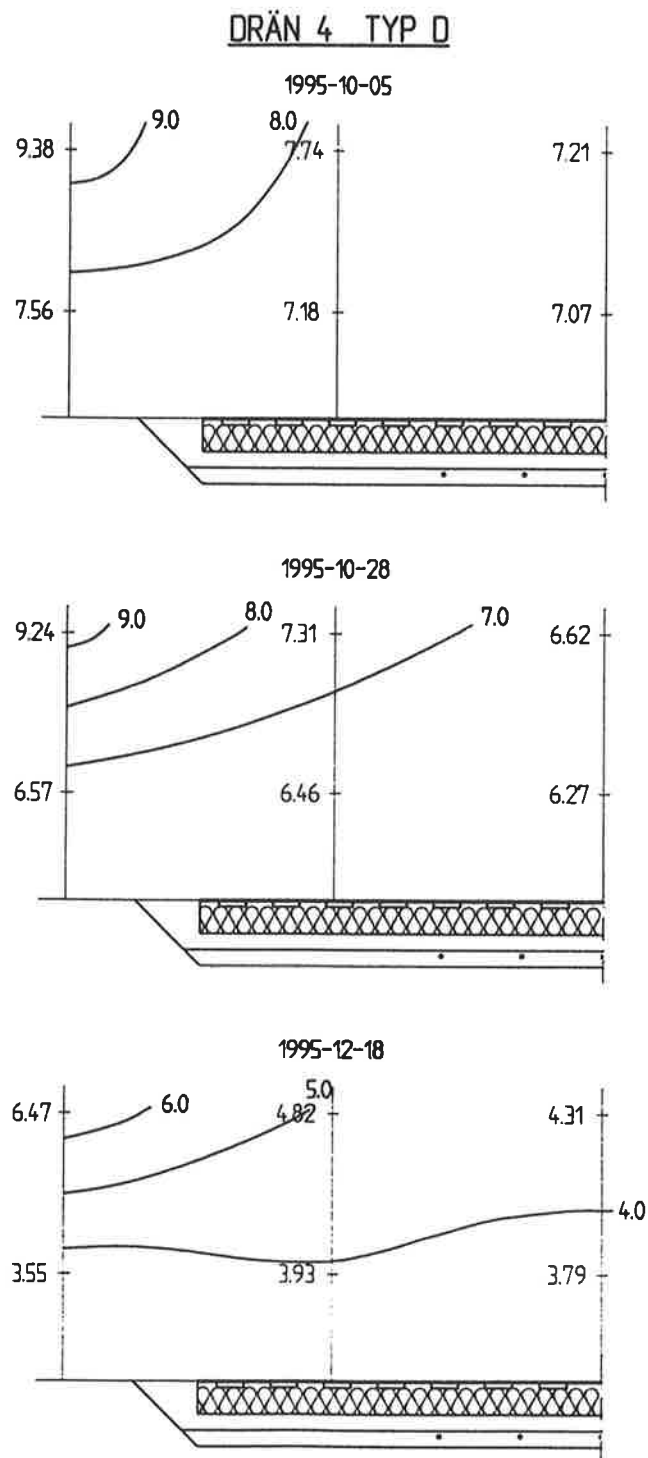
Figur 12 Isothermer över temperaturförhållandena i bergmassan bakom drän 8, typ B, vid tre olika tidpunkter.  
*Isotherms in the rockmass behind drain 8, type B, at three separate times.*



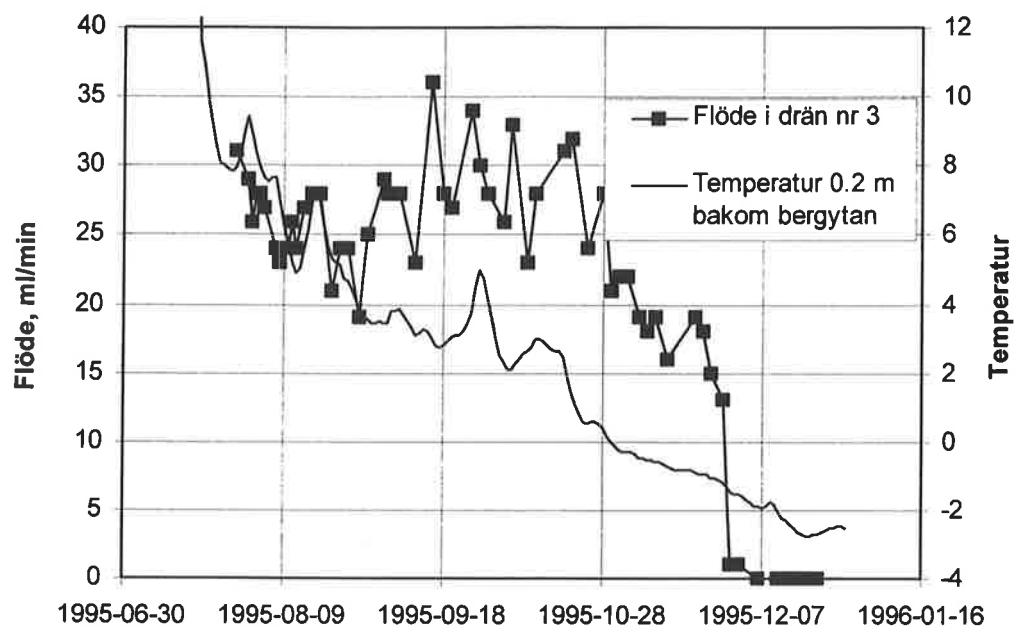
Figur 13

Isotermier över temperaturförhållandena i bergmassan bakom drän 9, typ E, vid tre olika tidpunkter.

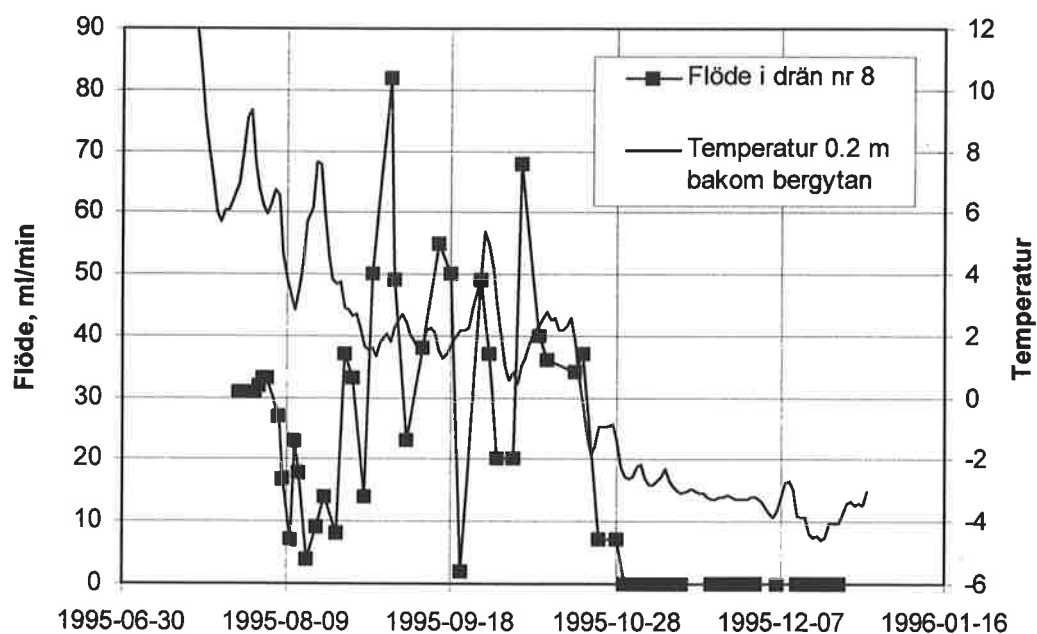
*Isotermis in the rockmass behind drain 9, type e, at three separate times.*



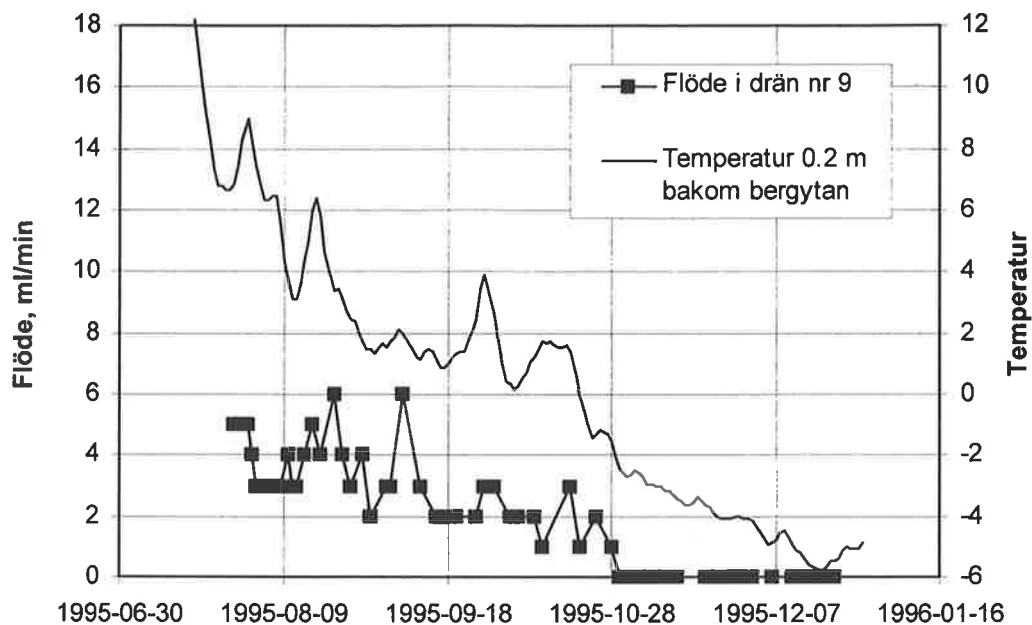
Figur 14 Isothermer över temperaturförhållandena i bergmassan bakom drän 4, typ D, vid tre olika tidpunkter.  
*Isotherms in the rockmass behind drain 4, type D, at three separate times.*



Figur 15 Flöde ur drän nr 3 samt temperatur i bergmassan .  
*Water flow out of drain 3 and the temperature variation in the rockmass.*



Figur 16 Flöde ur drän nr 8 samt temperatur i bergmassan.  
*Water flow out of drain 8 and the temperature variation in the rockmass.*



Figur 17 Flöde ur drän nr 9 samt temperatur i bergmassan.  
*Water flow out of drain 9 and the temperature variation in the rockmass.*

## SUMMARY

A large motorway ring, to large extent in rock tunnels, is planned around central Stockholm (The Stockholm Ring Road Project). As a part of the design work several critical construction details have been thoroughly investigated, among them the drainage system. The investigation of the drainage system has focused on:

- Construction and economy
- Drainage function
- Operational reliability (regarding freezing, clogging and dynamic loads)
- Maintenance (possibilities and needs)

The drains were subjected to dynamic tests in order to simulate the traffic load. No effects of fatigue was registered for any of the drains despite a loading level three times higher than the design load.

As the drains all are isolated, the freezing front enters the rock on the sides of the drain and chills the rock mass behind the drain so that the water flow is eventually stopped. No negative effects due to freezing have been observed during cooling down to  $-15^{\circ}\text{C}$ .

Micro-biological activity occurs locally in a rock tunnel and can in a worst case cause clogging of an entire drain after six months.

## INGENJÖRSGEOLOGISKA ERFARENHETER FRÅN BREIDADALS/BOTNSHEIDI-TUNNELN PÅ ISLAND

Experiences from road tunnels work on Iceland, Breidadals/Botnsheidi tunnel

*Håkan Stille, KTH*  
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### 1. TUNNELPROJEKT

I det nordvästra hörnet av Island har det isländska Vägverket, Vegagerdin, låtit utföra ett vägtunnelprojekt för att bättre knyta samman två fjorddalar, Breidadalur och Botnsdalur med Tungudalur där regionens centralort Isafjörður är belägen.

Tunnelsystemet skall ge en väderokänslig vägförbindelse och ersätta de bergsvägar som idag sammanbinder dalarna. Dessa vägar är ofta stängda under vinterhalvåret på grund av snöstormar, snöfall och snöskred.

Området är relativt glesbefolkat och trafikflödet har uppskattats till ca 300 bilar per dygn. Det ekonomiska underlaget är därför begränsat, vilket resulterar i en låg budget och sparsam design.

Entreprenör har varit ett svenskt-norskt-isländskt-danskt konsortium. Den svenska representanten var Skanska.

Tunnelsystemet är Y-format med ett ben i varje dal. Den totala tunnellängden i berg är ca 8,7 km. Från Tungudalur till korsningspunkten är tunneln dubbelfilig med en tunnelbredd på ca 7,5 m och en tvärsnittsarea på 48,5 m<sup>2</sup>. Från de andra två dalarna är tunneln enkelfilig med en bredd på 5 m och tvärsnittsytan på 29,5 m<sup>2</sup>, se figur 1.

I dessa tunnelben har mötesplatser sprängts ut på var 160 m. Alla tunnlar lutar uppåt ca 1-1,5%. Detta innebär att tunnlarerna kan utföras dränerade utan att behöva pumpa inläckande vatten. Tunnelpåslagen ligger på nivå ca 150 m. Högsta berget över tunneln ligger på ca 700 m, vilket ger en bergtäckning på upp till ca 550 m.

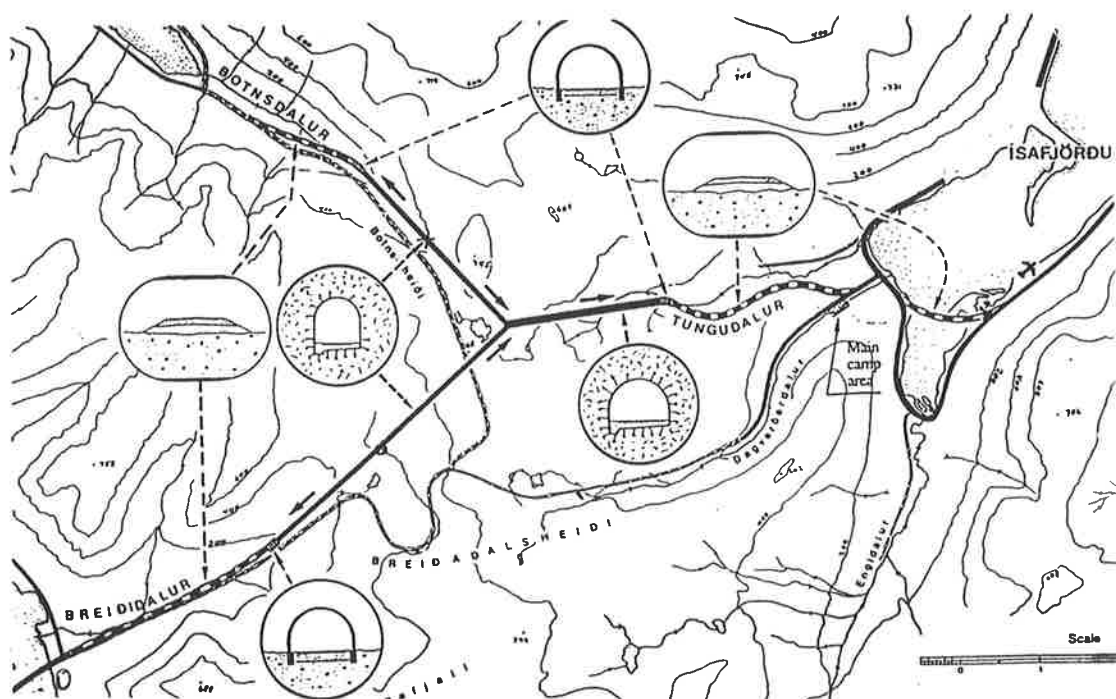


Fig. 1 Plan över tunnlarna  
Plan of the tunnels

## 2. GEOLOGI

### Allmänt

Island är beläget på den mittatlantiska ryggen som delar de europeiska och amerikanska kontinentalplattorna. Vid plattornas isärglidning uppstår vulkaniska aktiviteter och nybildning av magmatiska bergarter (basalter). Basalterna på nordvästra Island bildades för ca 10-15 miljoner år sedan och har en total tjocklek på ca 6-7 km. Basaltlagren stupar svagt mot sydost ca 4-5°. Varje lavaflöde kan strukturellt indelas i tre delar:

Den kristallina mittendelen som är själva basaltlagret omges upptill och nedtill av en scoria. Scorian består av fragment av basalt av varierande porositet bildande en typ av breccia som oftast är relativt välkonsoliderad. I toppscorian finns ofta inbäddade lager av sediment.

Själva kristallina mittendelen utgörs oftast av en hård, tung basalt med grå färg. Berget är normalt påverkat av ett typiskt irregulärt rombiskt sprickmönster på grund av avkylningsprocessen av lavaflödet. Detta gör att basalten kan uppvisa en hög porositet (5-10%), dvs betydligt större än vårt skandinaviska urberg, se figur 2.

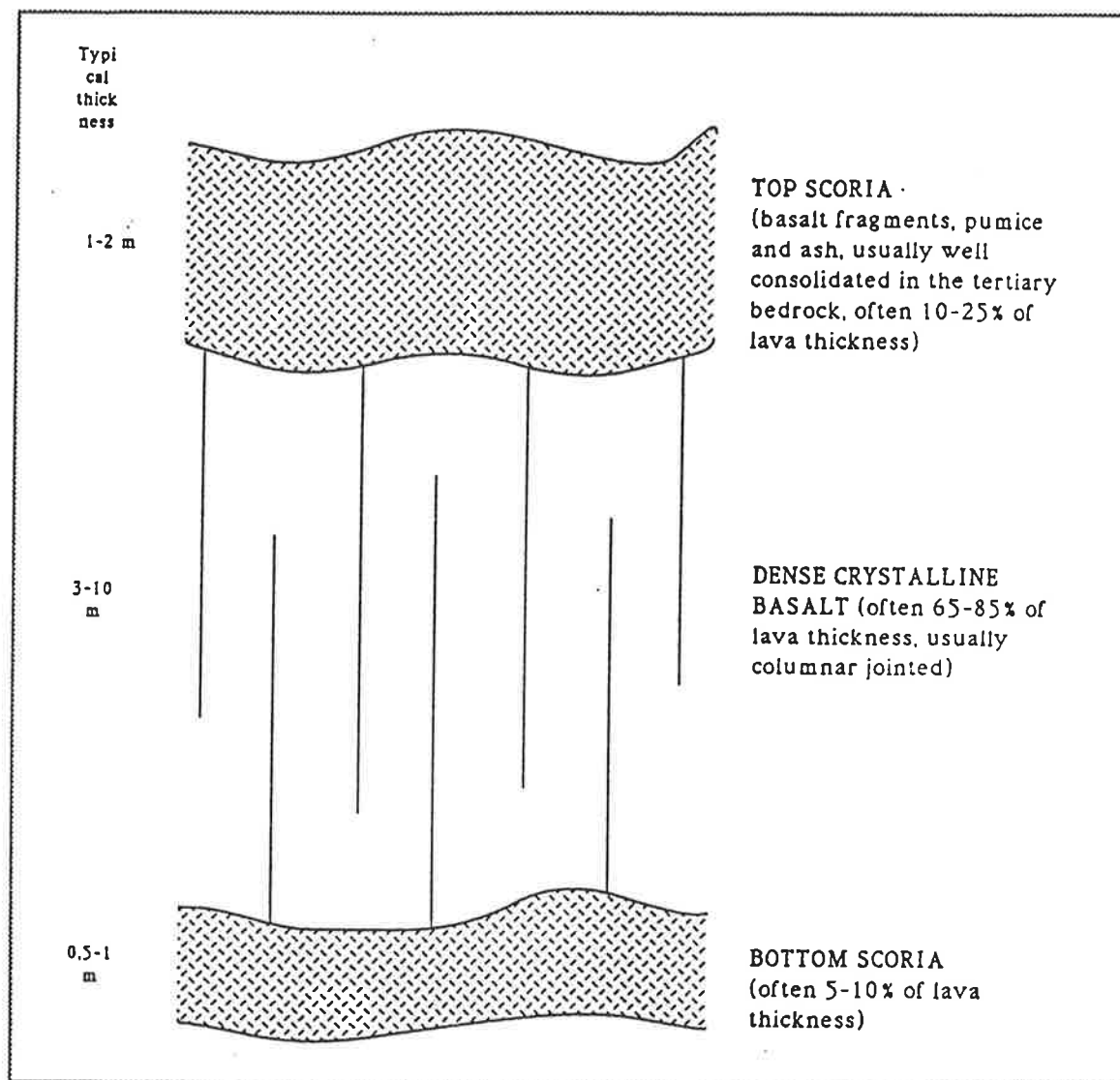


Fig. 2 Principbild av ett basaltlager  
Schematic structure of a single basalt lava

Tryckhållfastheten för dessa lager varierar men är normalt följande:

|          |               |
|----------|---------------|
| scoria   | 20 - 40 Mpa   |
| sediment | 10 - 20 Mpa   |
| basalt   | 100 - 150 Mpa |

Förkastningar är ofta förekommande. Dessa är oftast brantstående och uppvisar rörelser från någon meter upp till tiotals metrar. Breccia och krossat berg omger ofta själva förkastningen.

Vidare förekommer basaltiska brantstående gångar som övertvårar basaltflödena. Dessa är ca 0,5-15 m i tjocklek och normalt kraftigt uppspruckna.

Grundvattenytan ligger normalt nära markytan på grund av den höga nederbörden. Grundvattenflödet sker normalt på två sätt. Först ett horisontellt flöde längs basaltlagret eftersom de omgivande scorialagren är normalt täta eller mindre genomsläppliga. Sekundärt sker ett vertikalt flöde mellan de olika lagren genom förekomster av förkastningar och gångar. Dessa utgör de större vattenförande zonerna i bergmassan, eftersom de skär igenom lavaflöden och kan binda ihop de olika akvifärerna.

De ytliga akvifärerna får sitt vattentillskott direkt genom infiltration av nederbörden medan de djupare liggande akvifärerna för varmvatten och har ofta ett högre mineralinnehåll.

#### Vid projektområdet

Utförda förundersökningar gav vid handen att ett markant förkastningssystem med strykning NE-SW genomsätter bergmassan.

Dränagemönstret i form av källsprång och utförda undersökningar visade, att de översta 150-200 m av lavaflödena var mycket genomsläppliga medan basaltlagren blev generellt tätare mot djupet. Grundvattnet hade huvudsakligen ett lågt mineralinnehåll indikerande ett kallt ytligt vatten. Vattentemperaturen vid tunnelnivån förväntades vara ca 15-30°C.

Storleken på den övre reservoiren har uppskattats att ha en yta på ca 10 km<sup>2</sup> innehållande 20-40 · 10<sup>6</sup> m<sup>3</sup> vatten.

### 3. KRITISKA HÄNDELSER

#### 3.1 Stora vattenflöden i tunnlarna

##### Händelseförlopp

Det stora problemet med tunneldrivning var de stora inläckagen av vatten. Problemen finns beskrivna av Stattin, 1994.

Under drivning av de tre första kilometrarna tunnel stötte man lokalt på relativt stora mängder vatten men inget som påfordrade några speciella åtgärder, som t ex injektering. Vattentemperaturen var ca 10-15°C. Ungefär vid korset uppmätte man lokalt en betydligt lägre vattentemperatur, ca +4°C. Detta tolkades såsom att en av undersökningshålen ledde ned det kallare vattnet från den övre akvifären ned till tunneln.