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STIFTELSEN SVENSK BERGTEKNISK FORSKNING
SWEDISH ROCK ENGINEERING RESEARCH

och Svenska Bergmekanikgruppen



SVENSKA BIDRAG TILL ISRM:s ÅTTONDE INTERNATIONELLA BERGMEKANIKKONGRESS TOKYO 1995

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Experiences from the Äspö Hard Rock Laboratory: Site characterization approach

Expériences de l'approche de caractérisation en laboratoire et sur site des roches dures d'Äspö

Erfahrungen aus der Charakterisierungs-Methode für den Standort des Äspö-Hartgestein-Laboratoriums

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ABSTRACT: The Äspö Hard Rock Laboratory was launched 1986 for the main purpose of providing an opportunity for research, development and demonstration in a realistic and previously undisturbed underground rock environment down to the depth planned for the future deep geological repository of long-lived nuclear waste in Sweden. An access ramp has been excavated down to a depth of 450 m below surface where experiments will continue in 1995. Prior to excavation, the results of the pre-investigations of the bedrock were analyzed and modelled on several geometric scales. For each scale, key issues were selected. Several hundred parameter values were then predicted. During construction these predicted values are being compared to the situation in the tunnel and in the boreholes. Between 100 - 200 borehole sections have been monitored on-line. The prediction-outcome matches are now under evaluation. Many important results have been obtained and valuable experience has been gained. The work is presently carried through with participation from organizations in Canada, Japan, Finland, France, Switzerland, United Kingdom and United States.

ZUSAMMENFASSUNG: Der Gedanke für die Erstellung eines Äspölaboratoriums wurde 1986 geweckt. Der Zweck einer solchen Anlage ist realistisch Forschung, Entwicklung und Demonstrationen in einem unberührten Bergmiljö zu betreiben. Die Arbeiten sollen sich bis zur Tiefe des geplanten schwedischen Endlagers für langlebigen schwedischen Kernabfall erstrecken. Der Zufahrtstunnel ist auf eine Tiefe von 450 m gebracht worden wo umfangreiche Experimente durchgeführt werden sollen. Vor dem Ausbau des Tunnels wurden die Ergebnisse der Voruntersuchungen ausgewertet und Modellberechnungen durchgeführt in verschiedene geometrischen Maßstäben. Für jeden Maßstab wurden die wichtigsten Fragen ausgewählt und mehrere hunderte von Parametern wurden anschließend vorausgesagt. Während der Bauphase werden diese vorausgesagte Messwerte mit der Situation im Tunnel sowie in den Bohrlöchern verglichen. Zwischen 100 und 200 ausgewählten Abschnitten in den Bohrlöchern werden von Messgeräten ständig überwacht. Die Übereinstimmungen zwischen Vermessungen und vorhergesagten Messwerten werden ausgewertet. Viele wichtige Ergebnisse und wertvolle Erfahrungen sind dadurch gewonnen worden. Am Projekt nehmen Organisationen von Kanada, Japan, Finland, Frankreich, Schweiz, England und der USA teil.

RÉSUMÉ: Les travaux de construction du laboratoire d'Äspö ont été entrepris dès 1986. Leur objet est de susciter une possibilité de recherches, de développement et d'essai d'installation, dans un milieu stable et adapté, du stockage à long terme et en profondeur de déchets de l'industrie nucléaire suédoise. Un tunnel d'accès a été construit en direction de 450 mètres au-dessous du niveau de sol. L'expérience va continuer en 1995. Avant l'excavation, les résultats des investigations de site de la roche primitive ont été analysés et modélisés sur plusieurs échelles géométriques. Pour chaque échelle, sujets principaux ont été sélectionnés. Alors plusieurs centaines de valeurs de paramètres ont été pronostiquées. Pendant la construction ces valeurs pronostiquées sont comparées à la situation dans le tunnel et dans les forages. Entre 100 et 200 sections de forages ont été monitorées on-line. La comparaison entre prédiction et résultat est maintenant en évaluation. Plusieurs résultats importants ont été obtenus et l'expérience d'une grande valeur a été gagnée. Ce travail est réalisé avec participation d'organisations canadiennes, japonaises, finlandaises, suisses, britanniques, américaines et française.

BACKGROUND

SKB has decided to construct the Äspö Hard Rock Laboratory for the main purpose of providing an opportunity for research and development in a realistic and undisturbed underground rock environment down to the depth planned for the future deep repository in Sweden.

One of the stage goals is to "Verify pre-investigation methods" - e.g. to demonstrate that investigations on the ground surface and in boreholes provide sufficient data on essential safety-related properties of the rock at repository level.

A comprehensive site characterization programme was carried out prior to the start of construction in 1990. During the excavation work down to the 450 m level comparisons have been made between the predictive descriptions based on the pre-investigations and the actual outcome as documented in conjunction with the construction of the laboratory. The facility is depicted in Figure 1.

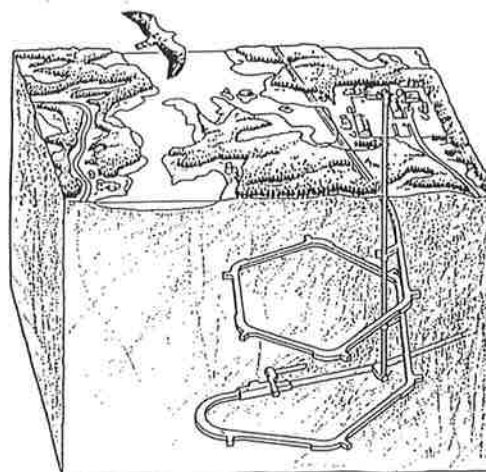


Figure 1. Sketch of the Äspö Hard Rock Laboratory

There is a strong international interest in the work with participation from Canada (AECL), Japan (PNC, CRIEPI), Finland (TVO), France (ANDRA), Switzerland (NAGRA), the United Kingdom (NIREX), and the United States (USDOE). Participation with German organizations are foreseen during 1995.

HOW WILL THE SITE INVESTIGATION RESULTS BE USED?

The purpose of pre-investigations or site investigations is to:

- demonstrate that a site has suitable geological properties
- obtain data and knowledge concerning the bedrock on the site so that a preliminary emplacement of the repository in a suitable rock volume can be done as a basis for constructability analysis
- obtain the necessary data for a preliminary safety assessment, which in Sweden shall serve as support for an application under NRL (the Act Concerning the Management of Natural Resources) to carry out detailed site characterization
- obtain data for planning of detailed site characterization, which is done in conjunction with the construction work down to repository level.

It is thus important to show that pre-investigations provide reasonable and robust results.

The reliability of the pre-investigations can be tested during the detailed characterization. The work at Äspö is thus a "dress rehearsal" for the site investigations and the detailed characterization for the Swedish deep repository.

WHAT SHOULD BE ACHIEVED IN SITE CHARACTERIZATION?

Site characterization shall provide an understanding of the site to:

- Demonstrate the existence of features (processes, structures) and their importance
- Justify division of the features into domains
- Make parameter estimations as an input to model realizations, to support division into domains and to prove the existence or non-existence of features

Demonstrating or rejecting the existence of features is of the utmost importance in the characterization programme. Features may have to do with lithology, chemical processes, structural features etc. At the outset of characterization it is important that the programme be broad so that unexpected features can also be detected.

Division of the rock into domains can be done for e.g. lithological bodies and groundwater composition such as fresh/saline water, redox conditions, or be applicable to the geometry of structural features.

Parameter estimation for each domain can be done with alternative evaluation models. The safety assessor's desire for "alternative models" is warranted for parameter estimation purposes.

The relevance of each feature, domain and parameter should

be assessed by both the engineer and the safety assessor.

SAFETY ASSESSMENT SHOULD ANALYZE THE REPOSITORY "AS BUILT"

From the view point of the safety assessor it is important that site characterization be so reliable that the engineering and construction work can be done in a reliable way. The reason is as follows:

The engineer basically divides the rock into two families. "Good rock" implies that the rock can be excavated without any sealing or support work. "Bad rock" means that the rock needs grouting or reinforcement by steel and/or concrete.

It is argued that this very simple division is of great importance. Rock bolting with steel bars provides a connection between the backfill of a deposition drift and a possible disturbed zone; shotcreting will interfere with the required close interaction between the backfill and the host rock over thousands of years due to the deterioration of the concrete leaving a gap between the backfill and the rock; pre-grouting of deposition tunnels will make it difficult to obtain accurate values of transmissivity at canister positions.

The chemical inventory for a deep repository could include some 1000 tonnes of cement, maybe some 100 tons of nitrogen compounds (residues from blasting agents) and some minor inventory of oil, cellulose etc. The backfill material will include huge amounts of oxygen to be consumed in the closed repository.

Based on the site investigations a constructability analysis /1/ should be performed so that the safety assessment can give an account of the repository "as built" even prior to construction.

The engineer's wish to avoid "bad rock" is an advantage both for the engineering and for the safety of the repository. It will also simplify the safety assessment. The engineer's wish to find "good rock" can be an advantage. However there is a need to check for stress or thermomechanical problems that may appear in very good (unfractured) rock. Thus, ordinary engineering practice will be of the utmost importance in the licensing process for a deep repository. Utilizing centuries of knowledge supports confidence building for the site characterization and engineering of the repository.

The safety assessor sometimes asks for alternative models to be presented. This is not always acceptable to the engineer responsible for the layout of a facility. He must e.g. be certain that no major water conducting fracture zones are present at the underground facility; he thus needs a "best estimate" where the safety assessor would prefer alternative models.

THE APPROACH TO SITE CHARACTERIZATION AT ÄSPÖ

A basic description of the site characterization approach can be found in /2/. Site characterization is a multi- and inter-disciplinary task that necessitates integration in data acquisition, evaluation and presentation of results.

In order to facilitate such integration, four basic decisions were made for the site characterization.

- to divide the characterization and interpretation into distinct stages
- to devise models on several geometric scales (regional >> 1000 m, site scale 100 - 1000 m, block scale 10 - 100 m and detailed scale 1-10 m)
- to select key issues related to the geological-structural

Table 1. Condensed description of the groundwater flow model of the Äspö in site scale - example

GROUNDWATER FLOW MODEL OF ÄSPÖ SITE Stochastic continuum model	
<i>Scope:</i> natural flow, flow to laboratory tunnel, cross-hole tests (calibration)	
<i>Process description</i> Continuity equation (mass rate) Equation of motion (Darcy's law including density driven flow)	
CONCEPTS	DATA
<i>Geometric framework and parameters</i>	
3D box divided into: 2D fracture zones, planar with limited extent (location, orientation, size) Rock Mass Units (location of boundaries) Subvolumes (cells) between fracture zones	Size: 1.9x1.5x1.3 km Zone geometry from geologic model (descriptions basis for selection of "important zones") Regular grid of 20 m cubes Spatial distribution of 5 rock mass units (RMU) with 50 m thick slabs (with depth)
<i>Material properties</i>	
Zones: Transmissivity (T_i) Subvolumes: Hydraulic conductivity, isotropic (K_i) Salinity field	T and K from hydraulic borehole testing Salinity measurements in boreholes
<i>Spatial assignment method</i>	
Transmissivity: Deterministic assignment Hydraulic conductivity of cells: log-normal distribution based on RMU, depth. K and σ dependent on cell size	Transmissivity: single- and cross-hole testing Stochastic distribution of K from borehole testing
<i>Boundary conditions</i>	
Upper: fixed infiltration rate on Äspö, constant head at sea and peat areas Lower: no flow Side: prescribed pressure (hydrostatic) Salinity: prescribed initial conditions, linear increase with depth Tunnel: skin for rock and zones, prescribed pressure (atmospheric)	Infiltration data Salinity of Baltic Sea, salinity measurements in boreholes
<i>Numerical tool</i> PHOENICS	
<i>Output parameters</i> Pressure, density (derived parameters: flux, salinity)	

model, groundwater flow, groundwater chemistry, transport of solutes and mechanical stability

- to integrate and facilitate integration of data collection and data interpretation

For each of the three stages, summary evaluations were reported /3--5/.

The last stage was the "Prediction Stage" /6/ where the rock descriptions were put into predictions to be checked during the construction of the laboratory, cf Figure 2.

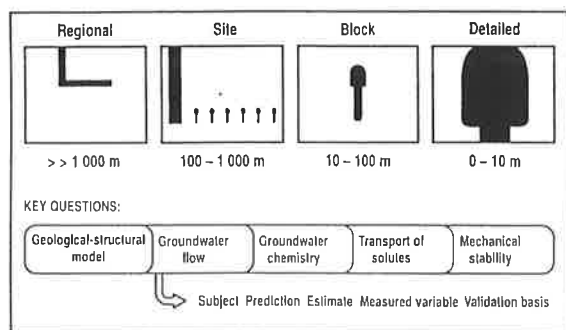


Figure 2 Analyses and models take place on different geometric scales with respect to a number of key issues.

The nomenclature for theories, models and conceptual models was found out to be confusing. A nomenclature for SKB work has now been established /7/. The basic elements of the nomenclature are illustrated by an example in Table 1.

The on-going evaluation work in progress is directed to a studying the feasibility and reliability of the concepts used for each key issues. The bases for these evaluations are the comparison between prediction and outcome.

SOME EXAMPLES OF RESULTS FROM COMPARISONS OF PREDICTIONS AND OUTCOMES

The evaluations of the pre-investigations and associated predictions were presented prior to the start of the excavation work /5, 6, 8, 9, 10/. Some supplementary work has also been reported /11, 12/.

Based on evaluations to date some of the results are discussed below.

Existence of features

The bedrock at Äspö is lithologically heterogeneous. Based on evaluation of significance the rock is basically divided into five different lithologies. One of these, the fine-grained granite, is much more fractured than other rock types and was assumed to be more water-onducting and thus would be of interest both to the engineer and the safety assessor.

The existence of the fine-grained granite was predicted and has been confirmed.

The existence of several major fracture zones was predicted. They have all been confirmed. No other major zones exist. To avoid confusion a nomenclature for fracture zones and reliability (Certain, Probable, Possible) was adopted /13/.

One subhorizontal zone classified as "Possible" has not been found. The existence of minor fracture zones was predicted. Their existence has been confirmed.

The existence of saline water was predicted and its existence has been confirmed.

Processes that were not predicted are oxygen consumption (reducing conditions) and sulphate reduction due to bacterial activity /14/.

Division into property domains

Due to the heterogeneity of the lithology it was considered inappropriate to predict the location of fine-grained granite. This proved to be a wise decision. Even after construction work, such a prediction is not appropriate. However, the average volume (%) of fine-grained granite was predicted.

The dip and strike of the most important fracture zone NE-1 was predicted and the outcome showed small deviations.

One major fracture zone NE-2 has a complicated dip. Parts of the zone dip north, other parts of the zone dip south. As the zone was considered to be of minor importance (a true prediction!) few investigations were made to determine the exact geometry of the zone. The outcome is that the zone dips south instead of north as predicted.

The strike of the minor fracture zones were predicted, but not their exact location. The strikes and dips of the minor fracture zones trending NNW have been confirmed to conform to the predictions. Their exact locations were determined in the detailed characterization in connection with excavation.

The groundwater chemistry was predicted to be composed of different bodies with higher salinity at greater depth. This has been confirmed.

Parameter estimation

Evaluation of the predicted parameter estimations is under way. An example of parameter estimation for the key issue of geology is shown in Table 2, where DEV is deviation from mean (%).

Table 2. Prediction - outcome match for lithology in tunnel

Lithological unit	Prediction (%)	Outcome (%)	DEV (%)
Småland granite	20($\pm 5\%$) _{60%*}	28	8
Äspö diorite	56($\pm 5\%$) _{60%*}	55	1
Greenstone	8($\pm 3\%$) _{60%*}	3	5
Fine-grained granite	14($\pm 3\%$) _{60%*}	14	0
Mylonite-hybridized	2($\pm 1\%$) _{60%*}	0	2

* Level of confidence

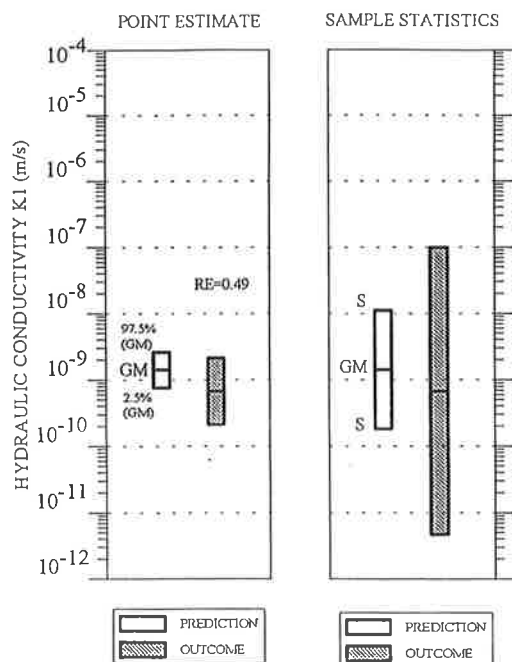


Figure 3 Hydraulic conductivity in tunnel section 2265 - 2875m (level 300 - 400 m). The left part of the figure shows the point estimate of the geometric mean of the hydraulic conductivity (predictive values scaled from 3m to the test length of the observed values, 14 m). GM = geometric mean, S = one standard deviation, RE = outcome/prediction of the geometric mean.

Figure 3 shows an example of evaluation of parameter estimation for the key issue of geohydrology.

The predicted composition of the groundwater in major fracture zones and the salinity in the zones is outside the range from the predictions. Water from greater depths is sampled at shallower depth than predicted. The underlying assumptions made in the prediction work are being examined.

EXPERIENCE FROM THE SITE CHARACTERIZATION APPROACH

The site characterization approach as described above has been useful. The idea of dividing the investigations into different stages, scales and key issues has simplified the evaluation work considerably and further use of this approach is advocated.

Due to the lithological heterogeneity, it seems that block scale predictions were not as useful as site and detailed scale predictions.

Constructability analysis should be added as a key issue to ascertain a good coupling between site characterization and the engineering work, cf Figure 2.

Based on a general site investigation strategy the site specific strategy shall not be fixed until some preliminary field work has been evaluated, remembering e.g. that major fracture zones can be low-conductive and minor fracture zones can be highly conductive.

In the pre-investigation phase it was difficult to determine:

- The exact position and orientation of minor subvertical fracture zones (e.g. NNW-structures at Äspö).

- The detailed character and importance of subvertical fracture zones.
- The relative importance of the different subhorizontal fracture zone indications.
- The location and distribution of minor rock units at depth (e.g. greenstone lenses and veins of fine-grained granite).
- The validity of the theoretical model of the scale dependency of hydraulic properties.
- The hydraulic properties of minor rock types.
- The absolute water pressures in boreholes at great depth due to varying salinity with depth.
- The groundwater chemistry in low conductive rock masses.

There is a need for a more stringent methodology for assessment of the accuracy of measurement and evaluation methods.

The need for appropriate classification systems should not be underestimated. The fracture zone classification used has been useful [13].

Three major drilling campaigns were carried out at Äspö prior to construction. The first three holes basically showed all the important properties to be found at Äspö. These three holes also provided a good basis for parameter estimation of both average values and the variance of the parameters. The second and third campaigns (11 deep cored holes) were basically useful for increasing confidence in the geometry of major fracture zones and learning more about the minor fracture zones at Äspö. The geometry of the major fracture zones was very important for the layout of the Äspö facility, to ensure that it was located in good rock.

CONCLUDING REMARKS

Evaluation of the Äspö work is under way. This evaluation concerns the scientific results as well as the experiences gained.

Altogether the work will be of great interest for the implementation of deep repositories in Sweden and elsewhere.

Some concluding comments are:

- Dividing the investigation strategy into stages, scales and key issues was useful
- Predictions useful for engineering were adequate
- Constructability analysis should be a key issue
- Evaluation of usefulness of data for safety assessments is in progress

ACKNOWLEDGEMENT

The Äspö work has been executed with several experts from Sweden and from abroad. Their dedicated efforts have been most appreciated and have provided inspiration and enhanced the quality of the work in many ways.

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The effect of anisotropy on the K_I calibration of ISRM standard fracture toughness specimens

Effet de l'anisotropie sur la calibration K_I de spécimens de résistance à la fracture standard de l'ISRM

Die Wirkung von Anisotropie auf die K_I -Kalibrierung eines ISRM-Standard-Bruchzähigkeitsmusters

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ABSTRACT: A 3-D finite element analysis was carried out in order to investigate the effect of material anisotropy on the K_I values of the ISRM standard specimens for the fracture toughness testing of rock. This paper analyses the case of transverse isotropy.

Numerical results show that the ratio n of Young's moduli has a significant effect on K_I for the ST and ARR configurations of SR specimen. The relation between K_I and $\log n$ is almost linear. This provides a simple calibration formula in the evaluation of the fracture toughness of transversely isotropic rocks. For the CB specimen, however, the effect of anisotropy is quite small. The effect of the Poisson's ratio is further almost negligible for both specimens.

RESUMÉ: Des analyses FEM tridimensionnelles ont été réalisées en vue de vérifier l'effet de l'anisotropie sur les valeurs de K_I des éprouvettes standardisées d'ISRM pour la ténacité des roches. Ce papier analyse le cas de l'isotropie transversale.

Les résultats numériques montrent un effet significatif du quotient n des modules d'élasticité sur l'éprouvette SR quand la fissure a la direction ST ou ARR. On a trouvé que K_I est une fonction de $\log n$ presque linéaire. Cette circonstance donne un facteur de calibrage très simple dans la détermination de la ténacité des roches avec l'isotropie transversale. Pour l'éprouvette CB d'autre part, l'influence de l'anisotropie est minimale. De plus, l'influence du facteur de Poisson est presque négligeable pour les deux éprouvettes.

ZUSAMMENFASSUNG: Mit einer dreidimensionalen FEM-Analyse wurde der Einfluss der Materialanisotropie auf die K_I -Werte der ISRM-Standardprobekörper für Bruchzähigkeitsmessungen an Gesteinen untersucht. Dieser Beitrag behandelt den Fall transversaler Isotropie.

Numerische Resultate zeigen, dass der Quotient n der Elastizitätsmodulen einen erheblichen Einfluss auf K_I des SR-Probekörpers hat, wenn der Riss in der ST oder ARR Orientierung liegt. K_I ist von $\log n$ fast linear abhängig. Dies führt zu einer einfachen Korrekturformel bei der Bruchzähigkeitsbestimmung von transversal isotropen Gesteinen. Für den CB-Probekörper ist der Einfluss der Anisotropie dagegen klein. Weiterhin ist die Abhängigkeit von K_I von der Querdehnungszahl für beide Probekörper fast vernachlässigbar.

1 INTRODUCTION

Interlaboratory testing according to the ISRM standard methods for determining the fracture toughness of rock has recently been carried out. Many numerical studies have also been made to provide verification of the experimental results (e.g., Ouchterlony 1987, Ingraffea *et al.* 1984, Matsuki *et al.* 1991, Taniguchi *et al.* 1994). Most of those studies, however, assumed that rock is isotropic. It is pointed out that for rocks such as granites and sandstones, anisotropy may be a major source of error in evaluating the fracture toughness and compliance from experimental results (Ouchterlony 1989).

This paper numerically investigates the effect of anisotropy on the K_I calibration of the ISRM standard fracture toughness specimens: chevron bend (CB) and short rod (SR) (ISRM, 1988). A 3-D finite element method is used to calculate crack opening displacements at crack tips for various values of anisotropic material constants. Stress intensity factors are obtained by using an explicit expression of displacements near the crack tip in the anisotropic elastic solid. The axes of material symmetry are assumed to coincide with the axes of symmetry of the test specimens so that the crack tip always lies in the mode I stress field. The anisotropy considered here is called transverse isotropy. Even in this simple case, there are five independent material parameters. Hence an appropriate assumption is made in the present work to reduce the number of the independent variables. In the numerical results, the stress intensity factors are shown as functions of the anisotropy parameters. From these results, we will propose calibration factors for the evaluation of fracture toughness of anisotropic rocks.

2 FINITE ELEMENT ANALYSIS

A 3-D finite element method is applied to obtain the crack opening displacements at crack tips in both CB and SR speci-

mens. The geometries and loading conditions of the specimens are illustrated in Fig. 1. In both specimens, the thickness t of the chevron notch is taken as zero and the crack front B is assumed straight and normal to the x_1 -axis. Since the specimen configurations are symmetric with respect to both the x_1 - x_2 and x_1 - x_3 planes, only a quarter of each specimen is modelled using isoparametric elements with twenty nodes except for the crack front segment. Along the segment of the crack front edge, collapsed quarter-point singular elements are arranged to represent the stress singularity accurately (Ingraffea & Manu 1980). Figs. 2 (a) and (b) show the element arrangements for a quarter of the CB and SR specimens, respectively. It is assumed that on the shadow areas in Fig. 2, normal displace-

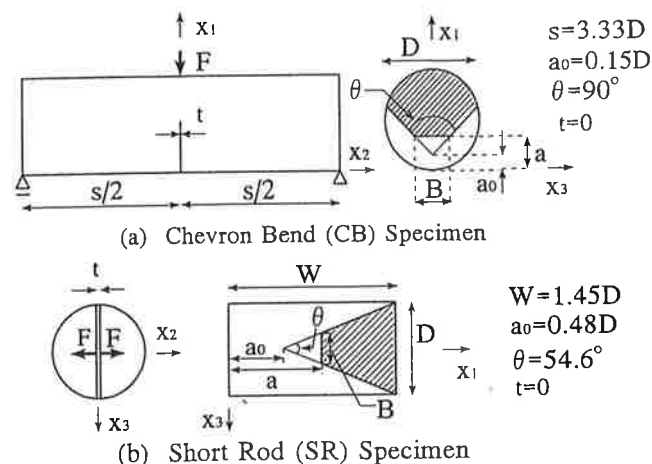


Figure 1: Geometrical configurations of ISRM standard CB and SR specimens.

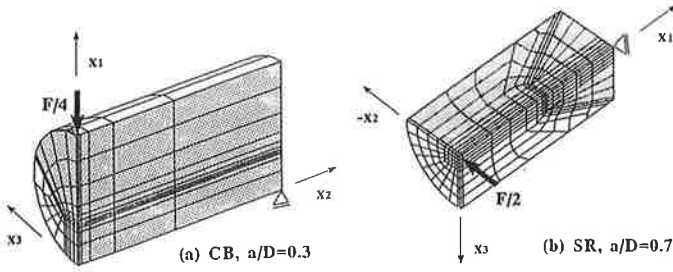


Figure 2: Arrangement of finite elements for a quarter of CB and SR specimens.

ments are fixed and tangential tractions are free to represent the symmetric conditions for the quarter size models in the FEM analysis. On the other boundaries, traction free conditions are given except for the loading points and the supporting points.

3 STRESS INTENSITY FACTORS

The basic equations which describe the behavior of anisotropic bodies are the same as those of isotropic bodies except for the constitutive equation given by the generalized Hooke's law:

$$c_i = a_{ij}\sigma_j \quad (i, j = 1, 2, \dots, 6, \quad a_{ij} = a_{ji}) \quad (1)$$

and its inverse is :

$$\sigma_i = c_{ij}\epsilon_j \quad (i, j = 1, 2, \dots, 6). \quad (2)$$

Sih *et al.* (1965) have shown that the relation between the crack opening displacements and the stress intensity factors in an anisotropic material is given by

$$\begin{Bmatrix} [u_1] \\ [u_2] \\ [u_3] \end{Bmatrix} = 2\sqrt{\frac{2r}{\pi}} [\vec{B}] \begin{Bmatrix} K_I \\ K_{II} \\ K_{III} \end{Bmatrix} \quad (3)$$

where $[u_i]$ is the crack opening displacement in the x_i direction and K_I , K_{II} and K_{III} are stress intensity factors for modes I, II and III, respectively. Also, r denotes the distance from the crack tip and the matrix $[\vec{B}]$ is given by

$$[\vec{B}] = \begin{bmatrix} \text{Re} \left[\frac{i}{s_1 - s_2} (s_1 p_2 - s_2 p_1) \right] & \text{Re} \left[\frac{i}{s_1 - s_2} (p_2 - p_1) \right] & 0 \\ \text{Re} \left[\frac{i}{s_1 - s_2} (s_1 q_2 - s_2 q_1) \right] & \text{Re} \left[\frac{i}{s_1 - s_2} (q_2 - q_1) \right] & 0 \\ 0 & 0 & (c_{44}c_{55} - c_{45}^2)^{-1/2} \end{bmatrix} \quad (4)$$

where s_1 and s_2 are the solutions with positive imaginary parts which satisfy the following equation;

$$a_{11}s^4 - 2a_{16}s^3 + (2a_{12} + a_{66})s^2 - 2a_{26}s + a_{22} = 0, \quad (5)$$

and p_j and q_j with $j = 1, 2$ are given by

$$p_j = a_{11}s_j^2 + a_{12} - a_{16}s_j, \quad (6)$$

$$q_j = a_{12}s_j + a_{22}/s_j - a_{26}. \quad (7)$$

For an isotropic material, the matrix $[\vec{B}]$ can be expressed as

$$[\vec{B}] = \begin{bmatrix} \frac{4(1-\nu^2)}{E} & 0 & 0 \\ 0 & \frac{4(1-\nu^2)}{E} & 0 \\ 0 & 0 & \frac{4(1+\nu)}{E} \end{bmatrix}, \quad (8)$$

where E and ν are the Young's modulus and the Poisson's ratio, respectively.

As mentioned before, we introduced collapsed quarter-point singular elements around the crack front edge to represent the stress singularity accurately. Using the nodal displacements in the neighborhood of the crack front as shown in Fig. 3, the

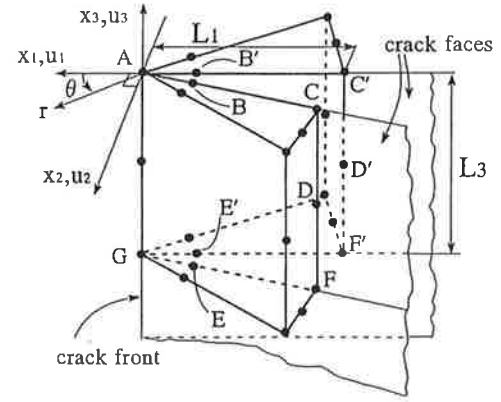


Figure 3: Arrangement of quarter-point singular elements along the crack front edge.

crack opening displacements are expressed as follows (Ingraffea & Manu 1980);

$$\begin{Bmatrix} [u_1] \\ [u_2] \\ [u_3] \end{Bmatrix} = 2 \{ \vec{A} \} \sqrt{\frac{r}{L_1}} \quad (9)$$

where L_1 is the length of the singular element and the vector $\{ \vec{A} \}$ is given by

$$\{ \vec{A} \} = \begin{Bmatrix} A_2 \\ A_1 \\ A_3 \end{Bmatrix}. \quad (10)$$

Here the component A_i of the vector $\{ \vec{A} \}$ is written as

$$A_i = (2[u_i^B] - [u_i^C] + 2[u_i^E] - [u_i^F] + [u_i^D])/2 + \eta(-4[u_i^B] + [u_i^C] + 4[u_i^E] - [u_i^F])/4 + \eta^2([u_i^F] + [u_i^C] - 2[u_i^D])/4 \quad (11)$$

where $\eta = -(2x_3/L_3 + 1)$, L_3 is the length of the element along the crack front and $[u_i^a]$ denotes the crack opening displacement at the node a , namely, $[u_i^a] = u_i^a - u_i^{a'}$.

From eqs.(3) and (9), the stress intensity factors can be evaluated using the nodal displacements in the neighborhood of the crack tip as follows (Saouma & Sikiotis 1986).

$$\begin{Bmatrix} K_I \\ K_{II} \\ K_{III} \end{Bmatrix} = [\vec{B}]^{-1} \{ \vec{A} \} \sqrt{\frac{\pi}{2L_1}}. \quad (12)$$

4 SELECTION OF ANISOTROPIC PARAMETERS

We here consider a transversely isotropic material. If an isotropic plane coincides with the x_1 - x_2 plane, Hooke's law, eq.(1), can be expressed in terms of engineering constants as follows:

$$\begin{Bmatrix} \epsilon_{11} \\ \epsilon_{22} \\ \epsilon_{33} \\ \epsilon_{23} \\ \epsilon_{31} \\ \epsilon_{12} \end{Bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & -\nu & -n\nu_T & 0 & 0 & 0 \\ & 1 & -n\nu_T & 0 & 0 & 0 \\ & & n & 0 & 0 & 0 \\ \text{sym.} & & & \frac{E}{G_T} & 0 & 0 \\ & & & & \frac{E}{G_T} & 0 \\ & & & & & \frac{E}{G} \end{bmatrix} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{23} \\ \sigma_{31} \\ \sigma_{12} \end{Bmatrix} \quad (13)$$

where E , ν and G are the Young's modulus, the Poisson's ratio and the shear modulus, respectively, for an inplane loading; $n = E/E_T$ and the subscript T indicates the quantities with respect to the antiplane direction x_3 , e.g., $\nu_T = \nu_{31} = \nu_{32}$ and $G_T = G_{23} = G_{31}$. For the material constants with respect to the isotropic plane, we have the relation:

$$E/G = 2(1 + \nu). \quad (14)$$

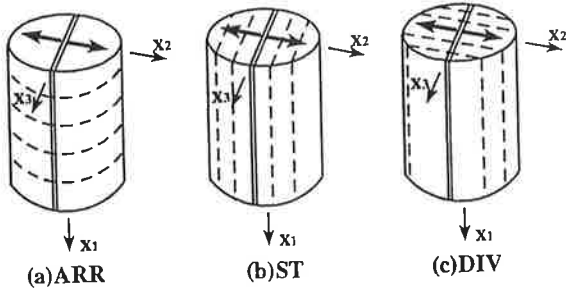


Figure 4: Arrester (ARR), short transverse (ST) and divider (DIV) configurations of transversely isotropic SR specimens.

In order to have a positive definite strain energy, we further have the restriction $1 - \nu > 2\nu\nu_T^2$. From eqs.(13) and (14), transversely isotropic materials still have five independent constants. It is, therefore, necessary to reduce the number of independent anisotropic parameters in numerical calculations. Extensive data from Lama & Vutukuri(1978, Table 9) show that the approximation $G_T \approx E/(1 + n + 2\nu\nu_T)$ suggested by Lekhnitskii(1977) is often useful and ν is approximately given by $\nu = n^{-1}\nu_T$. The Young's modulus E is, furthermore, chosen as 50GPa for all calculations as shown in the sequel. This leaves either n or ν_T as the independent anisotropy parameter in our calculations.

Suppose that the axes of the transverse isotropy coincide with the coordinate axes of the specimens as shown in Fig. 1. In such a case, there remain three choices for the orientation of the isotropic plane: arrester (ARR), short transverse (ST), and divider (DIV) configurations in which the isotropic plane coincides with the x_2 - x_3 , x_3 - x_1 and x_1 - x_2 coordinate planes, respectively. In Fig. 4, the orientations of the isotropic planes are schematically illustrated for ARR, ST and DIV configurations of transversely isotropic SR specimens.

5 NUMERICAL RESULTS

The local K_I distributions along the crack front segment B (see Fig. 1) are calculated by substituting the nodal displacements determined by the finite element method into eq.(12). In the sequel, the rms average $\bar{K}_I$ of the local K_I values is used to evaluate the fracture toughness of anisotropic rocks. In order to simulate the crack propagation process, various values of a/D are chosen for CB and SR specimens, i.e., $a/D = 0.2, 0.25, 0.3, 0.35$ and 0.4 for CB and $a/D = 0.6, 0.7, 0.8, 0.9$ and 1 for SR.

Since the problem is a three dimensional one, the crack front is in an intermediate state between plane strain and plane stress. In the present work, however, a plane strain state is assumed at the crack tip. For the plane strain, b_{ij} defined as $b_{ij} = a_{ij} - a_{i3}a_{j3}/a_{33}$ are employed instead of a_{ij} appearing in eqs.(1) to (12).

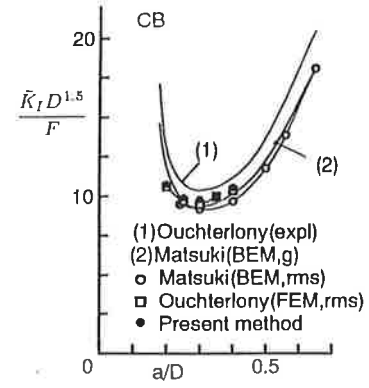


Figure 5: Non-dimensional stress intensity factors as a function of a/D for the isotropic CB specimen.

5.1 CB specimens

We first calculated the stress intensity factor for an isotropic material to confirm the accuracy of our method. Fig. 5 shows the non-dimensional stress intensity factor $\bar{K}_I D^{1.5}/F$ as a function of a/D . Here D is the diameter of the specimen and F denotes the applied force. The Poisson's ratio ν is taken as 0.25. Our results are in good agreement with other published solutions.

Fig. 6 shows $\bar{K}_I D^{1.5}/F$ as a function of a/D for various values of $n = E/E_T$. The Poisson's ratio ν_T is taken as 0.25. Figs. 6 (a) to (c) correspond to the results for the ARR, ST and DIV configurations, respectively. It can be seen that $\bar{K}_I$ increases or decreases monotonically with the variation of the anisotropic parameter n , yet the deviations from the isotropic case ($n = 1$) are limited within only a few percent. Hence, the effect of the parameter n on the fracture toughness of the CB specimen is quite small. From Fig. 6, it is also noted that the minimum value of $\bar{K}_I$ is observed at $a/D \approx 0.3$ for all cases.

Fig. 7 shows the $\bar{K}_I D^{1.5}/F$ as a function of a/D for various values of ν_T . The arrester orientation of the isotropic plane is considered and n is taken as 2.08. It can be seen that the effect of the Poisson's ratio on $\bar{K}_I$ is smaller than that of the parameter n . For the other configurations ST and DIV, the effect of the Poisson's ratio is also negligible.

5.2 SR specimens

As in the case of the CB specimen, the stress intensity factor for the isotropic SR specimen is firstly calculated. Fig. 8 shows the non-dimensional stress intensity factor $\bar{K}_I D^{1.5}/F$ as a function of a/D . The Poisson's ratio ν is taken as 0.25. Again, our results are in good agreement with other published solutions.

Fig. 9 shows $\bar{K}_I D^{1.5}/F$ as a function of a/D for various values of $n = E/E_T$. The Poisson's ratio ν_T is taken as 0.25. It is clear that $\bar{K}_I$ for the ARR and ST specimens is strongly affected by the anisotropy of the material. The maximum deviation due to the anisotropy amounts to twenty percent or more of the isotropic value. It is also observed that the minimum value of $\bar{K}_I$ is located at larger (ARR) and smaller (ST) values of a/D as the parameter n increases. As for the Poisson's ratio,

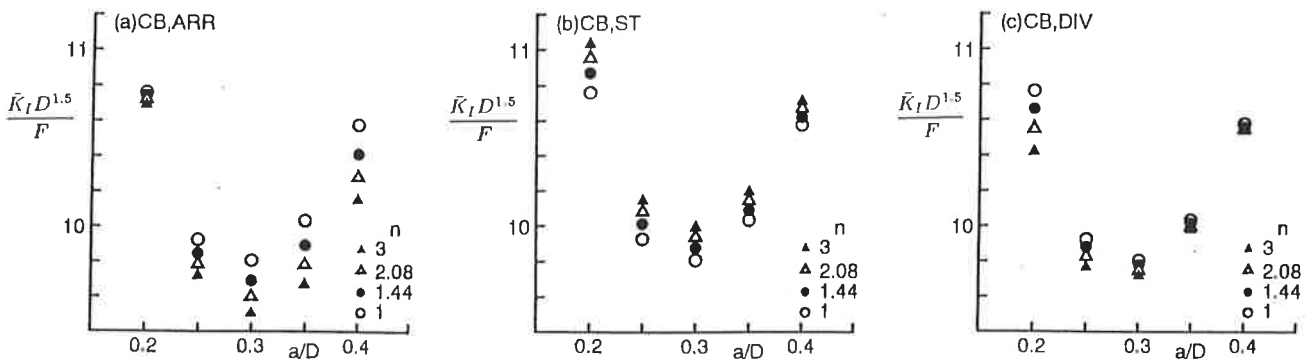


Figure 6: Variation of non-dimensional stress intensity factors with a/D for various values of n for transversely isotropic CB specimens: (a) ARR, (b) ST and (c) DIV.

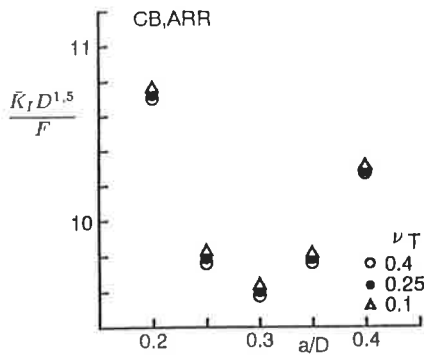


Figure 7: Variation of non-dimensional stress intensity factors with a/D for various values of ν_T for transversely isotropic CB specimens: Arrister configuration.

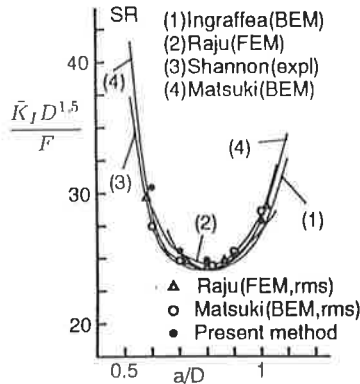


Figure 8: Non-dimensional stress intensity factors as a function of a/D for the isotropic SR specimen.

the effect on $\bar{K}_I$ is not significant for the SR specimen either, as in the case of CB specimen.

As a summary, Fig. 10 shows the minimum values of $\bar{K}_I$ as a function of the parameter n for the SR specimen as well as the CB specimen. For each specimen, ARR, ST and DIV configurations are considered. $\bar{K}_I$ is normalized by the stress intensity factor for the isotropic material denoted by $\bar{K}_I^{iso}$. As shown before, a significant effect of the parameter n is found for the ST and ARR configurations of SR specimens. It is noted that the relation between $\bar{K}_I$ and $\log n$ is almost linear. This relation provides quite a simple calibration formula for the evaluation of the fracture toughness of anisotropic rocks.

6 ACKNOWLEDGEMENT

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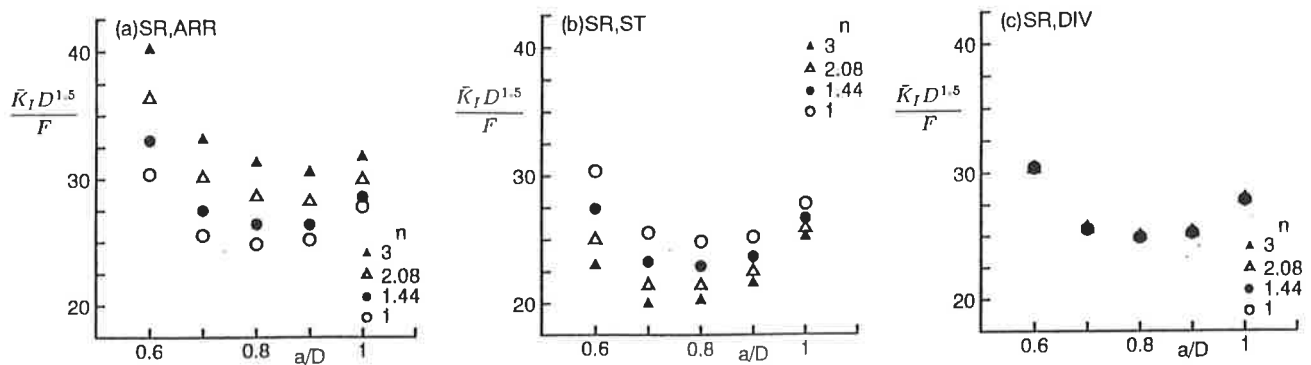


Figure 9: Variation of non-dimensional stress intensity factors with a/D for various values of n for transversely isotropic SR specimens: (a) ARR, (b) ST and (c) DIV.

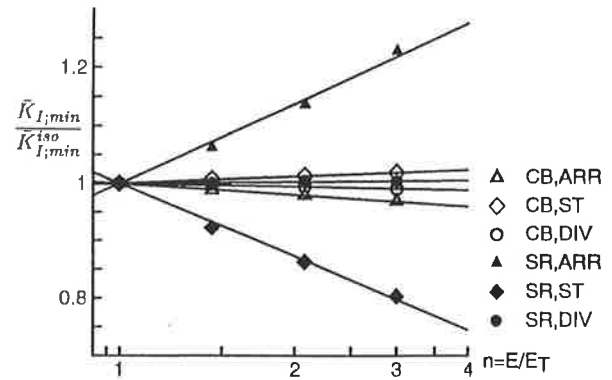


Figure 10: Minimum values of $\bar{K}_I$ as a function of n for the transversely isotropic CB and SR specimens.

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An analytical and experimental investigation of rock indentation fracture

Etude analytique et expérimentale de la fracture par entaille des roches

Eine analytische und experimentelle Untersuchung von Eindruckbrüchen an Felsgestein

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ABSTRACT: Through a similarity analysis based on a large number of experimental data for rock indentation, relationships between indentation force and either indentation depth or length of radial/median crack are established. These forces are induced by hemispherical, truncated, or cylindrical indenters. The formulas obtained can serve for general use in various kinds of rock with different diameters of the three indenter types. This analysis provides a great deal of information for practical use, such as full face boring and percussive drilling.

RESUME: A travers une analyse de similitude basée sur un grand nombre de données expérimentales sur l'indentation de la roche, on a établi les relations entre la force d'indentation et soit la profondeur d'indentation soit la longueur de la fissure radiale/médiane induite par des indentateurs hémisphériques, tronqués, et cylindriques. Les formules obtenues peuvent servir à une utilisation générale dans différentes catégories de roches avec des diamètres différents pour les trois types d'indentateurs. Avec cette analyse, on a obtenu d'importantes améliorations pour les applications pratiques, comme pour le forage par explosion et le forage par percussion.

ZUSAMMENFASSUNG: Durch eine Similaranalyse einer großen Anzahl von experimentell gewonnenen Daten zur Gesteinsinvasion werden Verhältnisse zwischen Intrusionskraft und Intrusions-Tiefe oder zwischen Intrusionskraft und Länge der radialen/medianen Spalte hergestellt. Diese Kräfte werden durch hemisphärische, stumpfe oder zylindrische Druckwerkzeuge verursacht. Die erhaltenen Formeln können allgemein auf verschiedene Gesteinsarten mit verschiedenen Durchmesser der Druckwerkzeuge angewandt werden. Diese Analyse liefert viele Informationen für die Praxis, beispielsweise beim Vollschnittvortrieb und bei der Schlagbohrung.

1 INTRODUCTION

Full face boring, percussive drilling, and drag bit cutting are today the most frequently used mechanical excavation methods for medium or hard rocks. However, the basic mechanisms involved in them are still not fully understood. Rock indentation by a bit is the fundamental mechanism in most mechanical excavation methods. Therefore, the investigation of the indentation mechanisms is important for improvement of rock excavation methods.

The main purpose of the present study is to propose a reasonable model for predicting the indentation depth and the radial/median crack distribution in rocks induced by different types of indenter. The former is important for the improvement of rock excavation efficiency. The later is important for the analysis of the hydraulic conductivity in the near-field of a nuclear waste storage. It can also contribute to understanding of the mechanisms of mechanical excavation and the improvement of excavation tools.

2 ROCK FRACTURES UNDER INDENTATION

Four types of Swedish rock, namely Kallax gabbro, Bohus granite, Ekeberg marble, and Lemunda sandstone, were tested in indentation by using two displacement-controlled rock mechanics testing machines, Instron 0-100 kN and Instron 0-4500 kN, with maximum loads ranging from 15 to 150 kN. Their mechanical properties are given in Table 1, where σ_c , E , ν and G are uniaxial compressive strength, Young's modulus, Poisson's ratio and energy release rate, respectively. Three kinds of indenters, namely hemispherical, truncated and cylindrical, were used in the indentation tests. The dimensions of the rock sample were selected to avoid if possible crack propagation longer than half way through the sample for the strongest rock loaded to 100 to 150 kN. The sample diameters were much larger than the diameters of the indenters, and confined in a manner similar to the confinement

Table 1. Mechanical properties of tested rocks¹

Mechanical properties	Kallax gabbro	Bohus granite	Ekeberg marble	Lemunda sandstone
σ_c (MPa)	279 [1]	152.8 [2]	160.4 [2]	55.1 [2]
E (GPa)	98.9 [1]	64.1 [2]	84.4 [2]	26.5 [2]
ν	0.31 [1]	0.31 [2]	0.35 [2]	0.38 [2]
G (J/m ²)	51.3 [1]	100-110 [4]	40.5 [3]	113 [3]

¹The figures in the brackets [] indicate the data resources: [1] Vutukuri et al., 1983, [2] Homberg and Forslund, 1989, [3] Ochternony, 1986, and [4] Div. of Rock Mechanics, Luleå Univ. of Tech., 1993.

provided in a semi-infinite rock mass during the indentation process. A constant indentation rate of 0.05 mm/s was applied for all experiments. This approximately corresponds to 1×10^{-8} of the wave speed of rock material and therefore, the load can be treated as static. After indentation tests, the samples were cut in slabs, and the cracks in the samples were observed under a fluorescent light by introducing a fluorescent substance inside them (Lindqvist et al., 1994).

From numerous experimental results, a general picture of fractures in rock under indentation can be summarized as in Fig. 1. Underneath the indenter there is a disintegrated and sometimes partly recompacted zone, a crushed but basically confined zone, and a cracked zone. Outside the cracked zone there are mainly three kinds of cracks: median cracks, radial cracks, and side cracks. In general, fully developed radial cracks on each side of the indented area connect with median cracks and form a radial/median crack system. We may also observe Hertzian cracks in the rock under indentation with an axially-symmetric indenter. Among the various fractures, the indentation depth plays an important role in the evaluation of rock excavation efficiency and the improvement of cutter design. The radial/median cracks are

important for the hydro-conductivity in the near-excavation-field of a nuclear waste storage. We shall therefore concentrate on discussing them in this paper.

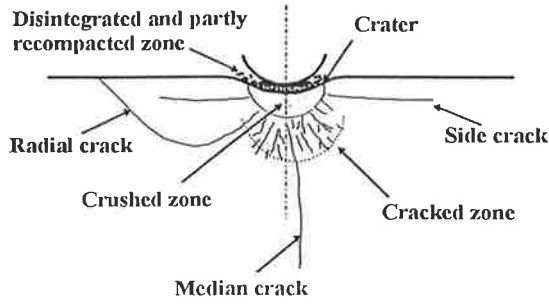


Fig. 1 General picture of cracks in rocks under indentation.

3 ROCK BEHAVIOURS UNDER COMPRESSION

A complete stress-strain relationship of structured coal is shown in Fig. 2 (Kou et al. 1990). It is typical for various kinds of rock-like material, although it was obtained from the experiments using the so-called structured coal. The stresses corresponding to the letters d, c, and r in the figure are yield strength (defined as the onset of dilatancy), limit strength, and residual strength, respectively. These three strengths split the complete curve into four regions, i.e., elastic region, initial damage region, post failure region, and shearing and slipping region. In addition, it seems reasonable from the test (Kou, et al., 1990, Bieniawski, 1968) to assume that all of the three strengths have linear relationships with the values of confining stress.

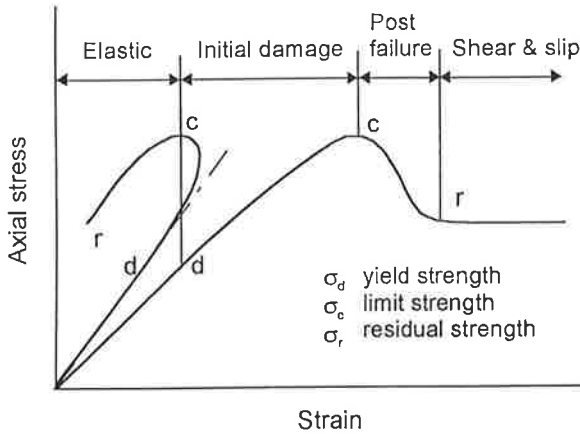


Fig. 2 Typical stress-strain curves of structured coal under triaxial compression (Kou et al., 1990).

The relationship between the physical processes during indentation and the mechanical properties of the rock material can be defined as

1. Rock material behaves elastically in the far-indenter-field, where the main long cracks propagate.
2. The initial damage region is due to the growth and formation of microcracks during deformation, and it, therefore, corresponds to the cracked zone.
3. In the post failure region, the damage of the rock material has already amounted to such a level that no extra stress can be sustained by the material and, therefore, this corresponds to the crushed zone.
4. In the shearing and slipping region the material is disintegrated into granular rock without any cohesion and it, therefore, corresponds to the disintegrated and partly recompacted zone.

From the boundary of the crushed zone (some times together with side cracks) a crater is formed after removal of the indenter.

4 SIMILARITY ANALYSIS OF INDENTATION DEPTH AND LENGTH OF RADIAL/MEDIAN CRACK

In order to do an effective analysis, we assume that: the indenter is rigid, the tested rock mass is semi-infinite, the indentation load is static, and the indentation process is isothermal. The pertinent physical parameters which control the "near-field" phenomenon of rock indentation are then the indentation load magnitude, F ; the geometry of the indenter, i.e., the indenter diameter d and the other normalized geometrical parameter d_i ; the parameters characterizing the rock material, i.e., the grain size d_g , the uniaxial compressive strength σ_c , the other normalized characteristic strengths σ_i , Young's modulus E , and the energy release rate G . Indentation depth p is introduced to describe a particular response of the rock to the indentation. We assume that there is a physical law of the form

$$f(F, d, d_i, d_g, \sigma_c, \sigma_i, E, G, p) = 0. \quad (4.1)$$

Only seven independent dimensionless products can be formed from the nine quantities. There is then an equivalent physical law between the independent dimensionless quantities according to the well-known π theorem in similarity analysis (Baker, et al., 1978). The relationship between the seven π terms can be written as:

$$\frac{p}{d} = f(d_i, \frac{d_g}{d}, \sigma_i, \frac{E}{\sigma_c}, \frac{F}{\sigma_c d^2}, \frac{G}{\sigma_c d^2}). \quad (4.2)$$

We further assume that the rock material is homogeneous and isotropic, and constitutive similarity exists for different types of rock, which means that the various types of rock have identically scaled strength and stiffness properties so that stress normalized by the corresponding Young's modulus is identical for all strains for these rocks. In addition, the observation of the experimental results can provide a further simplification, and then Eq. (2) becomes

$$\frac{pG}{d^2 \sigma_c} = f(d_i, \frac{F}{\sigma_c d^2}), \quad (4.3)$$

where d_i characterizes the indenter shape. Eq. (3) can be specified by a comparison with the experimental results for individual indenter type, and this gives the following three equivalent equations: for hemispherical indenter,

$$\frac{p}{d} \left(\frac{G}{\sigma_c d} \right) * 10^6 = -1.265 + 2.239 \left(\frac{F}{\sigma_c d^2} \right), \quad (4.4)$$

for truncated indenter,

$$\frac{p}{d} \left(\frac{G}{\sigma_c d} \right) * 10^6 = -3.633 + 1.263 \left(\frac{F}{\sigma_c d^2} \right), \quad (4.5)$$

and for cylindrical indenter,

$$\frac{p}{d} \left(\frac{G}{\sigma_c d} \right) * 10^6 = -15.99 + 2.692 \left(\frac{F}{\sigma_c d^2} \right). \quad (4.6)$$

The comparison between the experimental data and Eq. (4) is shown in Fig. 3.

By a procedure similar to that in the discussion about the indentation depth, we may obtain three equations to describe the lengths of the radial/median cracks induced by different indenters. When the crack length and Poisson's ratio are represented by c and ν respectively, these formulas become: for hemispherical indenter,

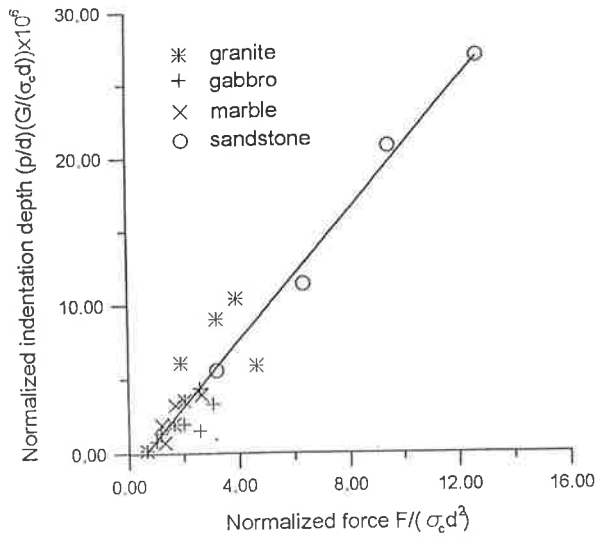


Fig. 3 Indentation depth vs. force in nondimensional form for hemispherical indenter.

$$c^{3/2} \left(\frac{EG}{1-\nu^2} \right)^{1/2} / F = 0.02803 + 0.03088 \left(\frac{F}{\sigma_c d^2} \right), \quad (4.7)$$

for truncated indenter,

$$c^{3/2} \left(\frac{EG}{1-\nu^2} \right)^{1/2} / F = -0.01632 + 0.01160 \left(\frac{F}{\sigma_c d^2} \right), \quad (4.8)$$

and for cylindrical indenter,

$$c^{3/2} \left(\frac{EG}{1-\nu^2} \right)^{1/2} / F = -0.05805 + 0.03987 \left(\frac{F}{\sigma_c d^2} \right). \quad (4.9)$$

The comparison between the experimentally obtained crack length and Eq. (7) is shown in Fig. 4.

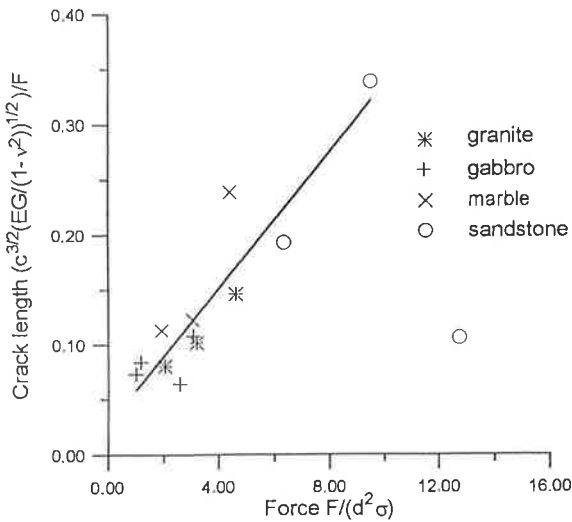


Fig. 4 Median/radial crack length vs. force in nondimensional form for hemispherical indenter.

All of these formulas, i.e., Eqs. (4) to (9), are valid when the force F is so large that the right sides of the formulas are not negative. The values of the crack lengths c are defined as zero when their right sides are negative. The quantities in the equations are in SI units, i.e., the force is in a unit of N, the stress in Pa, energy release rate in J/m^2 , and the length in m.

5 DISCUSSIONS AND CONCLUDING REMARKS

5.1 Relationship between indentation depth and loading force

The indentation depths in the four types of rock induced by a hemispherical indenter with various magnitudes of load are shown in Fig. 5. The intercepts on the ordinate show the threshold of the force to start penetration for different types of rocks. They are 4.5 kN for Lemunda sandstone, 12.4 kN for Bohus granite, 13.1 kN for Ekeberg marble, and 22.7 kN for Kallax gabbro with the indenter diameter of 12 mm. However, the order can be changed at a high load level. For example, at a load of 300 kN, the

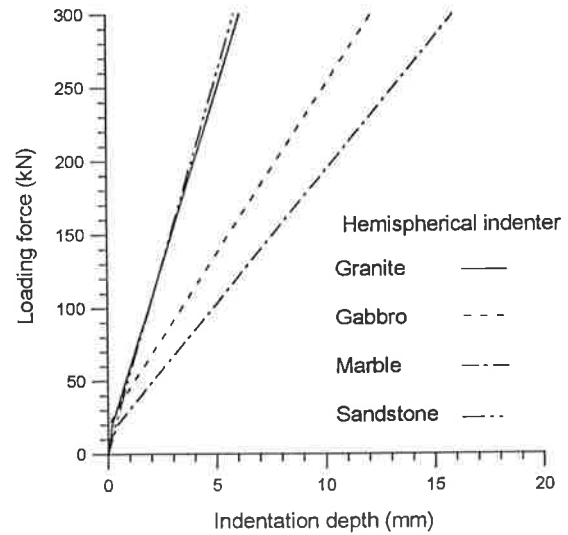


Fig. 5 Relationship between indentation depth and loading force for different rocks induced by a hemispherical indenter.

indentation depths for these four kinds of rock are in a proper order of marble, gabbro, granite, and sandstone from the larger to the smaller. Therefore, the load level control is important both in the design of percussive drilling and in practical operation if one wishes to obtain a maximum indentation depth in each percussion.

From Eqs. (4.4) to (4.6), we may also have relationships between indentation depth and loading force for a particular rock with different indenter types. For example, the loads to start penetrating the granite rock are 12.4, 22.7, and 130.5 kN for hemispherical, truncated, and cylindrical indenters respectively with the indenter diameter of 12 mm. In Bohus granite, a hemispherical indenter produces the largest indentation depth among the three indenters all the way until 300 kN, which stresses the importance of regrounding blunt indenters.

5.2 Relationship between crack length and loading force

The radial/median crack lengths in the four types of rock induced by a hemispherical indenter with various magnitudes of load according to Eq. (4.7) are shown in Fig. 6. The crack lengths in sandstone are the longest among the four rocks, followed by marble, granite, and gabbro. That explains why some of the samples of sandstone totally cracked (Lindqvist, et al. 1994) when we observed the design principle of experimental specimens described in section 2. Among the three types of indenter, a hemispherical one produces the longest lengths of radial/median cracks, followed by truncated and cylindrical indenters.

5.3 Relationship between crack length and indentation depth

Fig. 7 shows a relationship between length of radial/median crack

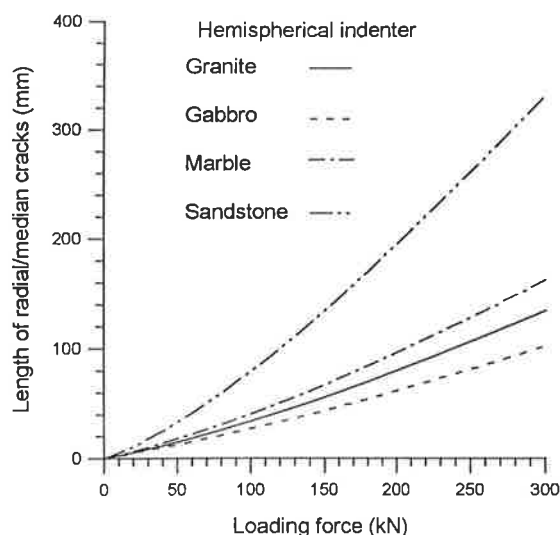


Fig. 6 Relationship between radial/median crack length and loading force for different rocks induced by a hemispherical indenter.

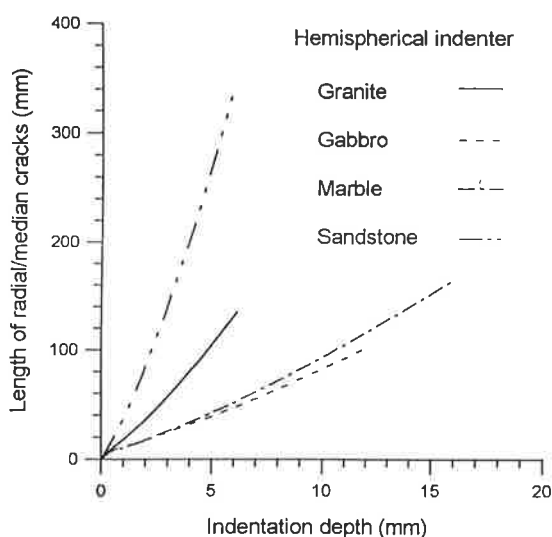


Fig. 7 Relationship between radial/median crack length and indentation depth for different rocks induced by a hemispherical indenter.

and indentation depth, where the ending point of each curve corresponds to 300 kN. From this figure, one may notice that to cut gabbro by a hemispherical indenter produces the shortest crack length among the four rock types with the same indentation depth, followed by marble, granite, and sandstone. In addition, one could possibly produce a shorter crack length by having a short indentation depth of each cut. This should be considered in the practical operation of both percussive drilling and full face boring.

5.4 Modification of relationships by including confinement

Confinement is an important factor affecting the rock fracture under indentation. The indentation depth predicated by Eqs. (4.4) to (4.6), and the crack lengths by Eqs. (4.7) to (4.9) are overestimated if the indentation is carried out underground. However, if the confining stress is not too high, the failure criterion in triaxial compression can be quite accurately given by

$$\sigma_c = \sigma_0 + q\sigma_p, \quad (5.1)$$

where σ_c , σ_0 and σ_p are triaxial compressive strength, uniaxial

compressive strength, and confining stress, respectively. According to the experimental results, the coefficient q for structured coal (Kou et al., 1990) can vary between 1.3 to 3.9, and it was set to 5 for Sierra granite by Cook, et al. (1984). Replacing σ_c in Eqs. (4.4) to (4.9) by Eq. (5.1), we can obtain six modified formulas to include the influence of the confining stress higher than that provided in the semi-infinite case. In this way we may estimate the indentation depths and crack lengths in deep underground by superimposing the tectonic pressure to the uniaxial compressive strength according to Eq. (5.1). In addition, we may also see that there is a possibility of increasing the penetration depth and crack length due to reduction of the confining stress. Especially, the indentation depths and lengths of cracks might be very long when the cutters or button bits work in the area close to weakness, such as faults, joints, or free surfaces.

CONCLUDING REMARKS: The present analysis results in an establishment of relationships between indentation force and either indentation depth or length of radial/median crack induced by hemispherical, truncated, or cylindrical indenters. The relationships depend on rock properties, diameter of the indenter, and the indenter shape. Good agreement between the model and the experimental data is obtained. A modification to the uniaxial compressive strength obtained in ordinary rock mechanics laboratories makes it possible to include the influence of confinement and, therefore, excavation at depths can be taken into account. The formulas obtained can serve for general use in various kinds of rock with different diameters of the three indenter types. This analysis provides a great deal of information for practical use, such as design of cutting tools and the operation procedure in full face boring and percussive drilling.

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Micromechanical study of rock deformation behaviour

Etude micromécanique du comportement de déformation des roches

Mikromechanische Studie van Fels-Deformationsverhalten

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ABSTRACT: The deformation of brittle rock is studied using a mechanical model that is developed based on the analyses of the microscopic fracture processes within rock. The simulations demonstrate that the elastic parameters (E and ν) for the rock matrix and for the fractured specimen can be measured during the unloading and reloading cycle; the axial splitting failure brings about more abrupt bursts than the shear faulting failure; and the size of specimen affects the post-peak behaviour.

RESUME: La déformation des roches fragiles est étudiée à l'aide d'un modèle mécanique qui est basé sur l'analyse du processus de microfracture à l'intérieur de la roche. Les simulations démontrent que les paramètres d'élasticité (E et ν) de la matrice de la roche et de l'échantillon fracturé peuvent être mesurés durant le cycle de décharge-recharge; la rupture axiale provoque plus d'explosions brutales que la rupture en cisaillement; et la taille de l'échantillon affecte le comportement post-pic.

ZUSAMMENFASSUNG: Die Deformation von Spödgestein wurde untersucht anhand eines mechanischen Modells, das auf den Analysen von mikroskopischen Bruchvorgängen im Gestein basiert. Die Simulationen zeigen, daß die elastischen Parameter (E und ν) der Gesteinsmatrix und der gebrochenen Proben während des Austragtaktes und der Frischladungsaufnahme gemessen werden können; das Versagen durch axiale Risse bewirkt ein abrupteres Bersten als das Versagen durch Scherbruch; und die Größe der Proben beeinflusst das Post-Peak-Verhalten.

1 INTRODUCTION

In underground excavations, it is often observed that a great number of parallel cracks appear in mine pillars and walls (Coates, 1981). Obviously, the cracks are induced under compression. Sometimes, pillars fail completely due to axial splitting, but most of the time pillars fail in a combination of spalling and shear faulting. In this case, axial cracks are initiated and grow to some extent within a pillar under compression. Then, spalling occurs in the middle of the pillar and develops towards the centre. At the final stage, pillars fail due to the shear faulting along the conjugate planes crossing the centre of the pillar or due to crushing.

Hori and Nemat-Nasser (1985, 1986) conducted a series of experiments using resin specimens containing a single flaw or a large number of flaws that were different in length to examine the fracture pattern of the specimens under compression. For the specimen containing a single flaw, wing cracks were initiated at the tips of the flaw under compression. Then, they tended to grow in the loading direction. For specimens containing a large number of flaws, it was reported that they failed in the form of either axial splitting under uniaxial compression or shear faulting under biaxial compression. Wing cracks were first initiated at the tips of those large flaws and grew in the direction of the maximum compressive stress. The wing cracks continued to grow with an increase in the uniaxial compressive stress. Finally, they coalesced and formed axial splitting fractures through the specimen. However, the fracture pattern was different when confining pressure existed. Wing cracks were first initiated at the tips of large flaws, but their growth was soon suppressed by the confining pressure. Wing cracks were then initiated on a large number of small flaws and coalesced to form shear faults.

It is concluded, based on the observations in the field and the laboratory experimental results, that axial splitting is the basic local fracture pattern for brittle rocks under compression. In the pre-peak stage, axial splitting predominates the fracture process, while in the post-peak stage shear faulting does. A constitutive model has been developed for brittle rocks based on the analyses

of the fracture process within rock. The deformation of rock, in this model, is decomposed into four components which are caused by (i) the elastic response of rock matrix, (ii) the closure of open cracks, (iii) the growth of cracks, and (iv) the shear faulting, respectively. In the pre-peak stage only the first three are considered, but all four components are involved in the post-peak stage.

In this article, we only give a brief review of the model. The details of the model can be referred to Li and Nordlund (1993) and Li (1993, 1995). Emphases will be put on studies of the rock fracture and deformation behaviour using the model. The studies show that the apparent Young's modulus and Poisson's ratio for the rock matrix and for the fractured specimen can be measured in the unloading and reloading periods, respectively. The study of the post-peak behaviour demonstrates that axial splitting brings about more abrupt bursts than shear faulting. The class I and II load-displacement behaviour of rock is associated with the size of specimen. Class I behaviour might be transformed to class II when the height of specimen is increased.

2 BRIEF DESCRIPTION OF THE MODEL

The model is developed under the two-dimensional condition. The 2-D specimen is subjected to compression by σ_1 and σ_2 , see Figure 1a. It has been assumed that the original microcracks are distributed uniformly in the specimen and that all the microcracks are initially open and have an average length of $2a_0$.

In the early stage of loading, two processes occur within the specimen. The first one is the elastic response of the rock matrix. The second one is the closure of open cracks. Kranz (1979) and Sprunt *et al.* (1974) reported that the aspect ratio of cracks, α , (defined as the ratio of the opening to the length) in rock was between 10^{-5} and 10^{-3} . Under hydrostatic pressure, all the open cracks can easily be closed at the same time, no matter how they are oriented. The hydrostatic crack-closure stress σ^c is directly proportional to the aspect ratio as follows:

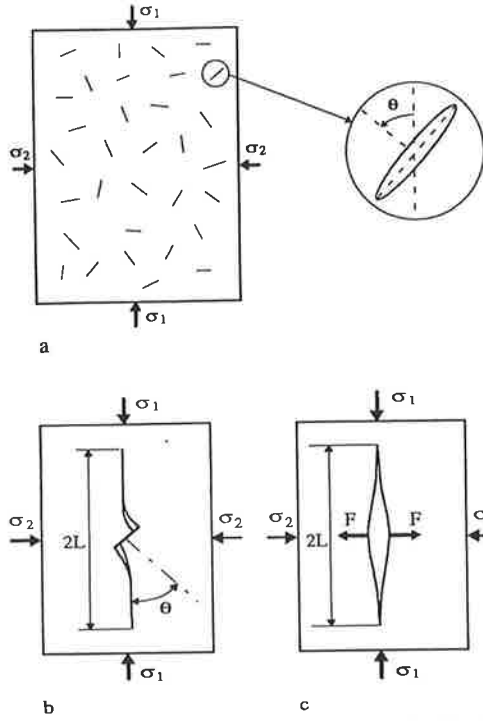


Figure 1. (a) A 2-D specimen containing uniformly distributed microcracks. (b) Wing cracks are initiated and propagated in the direction of the maximum compressive stress. (c) A crack loaded by a central wedge force, which is a simplification of the growing wing cracks.

$$\sigma^c = \frac{2\alpha E}{(1+\nu)(1+\chi)} \quad (1)$$

where E is the Young's modulus, ν Poisson's ratio, and $\chi = (3-\nu)/(1+\nu)$ for the plane stress and $(3-4\nu)$ for the plane strain. Under uniaxial compression, the crack-closure stress σ^c depends not only on the aspect ratio but also on the angle of crack θ (see Figure 1a). It is expressed as

$$\sigma_1^c = \frac{2\alpha E}{(1+\nu)(1+\chi)\cos^2\theta} \quad (2)$$

It is seen from equation (2) that the cracks perpendicular to the loading direction are most easily closed under uniaxial compression. The crack-closure stress increases rapidly with the increase in the angle of crack. The nearly vertical cracks (*i.e.* $\theta \approx 90^\circ$) actually could not be closed.

When the open crack is completely closed, wing cracks will be initiated at its two tips under the shear stress on the crack plane. Both theoretical and experimental studies have proven that the wing cracks are initiated at an angle of about 70° to the crack plane. Using the fracture criterion for Mode II, we obtain the crack initiation stress as shown in Figure 2. Wing cracks are difficult to be initiated on those cracks whose angle θ is close to either 90° or to the friction angle ϕ . The aspect ratio has a small influence on the crack initiation stress (see Figure 2a). The confining stress retards the crack initiation through increasing the friction resistance on the crack surface (see Figure 2b). The cracks, on which wing cracks are most easily initiated, are located in the range of θ from 40° to 70° under the condition of the friction angle $\phi = 30^\circ$.

As soon as the wing cracks are initiated, they tend to grow in the direction of stress σ_1 (see Figure 1b). The wing cracks either continue to grow until the specimen fails in the form of splitting, which occurs only under uniaxial compression, or stop growing in the direction of σ_1 at some stage and coalesce and form shear faults under the shear stress, which occurs more often in the

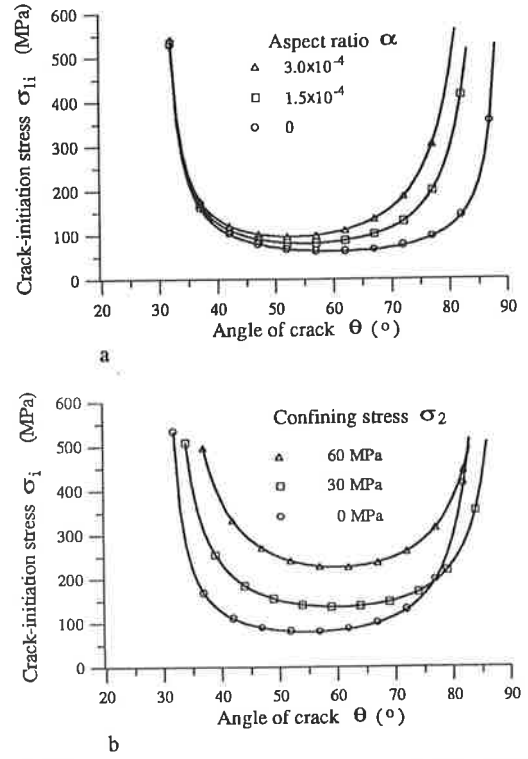


Figure 2. The relationship between the crack initiation stress and the angle of crack (the friction angle $\phi = 30^\circ$). (a) Under uniaxial compression. (b) Under biaxial compression. ($\sigma_i = \sigma_{ii} - \sigma_2$, and the aspect ratio $\alpha = 1.5 \times 10^{-4}$).

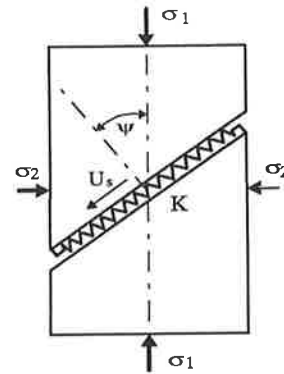


Figure 3. Schematic illustration of the conceptual model for shear faulting.

presence of the confining stress.

The formation of shear faults is simplified and represented by a faulting plane as illustrated in Figure 3. The normal of the plane is inclined to the direction of σ_1 at an angle of ψ . Supposing that the faulting occurs in the direction of the maximum shear stress, we have $\psi = (\pi/4 + \phi/2)$ where ϕ is the internal friction angle. The progressive shear faulting process is represented by the decrease in the shear stiffness of the faulting plane, K , defined as

$$K = K_0(1-\delta) \quad (3)$$

where K_0 is the initial shear stiffness and δ the shear faulting factor. The factor δ varies in the range from 0 to 1, where $\delta=0$ corresponds to no faulting and $\delta=1$ to the state that the shear fault has been formed through the specimen.

As stated before, there exist four processes within a rock specimen under compression, which are the elastic response of rock matrix, the closure of open cracks, the growth of wing cracks, and the shear faulting. Their corresponding deformation

components are designated as D_e , D_c , D_f and D_s , respectively. The total deformation of the specimen is the summation of them, that is

$$D = D_e + D_c + D_f + D_s \quad (4)$$

Hooke's law is used to calculate D_e and D_c is the sum of the closure of all cracks within the specimen. In deriving the equation for the calculation of D_f , the wing crack is simplified into a growing crack loaded by a central wedge force F expressed as (see Figure 1c)

$$F = a_o [(\sigma_1 - \sigma_2)g_1(\theta, \phi) - 2(\sigma_1 - \sigma_c) \cos \theta \tan \phi] \quad (5)$$

where $g_1(\theta, \phi) = 2 \cos^3 \theta (\tan \theta - \tan \phi)$. The length of the growing crack has the following relationship with the compressive stresses:

$$L = \frac{1}{4\pi} \left[\frac{\sqrt{K_{IC}^2 + 4\sigma_2 F} - K_{IC}}{\sigma_2} \right]^2 \quad (6)$$

where K_{IC} is the fracture toughness. The lateral displacement of the crack contour, u^L , has been derived, and is expressed as

$$u^L = \frac{4}{\pi E} L^3 N^3 \left[\ln \left(\frac{L}{L_o} \right) + 1 \right] F \quad (7)$$

where N is the crack density expressed as the traverse-crack intersections per unit length, L is the length of the growing crack and L_o the initial value of L . The calculation of D_f can be done using u^L of all growing cracks.

The faulting plane divides the specimen into two blocks. The analysis of the shear displacement along the faulting plane is similar to that for a rock joint (Brady *et al.*, 1985). The sliding along the faulting plane is resisted by the shear stiffness K and the friction resistance on the plane. The equilibrium equation is expressed as

$$KU_s = P_t - \delta P_n \tan \phi_f \quad (8)$$

where U_s is the relative shear displacement of the two blocks, and P_t and P_n are the shear and normal forces on the faulting plane, respectively, and ϕ_f is the friction angle of the faulting plane. Then D_s is calculated using U_s .

3 STUDY OF THE ROCK BEHAVIOUR BY THE MODEL

Figure 4 shows the comparisons of the model simulations with the experimental data for four types of rock. The experimental curves for the basalt and Tennessee marble are after Wawersik and Fairhurst (1970). The simulations involved only axial splitting fractures for the Malmberget gneiss specimen, but both axial splitting and shear faulting for the basalt and Tennessee marble specimens. The model simulated the rock behaviour well.

3.1 The apparent elastic parameters E' and ν'

Figure 5 shows the stress-strain curves simulated under uniaxial compression. The nonlinear "toe" of the stress-axial strain curve is due to the closure of open microcracks, while the nonlinear behaviour occurring at high stress levels is due to the growth of wing cracks. The hysteresis is formed under cyclic loading due to the friction on closed crack surfaces. The apparent Young's modulus E' and the apparent Poisson's ratio ν' vary during the deforming process due to the fracturing within the rock. In other words, E' and ν' are quantitative measures for the fracturing process. Based on the curves in Figure 5, E' and ν' are calculated and illustrated in Figures 6 and 7, respectively. In the simulations, the Young's modulus for the rock matrix is defined as 75 GPa and the Poisson's ratio is 0.2.

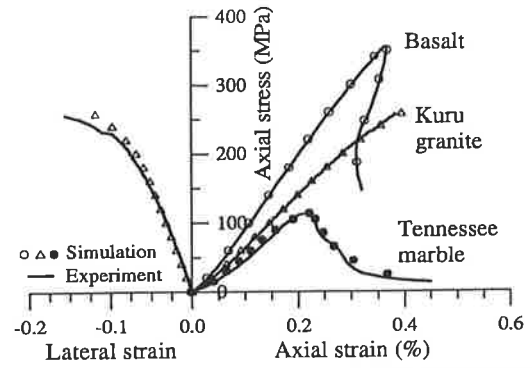


Figure 4. Comparisons of the model simulations with the experimental data.

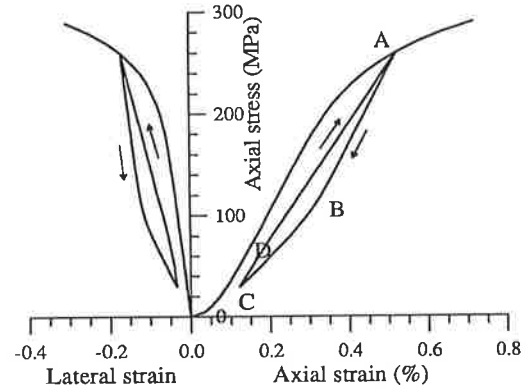


Figure 5. Model simulations of the stress-axial and lateral strain curves.

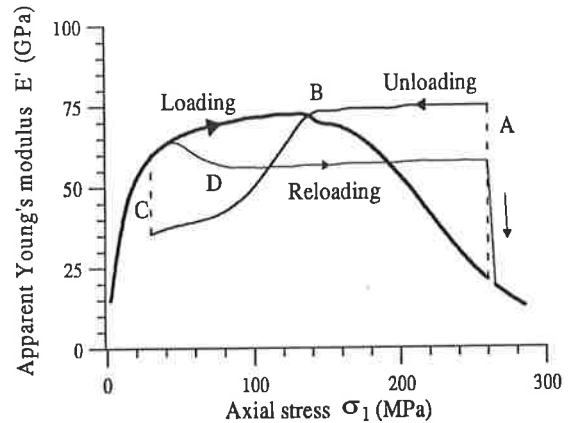


Figure 6. Variation of the apparent Young's modulus during the loading, unloading, and reloading processes.

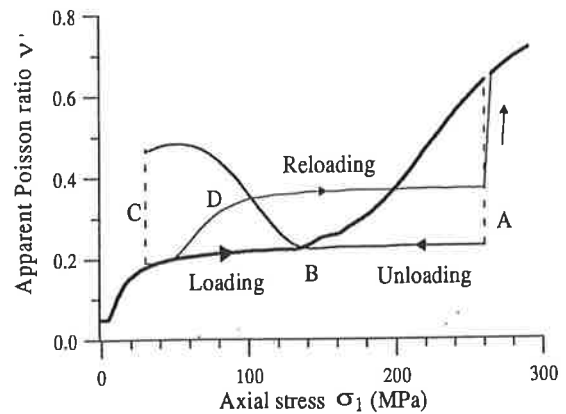


Figure 7. Variation of the apparent Poisson's ratio during the loading, unloading, and reloading processes.

Figure 6 shows that the apparent Young's modulus of the specimen is much lower than 75 GPa at the beginning of loading. This is due to the closure of open cracks. The modulus E' increases rapidly and approaches the level of 75 GPa. The modulus E' starts to decrease with the initiation of new cracks. When the unloading begins at point A (see Figure 5), E' jumps to 75 GPa from a low level in the loading period. The reason for this is that sliding does not occur on the crack surfaces in the transition from loading to unloading until the load drops to a certain level marked by B. Most of the open cracks have been closed at high levels of load. Therefore, the response of the specimen in the period between A and B is only dominated by that of the rock matrix. From this point of view, the apparent Young's modulus measured in this period is the Young's modulus for the rock matrix. When the load drops below the level marked by B, both sliding on the crack surfaces and opening of cracks occur, resulting in the decrease in E' . During the transition process from unloading to reloading, sliding does not occur on the crack surfaces in the beginning. With the increase in the load, sliding gradually occurs and E' reaches an almost constant level in the period between D and A. The apparent Young's modulus in this period is a measure of the fracture developed within the specimen corresponding to the load level marked by A. When the load is beyond point A, new fractures will be developed and E' reverts to the curve for the loading process.

The apparent Poisson's ratio ν' varies in a similar way (see Figure 7). The ratio ν' increases rapidly in the beginning of loading and approaches the level of the Poisson's ratio for the rock matrix, ν . When new fractures are initiated at high load levels, ν' increases rapidly again and remains at the level of ν in the period from A to B in the unloading process. It remains at a relative high level in the period from D to A in the reloading process. The explanation for the variation of the apparent Poisson's ratio is the same as that for the apparent Young's modulus.

It is concluded that the Young's modulus and the Poisson's ratio measured in the section from A to B (see Figure 5) are for the rock matrix, and those in the period from D to A are for the fractured specimen corresponding to the load level marked by A.

3.2 The post-peak behaviour

Wawersik and Fairhurst (1970) reported that two classes of rock behaviour exist in the post-peak stage. For class I, fracture propagation is stable in the sense that work must be done on the specimen for each incremental decrease in load-carrying ability. For class II, fracture is unstable, and the elastic strain energy absorbed in the material is sufficient to maintain fracture propagation until the specimen has lost all strength.

When shear faulting occurs in the post-peak stage, the relative displacement of the two ends of the specimen is the result of the axial component of the shear movement on the faulting plane and the elastic expansion of the rock material. The average strain (defined as the relative displacement of the specimen), is there-

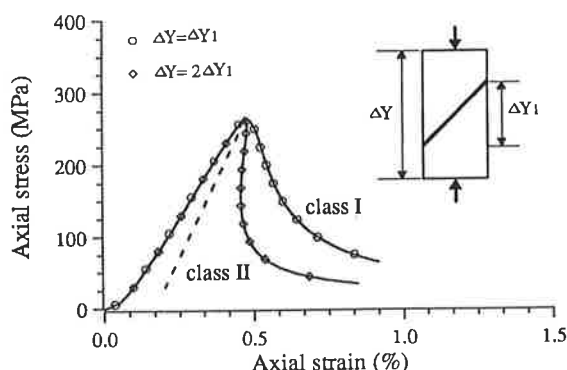


Figure 8. Simulations of the stress-strain behaviour when the height of specimen is changed.

fore not only associated with the properties of rock but also with the length of specimen. Figure 8 illustrates the transformation of the apparent stress-strain behaviour from class I (curve 1) into class II (curve 2) when the length of specimen is increased. This kind of transformation occurs when the elastic expansion of rock material is larger than the displacement caused by the shear faulting. Class II behaviour is likely to occur for a long specimen since it has a large expansion. It is evident that the final failure occurs more abruptly for class II than for class I since more strain energy absorbed within the material will be transformed into kinetic energy for class II. In the case that no shear faulting occurs and the specimen fails completely due to axial splitting, the post-peak curve will drop along the dashed line in Figure 8. It is obvious that the final failure related to the axial splitting failure occurs most abruptly.

4 CONCLUSIONS

1. The confining pressure retards the initiation of new fractures and has a great effect on suppression of the axial splitting.
2. The Young's modulus and Poisson's ratio measured in the straight upper section of the unloading stress-strain curve are for the rock matrix. Those measured in the straight upper section of the reloading stress-strain curve are for the fractured specimen corresponding to the load level at which unloading starts.
3. The axial splitting failure of rock brings about more abrupt failure than the shear faulting.
4. The height of a rock specimen affects its post-peak deformation behaviour. Class I behaviour might be transformed to class II when its height is increased, resulting in more unstable failure.

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Characterisation of fracture apertures – Methods and parameters

Caractérisation des ouvertures de fracture – Méthodes et paramètres

Charakterisierung von Bruchöffnungen – Methoden und Parameter

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ABSTRACT: The aperture distribution of a rock fracture can be studied with different experimental methods. The methods are based on (I) surface topography measurements, (II) resin injection or (III) casting techniques and are briefly reviewed in this paper. Differences between the methods are discussed and parameters that may be chosen in presenting the results are recommended.

RESUME: La distribution de l'espace fissural peut être étudié avec différentes méthodes expérimentales. Ces méthodes sont basées sur (I) des mesures de la topographie de la surface, (II) des techniques d'injection de résines ou (III) de moulages; celles ci sont brièvement passées en revue. Les différences entre les méthodes sont discutées et les paramètres qui pourraient être choisis pour la présentation des résultats sont recommandés.

ZUSAMMENFASSUNG: Kluftöffnungsverteilungen können mit verschiedenen experimentellen Methoden untersucht werden. Die drei meistverwendeten Methoden sind: (I) Messung der Kluft-Oberflächentopographien, (II) Injektion von härtenden Kunststoffen und (III) Abdruckmethoden. Diese Methoden werden zuerst kurz beschreiben, worauf die wesentlichen Unterschiede diskutiert werden. Zum Schluss wird kommentiert, welche Parameter zur Beschreibung der Kluftöffnung mit diesen Methoden ermittelt werden können.

1 INTRODUCTION

Fractures have a major influence on rock mass behaviour, and fracture studies have always been a central part of rock mechanics. Ground water flow in rock is today an important research area, mainly in conjunction with the assessments of underground disposal of hazardous waste. As a consequence, many rock mechanics researchers have focused on the hydromechanical properties of rock fractures during the last decade.

The conductivity of a rock fracture is governed by the geometry of the void space between the two fracture surfaces. This geometry is complex and its detailed nature is not very well known. The aim of this paper is to give a very brief review of methods and parameters that have been used to measure and describe the fracture void geometry.

2 MEASUREMENT METHODS

The measurement methods can be grouped with regard to the basic measurement procedure (see Figure 1). One procedure (I) is to measure the topography of the two fracture surfaces on each side of the void space and to define the aperture as the space between the surfaces. Another procedure (II) is to inject resin into a fracture to fill up the void space. The specimen containing the fracture can then be cut into slices and the aperture be measured as the resin thickness along the fracture intersections on each slice. A third procedure (III) is to make a replica of the void space between the surfaces of a fracture, or to make replicas of the fracture surfaces. The different methods are explained in some more detail as follows.

2.1 Surface topography approach

This approach has been used by Gentier (1986). The topography of a natural granite fracture was measured with a profilometer, a mechanical device which has a stylus that moves on the surface.

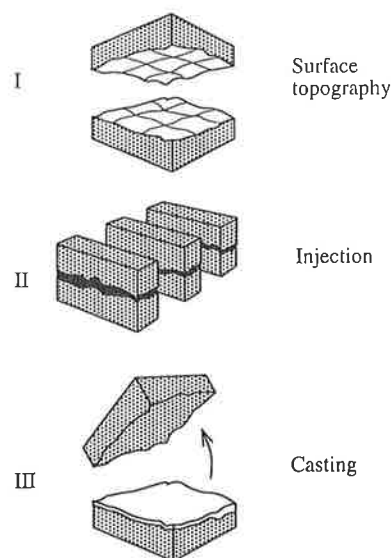


Figure 1. Different approaches to make measurements of fracture void space geometry.

For each fracture surface the surface heights and the location along fracture profiles is continuously recorded. The opposite profiles are matched and the resulting aperture is calculated. The difficulty with this method lies in the matching of profiles since it is difficult to measure the exact relative position of the profiles. Also the size of the profilometer stylus will determine the accuracy of the measurement.

An analogous procedure was used by Iwano & Einstein (1993 and 1995). They used a laser beam profilometer to measure the surface topography of the two fracture surfaces. Five fracture specimens from different rock types were included in their study. The size of each sample was 25 cm² and the spacing between measurement points was 1 mm. The accuracy of surface topography and, hence, of the aperture determination is considerably improved with the laser beam equipment.

2.2 Injection approach

This approach has been used by Gale et al. (1987a,b and 1990). They used circular fracture specimens which were resin injected from a central hole, while the sample was subjected to normal load. The resin was allowed to cure before the sample was unloaded and cut along profiles in different directions. The thickness of resin and void along the sections was examined using a photo-microscope. The accuracy of the measurements was about 0.01 mm.

Hakami (1993) and (1994) used a similar method where a resin or a cement grout was injected in the fracture in-situ. Specimens were taken from core holes drilled parallel to a fracture plane, moulded in concrete and cut into slices. The resin thickness was measured from close-up photographs along the fracture profile on the slices. The procedure has now been developed further, using fluorescent epoxy resin and image analysis techniques which facilitates the measurements and data collection (see Figure 2).



Figure 2. Digital image from microscope showing a 4 mm long section of a fracture filled with fluorescent epoxy. The vertical lines across the void indicate the data collection points. The length of the lines is a measure of the aperture, b . The locations of the lines on each image are stored in a database. The accuracy of the measurements depends on the magnification used on the microscope and the contrast and sharpness of the image.

Pyrak-Nolte et al. (1987) also used injection of a fracture specimen under normal load in their aperture studies. They used Wood's metal, a metal with a melting point of about 170° F, and the temperature was held above the melting point during the injection. Instead of cutting the specimen, they split the fracture after the metal had solidified and studied the pattern of areas covered by metal and those free from metal with the aid of image analysis technique. Using this method the difference in aperture pattern at different load levels was analysed.

2.3 Casting approach

Gentier (1989) made a transparent coloured replica of the void space of a fracture as illustrated in Figure 3. An images of the replica, produced by taking a photograph, have different grey-scales according to the thickness of the coloured resin. The thickness can thus be calculated using image analysis techniques. Each pixel covered a square with a side length of 0.3 - 0.5 mm.

The method used by Hakami (1992) also belongs to this group although the details are different. Small volumes, V , of a silicon material (bouncing putty) were put between the fracture surfaces. The rubber deformed during normal loading of the sample and covered a smaller or larger area, A , depending on the local aperture. The fracture was opened after unloading and the size of the small patches covered by silicon were measured to give the average aperture at that location (see Figure 4).

Another way of using casting techniques is to make replicas of the fracture surfaces and match them to form physical models of fractures. This method has been used for mechanical studies of fractures by several researchers. To make the models suitable for

flow experiments, Hakami (1990) made the replicas of each side in a transparent material. When colour was injected to the water during the flow experiments the tortuous stream lines could be observed. The aperture was determined by placing water drops of known volumes between the fracture surfaces. The transparency of the replica makes the area stained by the drop visible and the average aperture over the small area can be determined in the same way as above (see Figure 4).

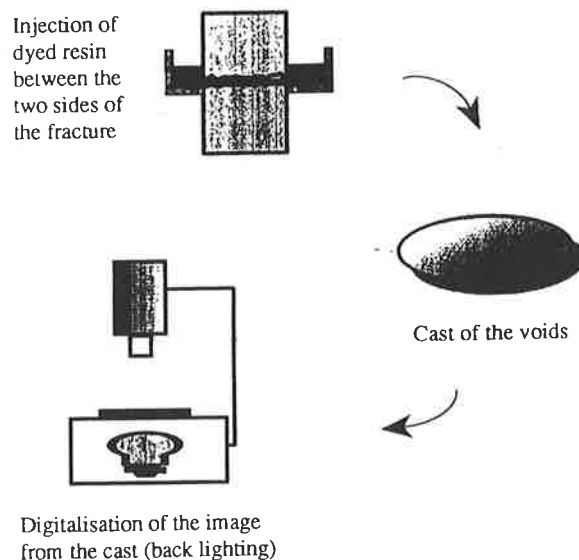


Figure 3. Principle of measurement technique using transparent void replica used by Gentier (1989).

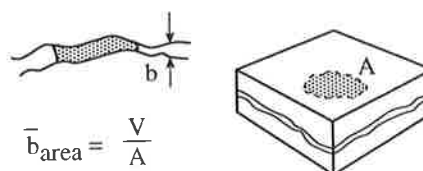


Figure 4. Principle for aperture determination using a small known volume of silicon placed between the fracture surfaces, Hakami(1992).

3 PARAMETERS

3.1 Fracture aperture

The term aperture is frequently used when describing fracture void space. This term can be defined as illustrated in Figure 5. The overall fracture surfaces are assumed to be parallel with a reference plane. The distance between the surfaces at a certain point, in the direction perpendicular to the reference plane, is the aperture, b , at that point. This aperture definition includes some considerations that should be noted:

- 1 The aperture, b , as it is defined here, is not the local perpendicular distance between the surfaces, which may be shorter at points where the surfaces are inclined relative to the reference plane. This should be recognised when aperture data are used for flow calculations.

- 2 The aperture, b , is defined at all points on a fracture surface, which means that the aperture can be zero and that the average aperture, $\bar{b}$, includes also the zero-aperture values.

- 3 The overall fracture characteristics can be quantified using averaging parameters of b such as the average aperture and the standard deviation of aperture.

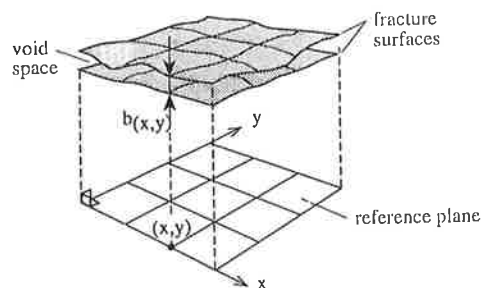


Figure 5. Suggested definition of fracture aperture, b .

4 The aperture varies with the applied load. A certain point on a fracture plane can thus have any aperture value depending on the stress level applied. This means that if the aperture distribution of two fractures are to be compared, a reference stress level must be defined.

3.2 Contact area

The term contact area has been used by several researchers in the literature. Since the true mechanical contact area is difficult to define, we suggest that the definition of contact area be the percentage of fracture area which have apertures smaller than a certain value. In detailed studies this threshold value may be kept small ($\approx 0.001\text{mm}$), but depending on the accuracy of the measurement method the threshold may be assigned a higher value ($\approx 0.1\text{mm}$).

3.3 Aperture frequency distribution

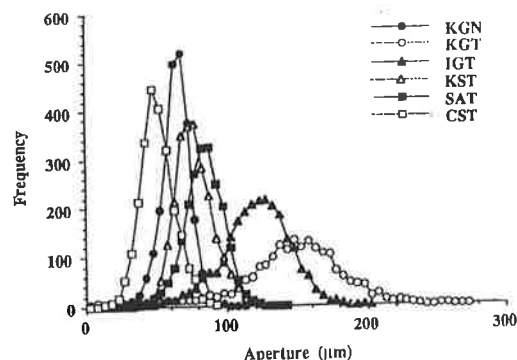
The aperture standard deviation or aperture variance are often used as a measure of aperture variation. Some researchers, e.g. Gale (1987a,b), use the transformed variable $\ln(b)$ or $\log(b)$ instead of b and in this case the standard deviation (variance) also refers to the this variable. Most aperture distributions reported to date are positively skewed, but there is no consistent indication that the aperture is log-normally distributed. It is therefore suggested that the standard deviation (s_b) or variance (s_b^2) of aperture be presented together with a frequency histogram of the data.

Figure 6 shows some results of aperture measurement results which have been presented as (probability) frequency histograms. The spread in the aperture distributions seems to increase with increasing average aperture. The explanation to this may lie in the difference in shear displacement that the fractures have experienced. However, the experimental data is still very limited and does not allow one to draw general conclusions.

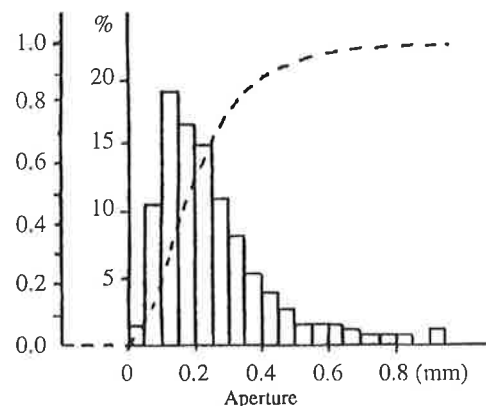
3.4 Correlation length

Two fractures with similar aperture frequency distribution may well have different hydromechanical characteristics due to differences in the spatial correlation of the aperture. The correlation length is a measure of how abruptly the aperture changes over the aperture surface area. Fractures with longer correlation length will have fewer, large, contact areas and a few channels conducting most of the ground water flow, as compared to the fracture with shorter correlation length. The correlation length can be expected to increase with increasing average aperture but there is not enough experimental data available to state the existence of such a relationship yet.

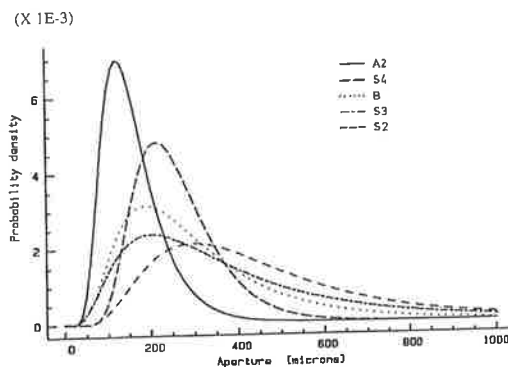
Spatial correlation of aperture data can be quantitatively analysed with geostatistical methods, which involves calculation of the correlogram or variogram (see for example Isaaks &



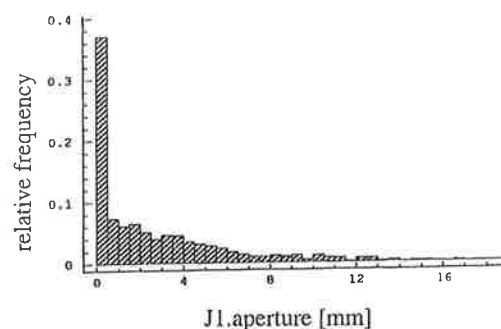
a)



b)



c)



d)

Figure 6. Examples of published aperture data from fractures of different nature, measured with different techniques. a) Natural and labmade tensile fractures, Iwano & Einstein (1993), b) Natural tensile fracture, Gentier (1990), c) Tensile (A,B) and shear (S2-4) fractures (best-fit log-normal functions), Hakami (1990); d) Minor fault, Hakami (1994).

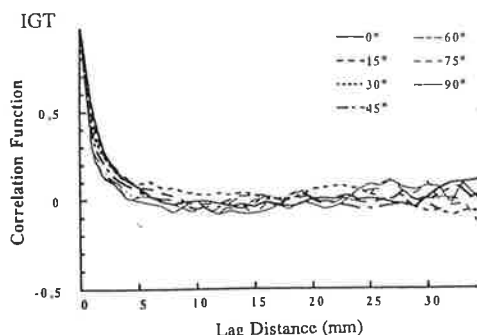


Figure 7. Example of correlogram of fracture aperture. Iwano & Einstein (1993).

Srivastava, 1989). The resulting diagrams present the difference in aperture value of data pairs as a function of the distance between the compared data points (lag distance). Different parameters can be used to describe the curve in a correlogram/variogram depending on the mathematical model which is chosen. To facilitate comparisons between experimental results, it is suggested that the parameter (*practical*) range be presented. The range is the lag distance at which the aperture variation of the compared points has reached 95 % of the overall variation (sill). In the case of anisotropic void geometry the range and/or the sill are different in different directions on the fracture surface. Figure 7 shows an example of correlograms calculated in different directions for one of the samples in Iwano & Einstein (1993) (range ≈ 4 mm). The correlation in this sample is isotropic.

4 DISCUSSION AND CONCLUSIONS

The methods for aperture measurements that were reviewed here do all have advantages and disadvantages. The suitability of each method depends on the type of fracture, the scale of the measurement and the aim of the investigation.

One important difference between the procedures (I) and (II) is that the aperture is not directly measured using (I), but it has to be derived from the surface topography. This means that the effect of normal stress cannot be studied with (I). Also, the contact area is difficult to assess since the surfaces are numerically matched. However, the surface topography methods seem to have a potential of becoming automated and not very expensive.

Procedure (II) has the advantage that the fracture is under compressive loading when the resin is injected and this is expected to give a realistic picture of the specimen aperture. This procedure seems to be the only suitable if the strength of the fracture walls or infilling materials are very low. A disadvantage could be that the specimen is destroyed during the measurement. The contact area can be determined well with (II) if the aperture threshold defining the contact area is not smaller than the resin (grout) penetration limit. Both procedure (I) and (II) can be used to analyse the spatial correlation of the aperture.

In some investigations it may be an advantage to use methods (I) and (II) as, apart from the aperture distribution, the surface roughness can also be characterised.

The methods in group (III) may offer ways of getting good comparisons between the average aperture characteristics of different fractures. The void casting method used by Gentier (1989) is also suitable for spatial correlation studies while the simpler methods reported by Hakami (1990 and 1992) can not be used for this purpose. An advantage with the casting methods is that the measurements do not destroy the fracture specimen. The fractures to be studied must, however, be competent enough to tolerate the casting procedure. The casting methods are generally more time consuming compared with other methods.

The aperture measurement methods described here are all intended for research purposes. In field mapping for site characterisation the measurements must be quick and simple, which can not be said for the procedures discussed here. One of the tasks of future research must be the development of quick fracture characterisation procedures.

The aim of current research on rock fractures is to find the span over which different fracture properties may vary and to understand the relationships between them. A step in this effort will be to agree on a common language with clearly defined terms that are accepted by most of researchers in the field.

ACKNOWLEDGEMENT

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A method to determine stress-transmissivity relationship of joints from hydraulic field testing

Méthode de détermination de la contrainte – Relation de perméabilité des fractures naturelles à partir de test hydrauliques sur le terrain

Eine Methode zur Bestimmung des Zusammenhangs zwischen Dehnung und Permeabilität von natürlichen Brüchen bei hydraulischen Feldversuchen

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ABSTRACT: A method to determine the large scale normal stress-transmissivity relationship of a rock fracture by hydraulic hole testing and numerical modelling is suggested. The testing consists of step pressure tests (hydraulic jacking) and fast pressure pulse tests. The method has been applied to six sub-horizontal joints intersected by a borehole at Luleå University of Technology, Sweden. The rock type at the site is granite and the joints are located at 80 to 417 metres depth. The study demonstrates that hydraulic aperture and joint normal stiffness can be determined accurately from the hydraulic borehole injection test.

RESUMÉ: On suggère ici une méthode pour déterminer la relation à grande échelle pression normale-transmissivité d'une fracture dans le roc à l'aide des essais de forages hydrauliques et l'utilisation d'un model numérique. La méthode a été appliquée à L'École Polytechnique de Luleå en Suède à six joints horizontales intersectés par un forage. Le rocher du site est du granite et les joints se trouvent de 80 à 417 mètres de profondeur. L'étude a démontré que l'ouverture hydraulique et la rigidité normale du joint peuvent être déterminées avec acuité à partir de tests d'injections hydrauliques pratiqués dans des forages.

AUSZUG: Ein Verfahren zur Bestimmung des örtlichen normalen Spannungs-Durchlässigkeits-Verhältnisses einer Gesteinsfraktur durch hydraulische Bohrlochprüfung und numerische Formgebung wird vorgeschlagen. Verfahren wurde an der Technischen Universität, Luleå, Schweden in sechs unter der Horizontalebene liegenden, von einem Bohrloch durchkreuzten Klüften angewandt. Der Gesteintyp vor Ort ist Granit und die Klüfte finden sich in 80 bis 417 Meter Tiefe. Die Studie zeigt, daß die hydraulische Öffnung und die Normalsteifheit der Klüfte durch die hydraulische Bohrloch-Einspritzprüfung genau bestimmt werden können.

INTRODUCTION

The effect of mechanical stress and deformation on the hydraulic permeability of rocks is an issue of concern in the performance assessment of nuclear waste repositories. In the Stripa Site Characterization and Validation Project, the inflow to a drift was reduced by one order of magnitude due to excavation effects (Gnirk, 1993). One possible reason is that induced stresses in the disturbed zone of the drift causes the joints to open or close which in turn strongly affects the hydraulic permeability (Fig 1).

Groundwater flow in fractured rock masses is modelled by discrete fracture models or equivalent continuum models. In these models, coupled hydromechanical effects are frequently considered by simple stress-transmissivity relationships for the fracture planes or for the equivalent continuum elements (Gnirk, 1993). Experimental and field studies have shown that the fluid

flow through natural joints is strongly affected by the stress normal to the fracture plane. Studies have also shown that the properties of the joint plane changes with sample size. This scale effect implies that data from a core sample test may not be representative for an entire fracture plane in the field. Therefore, a field test method is desirable to determine the stress transmissivity relationship of a fracture plane.

In this article single borehole hydraulic injection tests together with numerical modelling are used to determine an effective stress-transmissivity relationship for a fracture plane. An approximate analytical solution is also applied to analyse the field test results.

COUPLED HYDROMECHANICAL BEHAVIOUR OF A ROCK FRACTURE

The flow rate (Q) along a fracture (Fig. 1b) is affected by changes in the effective stress normal to the fracture plane (σ'_n). It is a coupled process where a change in effective stress causes mechanical deformation which in turn affects the fracture aperture and fracture transmissivity.

The mechanical normal displacement (Δv) of a fracture plane can be related to the effective normal stress (σ'_n) according to Goodman's classical model (Goodman, 1974):

$$\frac{\sigma'_n - \sigma'_{ni}}{\sigma'_{ni}} = \frac{\Delta v}{V_{mi} - \Delta v} \quad (1)$$

where V_{mi} is the maximum possible closure at the initial reference stress, σ'_{ni} . The initial normal stiffness can be calculated according to;

$$k_{ni} = \frac{\sigma'_{ni}}{V_{mi}} \quad (2)$$

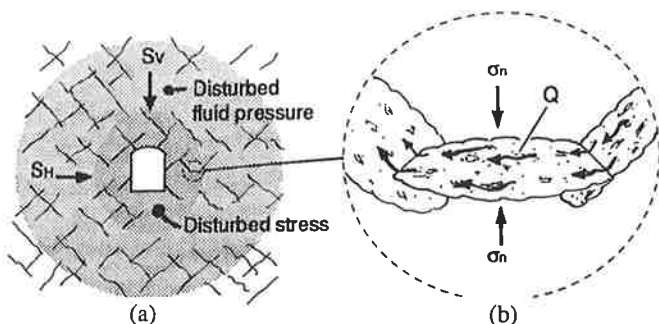


Figure. 1. Coupled hydromechanical processes in the rock mass around a tunnel; a) change of stress and fluid pressure due to excavation; b) coupling of stress and flow in single fracture.

Rearranging equation (1) and including the initial normal stiffness in equation (2) gives;

$$\Delta v = \frac{\sigma'_{ni}}{k_{ni}} \left(1 - \frac{\sigma'_{ni}}{\sigma'_n} \right) \quad (3)$$

The transmissivity of the fracture plane can be related to an equivalent hydraulic aperture (e) according to:

$$T = \frac{e^3 \rho_w g}{12 \mu_w} \quad (4)$$

where ρ_w and μ_w are the density and dynamic viscosity of the fluid. The hydraulic aperture at a given effective normal stress is (Zhao and Brown, 1992):

$$e = e_i + \Delta e = e_i + f \Delta v \quad (5)$$

where e_i is the initial hydraulic aperture at the initial reference stress, σ'_{ni} . Δe is the change in hydraulic aperture as a result of the mechanical normal displacement. The factor f reflects the deviation of a natural rough fracture from the ideal case of smooth fracture surfaces.

Combining equation (3) and (5) gives a relationship between the effective normal stress and the hydraulic aperture:

$$e = e_i + \frac{f \sigma'_{ni}}{k_{ni}} \left(1 - \frac{\sigma'_{ni}}{\sigma'_n} \right) = e_i + \frac{\sigma'_{ni}}{\kappa_{ni}} \left(1 - \frac{\sigma'_{ni}}{\sigma'_n} \right) \quad (6)$$

where κ_{ni} is named hydraulic stiffness and relates the change in effective stress to the change in hydraulic aperture. The hydraulic stiffness is equal or higher than the mechanical stiffness since the factor f is equal to or less than 1:

$$\kappa_{ni} = \frac{k_{ni}}{f} \quad (7)$$

Equation (4) and (6) will complete a relationship between the fracture transmissivity and effective normal stress:

$$T = T_i + \Delta T = \frac{\rho_w g}{12 \mu_w} \left[e_i^3 + \frac{\sigma'^3_{ni}}{\kappa_{ni}^3} \left(1 - \frac{\sigma'_{ni}}{\sigma'_n} \right)^3 \right] \quad (8)$$

Equation (8) shows that the initial aperture, initial normal stiffness and the initial effective normal stress are very important factors to define a relationship between stress and transmissivity of a fracture plane.

During a hydraulic injection test (Fig. 3a) the effective stress will be reduced by the increased fluid pressure inside the fracture. At the same time, the hydraulic aperture is measured from the injection flow. The coupled hydromechanical behaviour during an injection test is highly non-linear and requires numerical modelling to be analysed.

FIELD TESTS

Hydraulic tests were conducted on six sub-horizontal joints intersecting a 500 metres deep vertical borehole at the Rock Mechanics Laboratory of Luleå University of Technology, Sweden. The sub-horizontal joints are located at a depth of between 80 to 417 metres in granitic rocks. The downhole equipment consists of a straddle packer with 0.65 metres packer separation and downhole pressure transducer. A valve placed above the packer is used to conduct pressure pulse tests.

The joints were first pressurised and propagated by injection of 5 litres per minute. Thereafter the fractures were vented for 24 hours before the actual hydraulic injection testing was started.

A pulse test was conducted to determine the initial transmissivity and the initial hydraulic aperture (Rutqvist, 1995a) (Fig. 2a). Thereafter, a hydraulic jacking test was conducted in cycles of step-wise increase (forward step-pressure) and step-wise decreased (backward step-pressure) (Fig. 2b). First, the well pressure was increased step-wise until a maximum flow rate of 4-5 litres per minute was obtained. This cycle can be considered as mechanical unloading of the fracture plane because the effective stresses are reduced as the fluid pressure increases. Then, the well pressure was decreased step-wise until a back-flow from the fracture into the well was obtained.

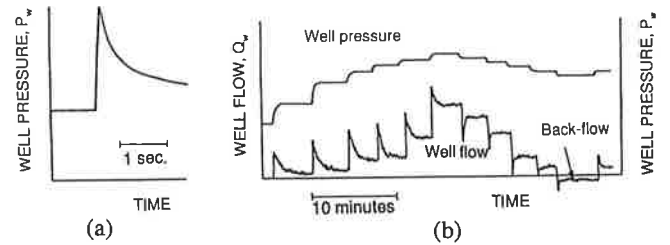


Figure 2. Typical results of single borehole hydraulic tests on a single fracture; a) pulse test. b) hydraulic jacking test.

NUMERICAL MODELLING OF FIELD TEST

The finite element model ROCMAS (Noorishad et al, 1992) is applied to simulate the single borehole injection tests of the fractures. The code simulates coupled hydromechanical processes of discrete fractures and a porous rock matrix.

The conceptual model of the field test (Fig 3a) is analysed with an equivalent axisymmetric model with an equivalent hydraulic boundary condition at an equivalent radius (Fig. 3b). The fracture is discretized into discontinuity elements with a non-linear relationship between effective normal stress and normal displacement (equation (3)). The current hydraulic aperture depends on the mechanical normal displacement according to equation (5) with factor f equal to 1. The fluid flow is governed by Darcy's law with the transmissivity calculated from the hydraulic aperture according to equation (4).

Figure 4 and 5 present the modelling and experimental results of the pulse test of the fracture at 356 metres depth. The experimental and numerical results of the pulse test are in close agreement to a pressure decay of about 50 % (Figure 4). Thereafter, the experimental result shows a slower pressure decay than the numerical modelling. This may be due to reduced aperture of the fracture away from the borehole in the experiment. In the numerical model, the aperture was assumed to be constant and equal to 16 μ m over the entire fracture plane. At 85 percent of pressure decay, the pulse affects the area near the wellbore to a radius of about 1 metres (Fig. 4). An aperture of 16 microns may therefore be representative for the fracture plane of 2 metres diameter near the wellbore.

The numerical model was calibrated to the experimental results of the hydraulic jacking test giving the best fit for the input parameters presented in the Table 1. At well pressures below the virgin normal stress the fracture is jacked up with maximum opening taking place near the wellbore (Fig. 5). Therefore, the pressure-flow response is very sensitive to the hydraulic aperture (e) and normal stiffness (k_n) of the fracture plane near the well bore. The fracture radius will have a minor effect on the test results (Rutqvist, 1995b).

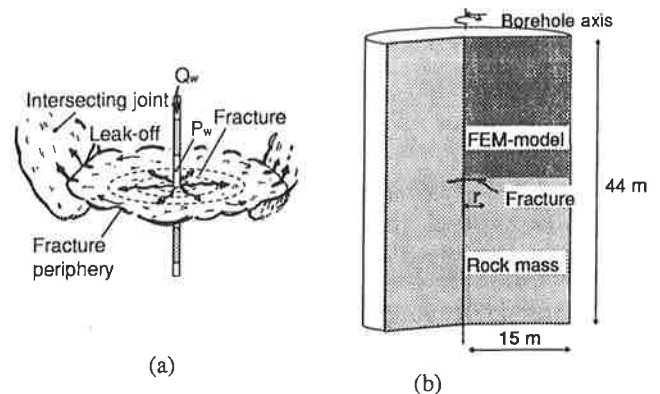


Figure 3. Models of the field test: a) conceptual model, b) finite element model.

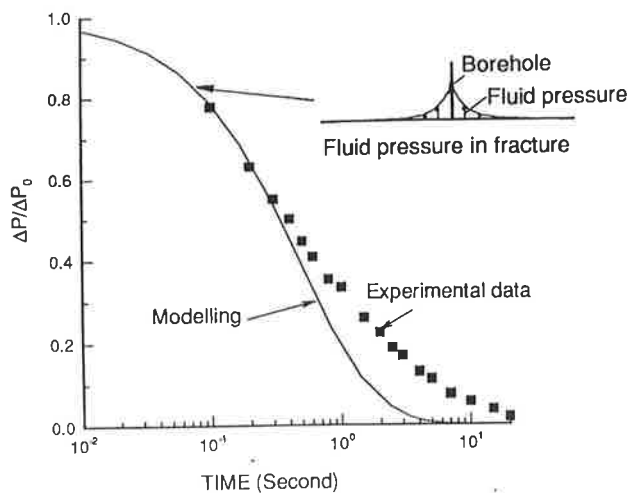


Figure 4. Modelling and experimental results of the pulse test at 356 metres. Fluid pressure in fracture is schematically presented.

When the well pressure is increased above the virgin normal stress the pressure at the fracture periphery is gradually increasing. At maximum flow rate of about 5 litres per minute, the fluid pressure at the fracture periphery is equal to the virgin normal stress. At this stage the fracture opening has occurred all the way from the well bore to the fracture periphery (Fig. 5). Therefore effective stress across the fracture and the fracture normal stiffness is relatively low and the pressure-flow response is dependent on the deformability of the ambient rock mass.

During the subsequent backward step-pressure cycle, the fracture may squeeze to a smaller volume if the leakage to the intersecting joints is larger than the injection flow rate. At zero-flow rate the pressure is uniformly distributed over the fracture plane and is equal to or less than the virgin normal stress (Rutqvist and Stephansson, 1995).

From the two types of test and the numerical modelling the three parameters needed to define the stress-permeability relationship in equation (8) can be determined. Table 2 presents the initial hydraulic normal stiffness, initial hydraulic aperture and initial effective normal stress obtained by modelling of the field experiments on the six horizontal fractures. The results are compared to the initial mechanical joint normal stiffness determined on the core samples containing the same fracture. The back-calculated hydraulic stiffness of the fracture plane in the field is up to one order of magnitude less than the mechanical

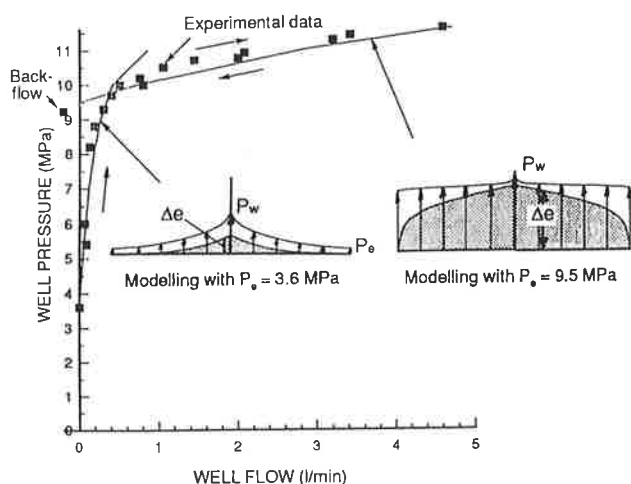


Figure 5. Modelling and experimental results of the hydraulic jacking test at 356 metres. Fluid pressure and change of hydraulic aperture in fracture are schematically presented.

Table 1. Material properties for modelling of the injection test at 356 metres.

Young's modulus of rock mass, E_{rm}	60 GPa
Initial normal stiffness, k_n	1000 GPa/m
Initial hydraulic aperture, e_i	16 μ m
Initial normal stress, σ_{ni}	9.5 MPa
Fracture radius, r	5 m

Table 2. Back-calculated parameters by numerical modelling and mechanical normal stiffness obtained from testing of core samples.

Test depth (m)	Initial hydraulic normal stiffness, K_{ni} (GPa/m)		Initial hydraulic aperture, e_i (μ m)	Virgin normal stress, σ_{ni} (Mpa)
Experiment and Model	Experiment	Model	Exp. and Model	Exp. and Model
81	300	30	8	2.2
131	1000	500	22	3.5
257	1000	125	18	6.0
283	2000	500	16	6.0
356	5000	1000	16	9.5
417	10000	1100	12	11.5

stiffness of the core samples. This is likely to be an effect of the different sizes of the fracture area in the field tests compared to the laboratory experiments.

AN APPROXIMATE METHOD TO DETERMINE STRESS-TRANSMISSIVITY RELATIONSHIP OF FRACTURES

During hydraulic injection the pressure and opening is uneven over the fracture plane (Fig. 5). Therefore, the one-dimensional model governed by equation (6) alone cannot fully simulate the three-dimensional hydraulic and mechanical response of a single well test. However, an approximate solution may be developed by considering that the mechanical response of the fracture plane is dependent on an "average" effective stress in the fracture around the borehole. Similarly, the well flow is dependent of an "average" hydraulic aperture in the fracture near the wellbore. The "average" hydraulic aperture is back-calculated from the well flow according to Darcy's law and equation (4):

$$e = \left(\frac{6Q_w \mu \ln(r_e/r_w)}{\pi(P_w - P_e)} \right)^{1/3} \quad (9)$$

The "average" hydraulic aperture is assumed to be related to the "average" normal stress according to equation (6). This equation can be written in the form:

$$e = e_i + V_{mi} + V_{mi} \sigma'_{ni} \frac{1}{\sigma'_n} \quad (10)$$

Now the hydraulic aperture is expressed as a linear function of $1/\sigma'_n$. The average effective stress in the fracture during the hydraulic injection test is approximated as:

$$\sigma'_n = \sigma'_{ni} + \Delta P_w \cdot C \quad (11)$$

The factor C is introduced assuming that the "average" change of fluid pressure in the fracture around the wellbore is proportional to the change of well pressure. The approximate solution was verified using the results of the numerical modelling at 356 metres depth. The calculation showed that the approximate solution can be applied with a factor of $C = 0.44$ (Fig. 6). The approximate solution follows the one-dimensional formula where the hydraulic aperture proportional to the inverse of effective stress until the well pressure increases above the virgin stress normal to the fracture plane.

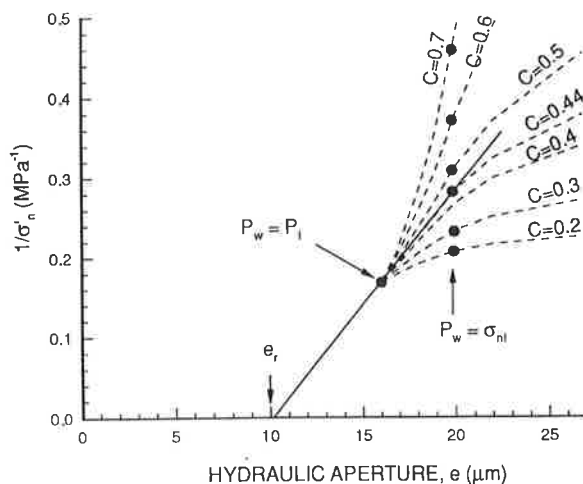


Figure 6. Inverse of effective stress as a function of hydraulic aperture. Solid line depicts input data to numerical model. Dashed lines are "average" values calculated from well flow and well pressure using equations (9), (10) and (11).

Figure 7 presents the stress-aperture relationship of the six fractures calculated from the field data using equations (9) and (10). Included is also the stress-aperture relationship based on the results of the numerical modelling in Table 2. Now a residual hydraulic aperture ($e_r = e_i + V_m$) can be determined where $1/\sigma'_n$ approaches zero. The slope of the line indicates that the fracture at 257 metres has relatively low stiffness and a negative residual aperture. This fracture was coated with calcite and weathered pyrite and may therefore be clogged at high stresses.

A variation of the C-factor of between 0.3 to 0.5 may change the stiffness value by a factor of two. However, the solution certainly gives a stiffness value within the same order of magnitude as the numerical modelling and serves as a good starting point for a fine tuned back-calculation with the numerical model.

CONCLUSIONS

Initial hydraulic aperture, initial effective stress and initial hydraulic normal stiffness are the most important factors governing the stress-transmissivity characteristic of a joint plane. These parameters can be determined by single borehole hydraulic injection tests of the joint. The hydraulic normal stiffness should be back-calculated with numerical modelling but an approximate one-dimensional expression (eq. 9, 10, 11) can be used.

The combination of testing and modelling was applied to six horizontal joints intersecting the same borehole in granitic rocks. The results indicate that the normal stiffness of the fracture is dependent on the size of the fracture surface area. However, it is likely that the method of field testing gives a stress-transmissivity relationship which is representative for the entire fracture plane.

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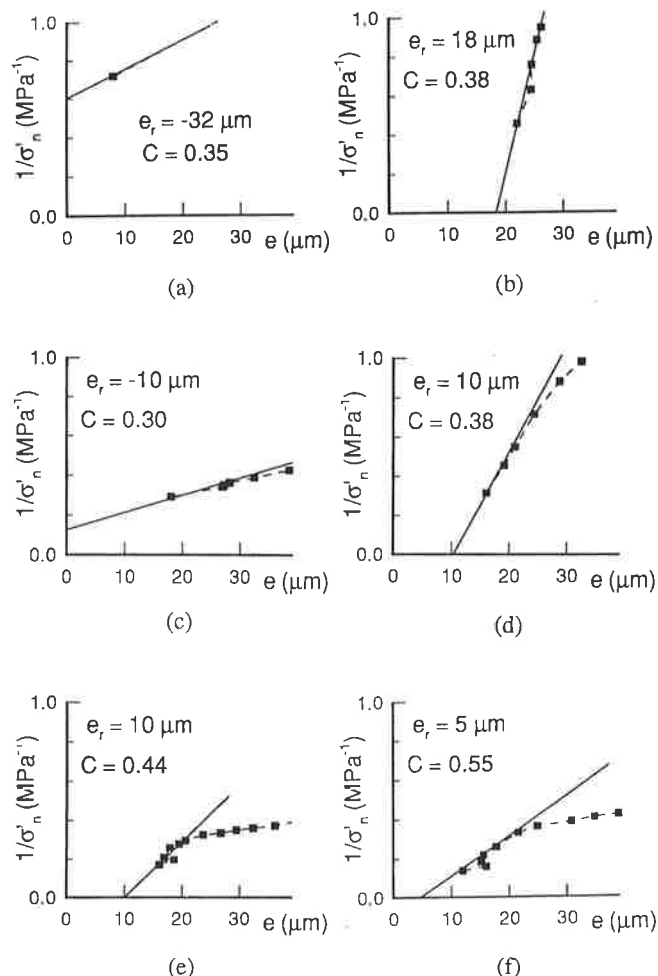


Figure 7. Stress-aperture relationship of the subhorizontal joints located at a depth of; a) 81 m, b) 131 m, c) 257 m, d) 283 m, e) 356 m and f) 417 m. Dashed lines are values calculated from the field data using equations (9), (10) and (11). Solid lines are Stress-aperture relationship based on data in Table 2.

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Water curtains in gas storage – An experimental study

Stockage de gaz dans des cavernes rocheuses sans revêtement – Effets du rideau d'eau à partir d'une étude pilote

Gasspeicherung in unverkleideten Kavernen – Die Einwirkungen von Wasservorhängen; Ergebnisse einer Versuchsstudie

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ABSTRACT: Water curtains comprise an interesting method to enhance the storage capacity of compressed, unlined gas storage and have been used for more than 20 years for this purpose. At the Röda Sten Rock Laboratory (RSRL), operated by Chalmers University of Technology, tests were performed in a pilot-scale rock cavern, explicitly constructed for gas storage experiments. The tests showed that no gas leakage occurred, provided the water curtain pressure, at all points, was higher than the gas pressure. At gas pressures above 1.0 MPa, complete gas containment was not achieved due to hydromechanical breakdown of the rock mass caused by the water curtain pressure.

RÉSUMÉ: Les rideaux d'eau représentent une méthode intéressante pour augmenter la capacité d'emmagasinage des dépôts sans doublure de gaz sous pression. Au laboratoire des roches de Röda Sten, en activité à l'Ecole Polytechnique de Chalmers, des essais dans une caverne pilote à l'échelle, exclusivement construite pour l'emmagasinage des gaz, ont été effectués. Ces essais montrent qu'aucune fuite de gaz n'a eu lieu aussi longtemps que la pression du rideau d'eau était en tous points supérieure à celle du gaz. Pour des pressions de gaz supérieures à 1.0 MPa, une retenue complète du gaz ne fut pas réalisable à cause de la rupture hydromécanique de la masse rocheuse, causée par la pression du rideau d'eau.

ZUSAMMENFASSUNG: Wasservorhänge umfassen eine interessante Methode zur Erhöhung der Lagerungskapazität beim Lagern von Gas unter Druck, in Räumen ohne Verkleidung. Sie haben seit über 20 Jahren für diesen Zweck Verwendung gefunden. Im Felslabor Röda Sten Rock Laboratory (RSRL), betrieben durch die Technische Universität Chalmers, wurde eine Modellversuchsanlage einer Felskaverne geprüft, die ausdrücklich für Versuche mit Lagern von Gas konstruiert worden war. Die Prüfungen zeigten keine Gaslecks an, unter Voraussetzung daß der Druck des Wasservorhangs, an allen Punkten, höher war als der des Gases. Bei Gasdrücken über 1,0 MPa wurde kein vollständiger Gaseinschluß erreicht wegen hydromechanischer Zerteilung der Felsmasse verursacht durch den Druck des Wasservorhangs.

1. INTRODUCTION

Storage of gas underground dates back to the beginning of the century when depleted gas and oil fields in the USA were used for this purpose. Even today, aquifer storage is widely used where an appropriate geology can be found. Another method where the geological conditions are used as a natural seal to prevent the gas from escaping is storage in leached salt formations.

However, nature has only formed such geological settings in certain areas. Storage in hard rock geologies must be based on man-made caverns to create the necessary volume. Storage of pressurized gas in such caverns can be divided in two categories; lined and unlined caverns, where the former has a gas-tight membrane of steel or plastic attached to the cavern walls.

For any unlined storage, gas containment depends on the presence and pressure of water in the rock mass around the storage. The water pressure situation, and thus the gas containment capacity, is combined effect of the natural ground water pressure and the influence from the storage. Control of the water pressure in the rock mass prevents leakage and increases the gas containment capacity; such control is possible by the use of a water curtain.

The following paper summarizes the most important results of a series of tests performed in a pilot-scale cavern at the Röda Sten Rock Laboratory (RSRL), operated by Chalmers University of Technology, Sweden. The material was presented as a doctoral thesis by the senior author (Söder, 1994).

The gas leakage phenomenon can be divided into two processes, each of which is controlled by different factors; gas entry (from the storage into the rock joints) and gas migration (further movement of gas in the joint system). These two processes were investigated thoroughly by Åberg (1977) and later by Goodall (1986) through theoretical analyses and laboratory tests.

The process of gas entry, i.e. at which conditions a meniscus of gas is spontaneously developed at the mouth of a fracture, is controlled by the capillary pressure in the interface between water and gas. Laboratory tests by Åberg (1977) and Goodall (1986) showed that the necessary capillary pressure to prevent gas entry varies significantly under different conditions and is controlled by parameters which are more or less unknown in a rock mass; natural fracture aperture, fracture connectivity and disturbances from the excavation. Consequently, Goodall, Åberg and Brekke (1988) concluded that the most practical design approach for gas storage was to prevent gas migration, not gas entry.

Gas migration is referred to as further movement of gas already existing in the joint system. A theoretical explanation of this process was first given by Åberg (1977) who found that if the hydraulic gradient in the vertical direction is larger than unity no gas can migrate from a storage. Later work has confirmed this criterion (Goodall, 1986; Åberg, 1989) which can be generalized for the entire cavern periphery: the water pressure along all possible leakage paths must increase for some small distance in the direction of potential gas escape (Goodall, Åberg and Brekke, 1988).

Various authors have reported from field tests on different scales where this criterion was adopted to establish gas-tight conditions; Gustafson, Rhén and Bodén (1989) on borehole tests, Lindblom (1988) on air containment of a Swedish LPG storage prior to commissioning and Goodall, Kjörholt, Tekle and Broch (1988) on the experience from nine air cushion surge chambers in Norway. These facilities are important since they represent full-scale, air containment test plants in various geological conditions, being operated at different air and groundwater pressures.

Three air cushion surge chambers, Kvilldal, Tafjord and Torpa, use a water curtain to control the air leakage. The performance of these air cushion surge chambers at different water curtain pressures was reported by Kjörholt, Goodall, Johansson and Stokkebo (1989) and by Kjörholt and Broch (1992). The investigations clearly showed that a water curtain can significantly improve the air containment capacity and probably established air tight conditions at these sites; the small air losses that occurred were explained to be caused by dissolution of air in the water bed.

2. AIMS OF TEST PROGRAM AT RSRL

As explained above, international experience in general and the Norwegian in particular, have shown that water curtains are effective in reducing or halting gas leakage. However, large-scale gas containment tests under well defined and controlled conditions was still lacking at the start of the test programme at RSRL. These tests had the following aims:

- The hydraulic characteristics of the rock mass should be determined
- Air leakage rates should be investigated at different air pressures in a cavern without a water curtain
- With a water curtain installed, the degree of air containment at different air pressures and water curtain pressures should be determined.
- The effects of high water curtain pressure on air containment, causing shearing and jacking of rock joints should be investigated.
- The effects of different borehole spacing in the water curtain should be investigated.
- Monitoring of microseismic events, should be performed during the tests.

Key results of the tests are presented.

3. SITE CONDITIONS

RSRL is located in Göteborg on the Swedish west coast, at a depth of 70 m and approximately 30 m below the ground water level (cavern roof). The site contains two caverns, cavern A and cavern B, figure 1. Cavern A was designed for high internal pressures while cavern B was designed for low temperature tests. This paper describes tests which were performed in cavern A in the period 1990-1994.

Cavern A was excavated through conventional drill-and-blast technique without any reinforcement measures and has the

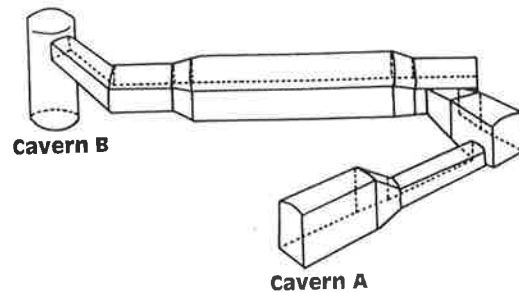


Figure 1: View of RSRL with cavern A and cavern B.

approximate dimensions as indicated in figure 2. At the entrance a concrete plug is located, which was designed for an internal pressure of 8 MPa to enable future tests with a liner attached to the cavern walls.

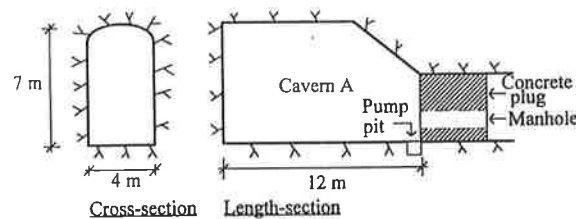


Figure 2: Principal layout of cavern A and the concrete plug.

A series of tests with compressed air, where the ground water pressure was not controlled, were performed to assess the natural air containment capacity of the rock mass, to be used as reference for the next stage of the project.

The water pressure around a storage can only be controlled if a water curtain is constructed and operated correctly. To achieve this, a water curtain was drilled, with the drillholes located at an average distance of 5 m from the cavern walls as shown in figure 3.

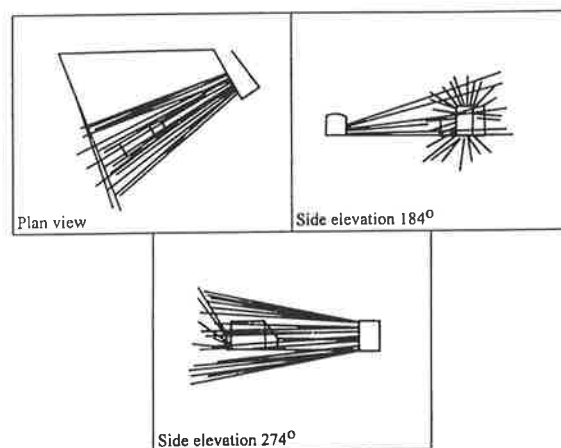


Figure 3: Water curtain at the cavern A, RSRL, based on deviation measurements.

In total 33 boreholes were drilled to create the water curtain. Out of these, 28 holes were drilled with a top-hammer percussion rig (diam. 64 mm.). 5 boreholes were core drilled (diam. 46 mm.) to gain geological information of the rock mass around cavern A. The length of the boreholes was approximately 40 m.

Packers were installed in the water curtain boreholes and were connected to a water supply system and a pump from which the pressure in the system was controlled. The system was very flexible and allowed several combinations of boreholes and pressures to be tested.

The rock mass at RSRL is a massive, non-weathered, medium-grained granite of pre-cambrian age. Narrow veins of mylonite, amphibolite and quartz appear in the rock mass.

During drilling of the core holes, measures were taken to enable orientation of the joints. The joints in all cores from the parts closest to cavern A are shown in figure 4.

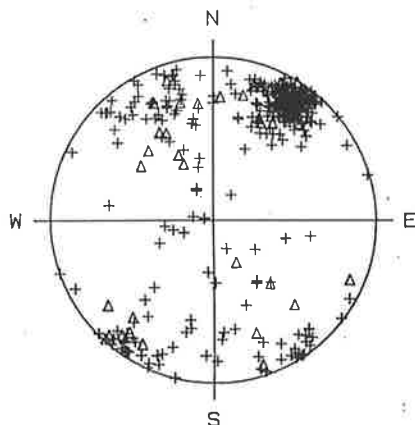


Figure 4: Observed joints in the cores close to cavern A. Plus signs (+) represent smooth joint surfaces and triangles represent rough joint surfaces.

As a complement to core logging, cell mapping was performed inside cavern A to obtain an estimation of the trace lengths for different joint sets. Based on joint frequency and trace lengths, the following joint sets dominate the rock mass around cavern A.

Joint set:	A	B	C	D	E
Attitude:	N60W/85W	0/0	N20/30W	N-S/60E	N65E/70E

A comparison with the observed joints in figure 4 indicated that core logs alone were inadequate to produce a correct model of the joint system. The core logs only produced clear evidence of joint sets A and E. In addition to joint frequency, the extension of joint planes is an important factor for the geohydrological behaviour of a rock mass (Long and Witherspoon, 1985; Andersson and Thunvik, 1986 and Andersson and Dverstorp, 1987). For the cell mapping at RSRL it was simply assumed that the larger the trace length, the larger the joint radius.

When possible, core logs should always, be compared with cell mapping of the same rock mass. The size of the cells must be chosen according to the trace length of individual joints; if a majority of the traces have one, or both, ends outside of the mapped area, the estimation of trace lengths will be difficult.

4. AIR CONTAINMENT RESULTS

4.1 Leakage criterion

The single most important aim with the tests in cavern A at RSRL was to verify the criterion by Åberg (1977) in a large-scale field test: if a vertical hydraulic gradient of at least unity exist around a storage, no gas can migrate away from the storage. This criterion can be generalised to: the water pressure in the rock mass close to the storage must be higher than the gas pressure along all possible leakage paths (Goodall et al, 1988).

A first series of tests was conducted to investigate the air containment capacity when only the natural ground water pressure existed in the rock mass. Thus, no water curtain existed at this stage.

Cavern A was filled with compressed air to a pre-defined pressure, the inlet valve was closed and the pressure and temperature continuously monitored. From the general state equation for an ideal gas, the total mass of air, m_{air} , in cavern A was calculated as:

$$m_{air} = \frac{p_a V}{R_{air} T_k} \quad [kg]$$

where: p_a = absolute gas pressure [Pa]
 V = volume [m³]
 R_{air} = individual gas constant for dry air,
 287 [J/(kgK)]
 T_k = absolute temperature [K]

The volume V was calculated as the total volume of cavern A, 330 m³, minus the volume of water in the water bed.

For practical purposes, the leakage was expressed as the normalised air volume per time unit. The unit used for normalised air volume is Sm³ (standard cubic meter), i.e. the air volume determined at atn. pressure and 20°C.

The maximum air pressure, p_g , in cavern A during these tests was approximately 500 kPa; p_g denotes pressure in excess of atmospheric. The resulting air leakage versus the air pressure, in cavern A can be seen in figure 5. Two significant trends, approximated by two lines, can be identified; below an air pressure of 114 kPa no air leakage was detected and above this value a small increase in the leakage occurred. At air pressures above 330 kPa, the increase in air leakage was considerable. Included in Figure 5 is also the global groundwater pressure in the area around cavern A, 350 kPa. A reduced piezometric, near-field pressure level of 170 kPa was monitored in an inclined borehole between cavern A and the near-by service tunnel.

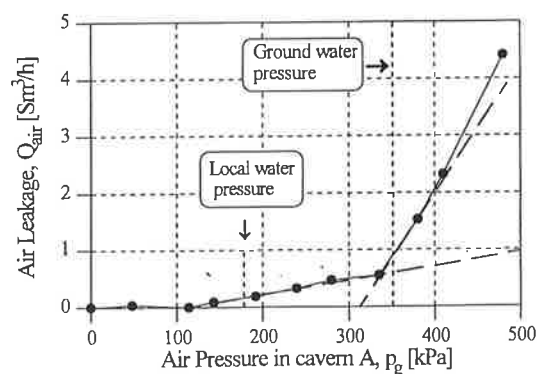


Figure 5: Air leakage rates without a water curtain

Figure 5 shows that the break-through pressure for air in cavern A was slightly lower than 114 kPa. Should no service tunnel have existed close to cavern A, the results indicate that breakthrough would have occurred at an air pressure of approximately 330 kPa, slightly smaller than the monitored ground water pressure of 350 MPa.

A theoretical calculation of the breakthrough pressure was performed, based on a method presented by Liang and Lindblom (1994). This method assumes a homogenous, isotropic rock mass; the influence of the service tunnel on ground water pressure can not be included in the analysis. Based on this method, the critical gas pressure p_{gc} becomes approximately 110 kPa. Compared to the true breakthrough pressure of 330 kPa, it can be concluded that this method, at RSRL, underestimates the breakthrough pressure.

The most likely explanation for this discrepancy is that pressure drawdown in a homogenous continuum model due to an excavation is overestimated; experience both from RSRL and the Stripa project (Olsson 1992) seem to indicate a zone of decreased permeability around the excavated cavern. Joint closure, due to the excavation and prevailing rock stresses, together with the existence of bonded water due to capillary forces in small joint openings, is a plausible explanation of this zone. On the other hand, blasting damage around an excavation tends to work in the opposite direction. Factors which changes the permeability locally around excavations are at present the subject of research within the Swedish nuclear waste disposal programme.

At the next stage of the project, a water curtain was used to control the water pressure outside of cavern A. The air pressure, p_g , during these tests ranged from 420 kPa to 1020 kPa. The procedure of a test included a stepwise decrease of the pressure difference between the water curtain and cavern A until the water supply pump was completely stopped. The pump was then restarted and the pressure difference increased in reverse order. Each pressure difference was maintained for at least 3 hours and a stable air leakage rate was determined.

The results of these tests are summarized in figure 6 as air leakage, Q_{air} , versus potential difference between the water curtain and cavern roof Δh_{wc} . A value of Δh_{wc} larger than 5.5 m corresponds to a vertical hydraulic gradient above unity which, according to Åberg (1977), should prevent gas leakage. Figure 6 clearly shows that Åberg's criterion was verified in these tests.

According to figure 6, the value of Δh_{wc} at breakthrough was zero at the smallest air pressure, and then gradually approached

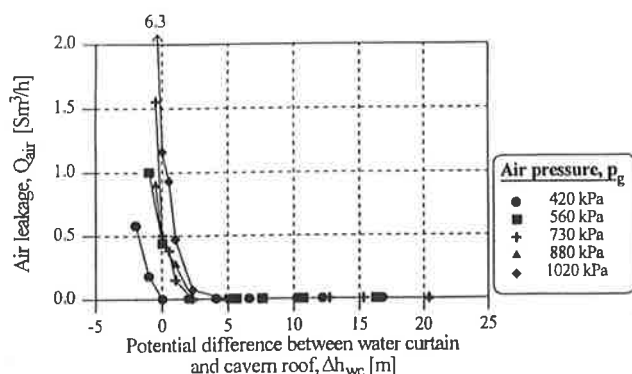


Figure 6: Air leakage rates with a water curtain. One value at an air pressure of 1020 kPa, $Q_{air} = 6,3 \text{ sm}^3/\text{h}$, is not included.

a value close to 5 m, as the air pressure increased. This implies that, at the lower air pressures, the air flow was directed more or less horizontally from the cavern roof. As the air pressure increased, a gradually more vertical flow is indicated. This behaviour could be an effect of capillarity in the vertical joint planes which could have prevented air entry into these joints at the lower air pressures.

This conclusion is supported by the results of well tests in the water curtain. A summary of these tests is shown in figure 7, where the hydraulic conductivity of each individual borehole is represented by a filled circle. Hydraulic communication paths are schematically presented as follows: a solid line corresponds to a clear communication; a broken line corresponds to an unclear or limited communication; no line corresponds to no communication.

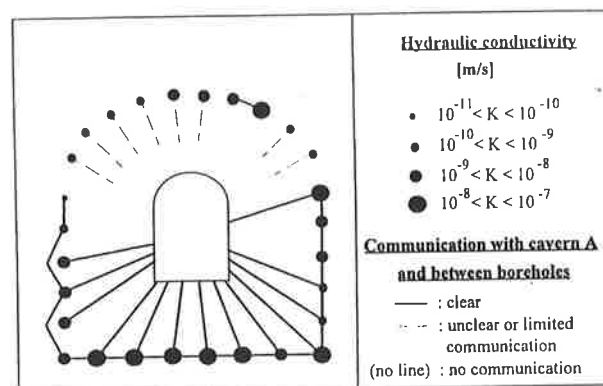


Figure 7: Summary of the geohydrological conditions around cavern A.

The results in figure 7 clearly show the comparatively low hydraulic conductivity and limited hydraulic communication in the area above cavern A. However, it is difficult to assess the effect of capillarity from field tests. Instead, any contribution of capillarity should be included as an increased factor of safety.

4.2 Air containment at high air pressure

An operator of a storage will be interested in the maximum amount of gas that can safely be stored. For this type of storage the limit is set by the maximum water pressure that can be used. The additional costs of a water curtain pressure increase (higher water consumption, treatment of collected water in the storage and higher design pressure for the equipment) are probably small compared with the advantages gained from a larger gas storage capacity.

An increased water pressure in the rock mass will eventually result in shear failures of existing rock joints. At even higher pressures, jacking of joints will occur (Pine and Batchelor, 1984). Due to dilation, shear failures result in an increased hydraulic conductivity around the water curtain. Jacking represents a process which inevitably puts a limitation to the water pressure. Normal stresses across joints are balanced by the water pressure then; joint openings in water flow will increase dramatically.

Shear failures were registred at RSRL by detection of microseismic events by a system of accelerometers, located around cavern A. The source location of these events was determined based on the difference in arrival times.

The source location analyses of microseismic events was performed by Camborne School of Mines Associates, U.K. and the result is shown in figure 8.

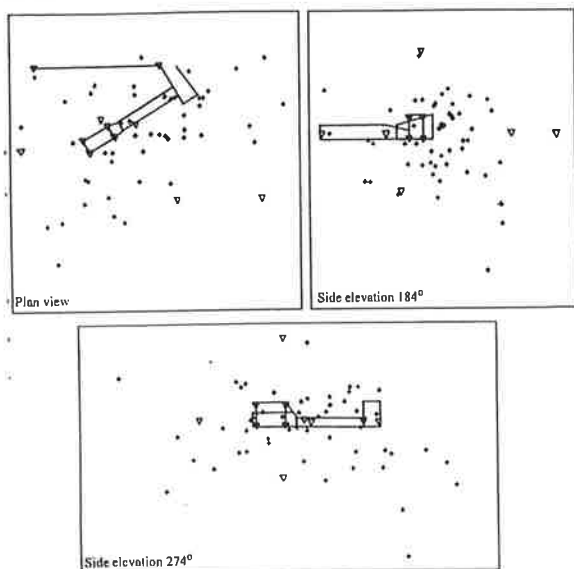


Figure 8: Location of all events which could be located (+), Sensor locations are marked with triangles (∇). (From CSMA, 1994).

It appears from figure 8 that the majority of the shear failures occurred in the rock mass outside the eastern wall of cavern A. Outside the west wall, the rock mass experienced jacking in earlier experiments (He, 1992) which probably released a large portion of the shear failures at this occasion.

The microseismic events started to occur at a water curtain pressure of approximately 650 kPa whereas the maximum pressure during the experiments was about 1650 kPa. An attempt was made to predict the pressure at which shearing of rock joints would start, based on the method suggested by Pine and Batchelor (1984). According to this method, the limiting water pressure is calculated from the prevailing in-situ stresses and strength properties of rock joints.

From the parameters determined at RSRL, the resulting critical water pressure was calculated; the result is presented in figure 9 where the critical pressure (MPa) is given for a joint with a defined dip and strike in relation to the major principal rock stress σ_1 . Included are also the three dominating joint sets together with a Schmidt-net to show their orientation. Joint set B, which is almost horizontal, was assumed to have a dip variation of the order of 10° , corresponding to the results of almost vertical core logging. The strike of this joint set was not well defined due to its orientation close to horizontal.

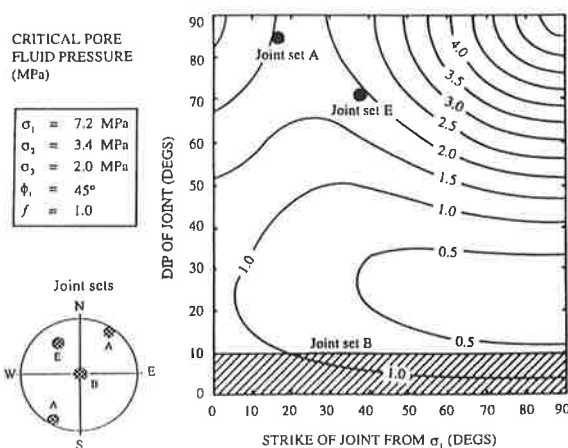


Figure 9: Critical water pressure of the rock mass at RSRL.

Figure 9 shows that joint set B is the most critically oriented; shearing was predicted to start at an injection pressure of 500-1000 kPa (0.5- 1.0 MPa). With a maximum dip of 10° , the critical pressure becomes approximately 700 kPa. Joint set A and E would require an injection pressure of almost 2000 kPa (2.0 MPa) to initiate shearing.

Thus, the method by Pine and Batchelor (1984) resulted in a prognosis of the critical injection pressure (appr. 700 kPa) in very good agreement with the true value (appr. 650 kPa).

Shear failures did not seem to influence the air containment capacity of cavern A; complete air tight conditions was possible up to an air pressure of 1020 kPa, well above the pressure where shear failures started. When the air pressure was increased further, however, it was impossible to achieve air tight conditions, even at very large water curtain pressures. The most likely explanation of this behaviour is the opening of horizontal joints. Such opening was successfully registered by instruments inside cavern A, starting at a pressure of approximately 1000 kPa and increasing rapidly towards the jacking pressure of 1700 - 1800 kPa.

Thus, complete air containment seems linked to opening of joints rather than to shearing of joints.

5. CONCLUSIONS

5.1 Air containment

Without a water curtain around cavern A the breakthrough pressure, controlled by the global groundwater pressure, was determined to approximately 330 kPa, slightly smaller than the prevailing ground water pressure of 350 kPa. A homogeneous, 2-D, continuum model underestimated the breakthrough pressure, drastically.

Local changes in the hydraulic conductivity due to the excavation seems to be a plausible explanation for the breakthrough pressure being higher than predicted by the models. This is, however, a field where the knowledge is not satisfactory. Further research would improve prognoses of, for example, water inflows to tunnels and other excavations.

With a water curtain in operation it was possible to maintain complete air containment at a maximum pressure of 1020 kPa (in excess of atmospheric pressure), which is about three times larger than without a water curtain. It was shown that no air leakage occurred through the rock mass in situations when the water pressure increased along all migration paths away from the cavern; the results implied that capillarity enhanced air containment at the lower air pressures and that this effect was reduced when the air pressure increased. However, it was found unlikely that such effects could be predicted from field investigations. Instead, gas containment assessments are recommended to be based on the general criterion by Goodall et al (1988); if capillarity effects exist, they can simply be included as an increased factor of safety.

The requirement to maintain the rock mass water saturated at all times was found to be inadequate; the experience of all tests at RSRL revealed that extensive air leakages into the rock mass had no influence on the terminal air containment capacity of the cavern. The same held true even in the tests without a water curtain; when the air pressure decreased below the ground water pressure, after a breakthrough, the containment capacity was soon re-established.

5.2 Hydromechanical breakdown

The microseismic system at RSRL successfully registered several events identified as shear failures of existing rock joints. The injection pressure required to initiate shearing was calculated to approximately 700 kPa, with expected variations of the orientation of rock joints accounted for. Actual shearing was detected when the injection pressure reached 650 kPa. A significant result is that shear failures alone did not seem to influence the air containment capacity of the rock mass.

When the water curtain pressure was increased above approximately 1000 kPa, deformation gauges registered the opening of horizontal joints in cavern A. At slightly larger pressure, air-tight conditions could not be maintained any more. It should be recognized that the actual jacking pressure at RSRL was of the order of 1800 kPa, calculated by the method presented by Broch (1982) and verified by He (1992).

Thus, the opening of joints seems to start at lower hydraulic pressure than predicted by theoretical methods. Since air containment capacity appears to be jeopardised by opening of joints, a more refined analysis of non-elastic deformations due to the pressure both in the water curtain and in the cavern is necessary. Today, the knowledge in this field is limited.

Before any analyses, as described above, are performed it is suggested that the conservative criterion of shear failure is used to define an upper limit of the water curtain pressure. As shown by Gustafson, Lindblom and Söder (1991), this would allow for a maximum over-pressure ratio of 2.0 times the prevailing ground water pressure. Compared with a gas storage without a water curtain this could still allow for a considerable increase of storage capacity.

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Grouting in theory and practice – A case study from Äspö HRL, Sweden

Théorie et pratique du forage des puits par cimentation – Une étude de cas d'Äspö HRL, Suède
Grouting in Theorie und Praxis – Eine Fallstudie vom Äspö-HRL in Schweden

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ABSTRACT

The tunnelling work for the tunnel at the Äspö Hard Rock Laboratory, Oskarshamn, Sweden, is completed. The tunnel has passed rock of different quality and several zones of difficult water conditions. Grouting technology has been developed in order to deal with the difficult conditions, high permeable zones with high water pressure and limited penetration of the grout into the rock. Grout properties have been tested for different types of grout. The grout properties vary depending on concentration and type of admixture. The theory of grouting has also been developed in order to better predict grout take and penetration. The theory indicates that there are three main groups of factors : grouting technology, hydrogeology and geology. Comparison of theoretical grout take and penetration with measured values has been carried out with promising results.

RESUME

Le percement en lui même du tunnel du Äspö Hard Rock Laboratory à Oskarshamn en Suède est maintenant terminé. Le tunnel traverse des roches de différentes qualités et plusieurs zones de problèmes hydrauliques délicats. Des techniques d'injection ont été développées pour faire face à des difficultés comme des zones très perméables à haute pression hydraulique et à faible capacité d'injection dans la roche. Les propriétés de différents types de coulis ont été testées. Ces propriétés dépendent de la concentration et du type d'adjuvant utilisé. Les recherches ont aussi été orientées en vue d'obtenir une meilleure prédiction théorique de la pénétration et de la prise du coulis. Ces recherches font apparaître trois facteurs déterminants: la composition du coulis, l'hydro-géologie et la géologie du milieu. Les conclusions obtenues après la comparaison des résultats théoriques de la pénétration et de la prise du coulis avec des valeurs mesurées sur le terrain sont prometteuses.

ZUSAMMENFASSUNG

Die Tunnelarbeiten für das Äspö Hard Rock Laboratory in Schweden sind abgeschlossen. Der Tunnel läuft durch durch Fels mit unterschiedlicher Qualität und mehrere Zonen mit schwierigen Grundwasserverhältnissen. Injizierungstechnik wurde entwickelt, um die schwierigen Felsbedingungen mit durchlässigen Zonen unter hohem Wasserdruck und geringer Eindringtiefe des Injiziermaterials zu beherrschen. Die Mörtleigenschaften änderten sich mit deren Konzentration und Zusammensetzung. Ein theoretisches Modell wurde entwickelt, um besser die Eindringtiefe und den Mörtelverbrauch vorhersehen zu können. Die Theorie zeigt, dass drei Faktoren hauptsächlich von Bedeutung sind: das Injizierverfahren, die Hydorlogie und die Geologie. Ein Vergleich des theoretisch ermittelten Mörtelvolumens und der Eindringtiefe mit gemessenen Werten ist vielversprechend.

1 INTRODUCTION

The purpose of grouting is to create a watertight zone around a tunnel. This watertight zone should fulfil a tightness requirement which corresponds to a certain penetration length and a filling of the aperture of the fractures. Research into the fundamental mechanics of grouting has been going on at the Dept of Soil and Rock Mechanics, Royal Institute of Technology Stockholm, since 1983, sponsored by the Swedish Rock Engineering Research (SveBeFo). This has led to two doctoral theses on theory of grout flow (Hässler, 1991) and grout properties (Håkansson, 1993). Some theoretical developments have shown that the penetration length is dependent on the pressure, grout mix and the fracture aperture while the grout volume is dependent on the penetration length and the rock porosity for used mix.

Particularly difficult and comprehensive grouting work have been carried out for the tunnelling project at the Äspö Hard Rock

Laboratory, Oskarshamn Sweden, (Stille et al, 1993, 1994). The grout mix has been developed and investigated in order to find suitable combinations which deal with difficult conditions, highly conductive zones with high water pressure. A particular requirement of this project is that a limited grout penetration is desired in order to avoid excessive ground water contamination around the tunnel. Three analytical models have been developed to predict the grout take. A theory for predicting grout take and penetration is presented and applied to the experience gained in the tunnel.

2 PREDICTION OF THE GROUTING

2.1 General

The analytical models are based on the following assumptions : the flow of grout has to come to a standstill, the flow is slow

(laminar), and both the grouting pressure, ΔP , (pressure between grouting over pressure and pore pressure) and the yield value, τ_0 , of grout are known and have a constant value during the grouting.

2.2 Model I

Assumption

The fracture can be described as a circular disc with a constant aperture b (figure 1). The fracture disc intersects the drill hole and the grout mix will fill the disc symmetrically. The flow of grout has come to a standstill when the penetration reaches the maximum length I_1 . The number of injected discs along the drill hole, N , is supposed to be the same as the number of conductive fractures.

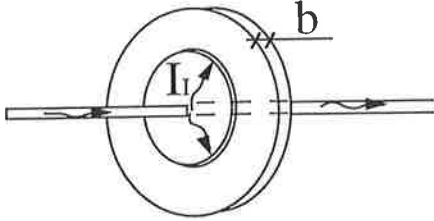


Figure 1 In model I the penetration takes place in a circular disc

Connections

The penetration length for the grout is expressed as :

$$I_1 = \frac{\Delta P \cdot b}{2 \cdot \tau_0}$$

The grout volume for all the discs along the borehole can be calculated according to :

$$V = I_1^2 \cdot \pi \cdot b \cdot N$$

From the water loss measurements the transmissivity, T , can be expressed as :

$$T = \frac{Nb^3 \cdot \rho_w \cdot g}{12 \cdot \mu_w}$$

Based on the relationship between the penetration length and the number of discs along the drill hole and transmissivity, the predicted grout volume can be estimated as :

$$V_{\text{hole,cal}} = \left(\frac{\Delta P}{2 \cdot \tau_0} \right)^2 \cdot \frac{12 \cdot T \cdot \mu_w}{\rho_w \cdot g} \cdot \pi$$

From the estimated number of conducted fractures the fracture aperture and thus the penetration length for the grouting can also be determined.

2.3 Model II

Assumption

This model is an analytic simplification of Hässler's (1991) geometrical model. The principle is that the grout flows out in a system of channels in the rock. The area of each of the injected channels is added together and the total area is described as a section of the disc. The disc section is defined by a spreading angle α , which is dependent on the geology, and is assumed to have a constant aperture b_{II} (figure 2). It is further assumed that water flows in the same channels as the grout.

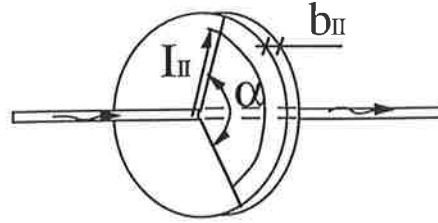


Figure 2 In model II the penetration takes place in a part of a disc

Connections

The penetration length for the grout in the section can be expressed as :

$$I_{II} = \frac{\Delta P \cdot b_{II}}{2 \cdot \tau_0}$$

The grout volume for all the discs along the borehole can be calculated according to :

$$V = I_{II}^2 \cdot b_{II} \cdot \frac{\alpha}{2} \cdot N$$

The water loss measurement in a drill hole can be expressed as (Hässler 1991) :

$$Q_w = \frac{\Delta P_w \cdot Nb_{II}^3}{12 \cdot \mu_w} \cdot \frac{\alpha}{\ln(1 + \alpha \cdot L/W)}$$

where ΔP_w is the water over pressure calculated from a certain distance L from the drill hole. W is the length of fracture aperture crossing the drill hole wall, normally assumed to be the circumference of the drill hole.

From the water loss measurement and the estimation of the spreading angle a prediction of the grout volume can be carried out as :

$$V_{\text{hole,cal}} = \left(\frac{\Delta P}{2 \cdot \tau_0} \right)^2 \cdot \frac{12 \cdot Q_w \cdot \mu_w}{2 \Delta P_w} \cdot \ln \left(1 + \frac{\alpha \cdot L}{W} \right)$$

From the estimated number of conducted fractures the fracture aperture and thus the penetration length for the grouting can also be determined.

2.4 Model III

Assumption

In model III the rock is described in more detailed than the other models. Grouting in model III is based on the assumption that the grout flows out into a system of discs (figure 3). The geometry of discs and penetration of grout are described by a number of geological parameters.

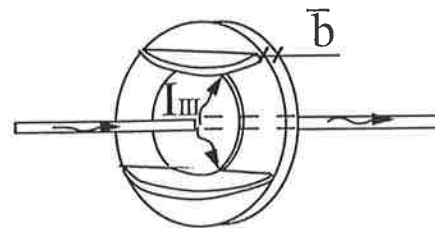


Figure 3 In model III the penetration takes place in a system of discs

The discs have an average aperture $\bar{b}$, where $\bar{b}$ can be described as the product of a parameter θ and the hydraulic aperture b from the water loss measurement :

$$\bar{b} = \theta \cdot b$$

The parameter β is the ratio between total disc area and the area of the primary disc.

N_g is the number of discs which are grouted and intersect the drill hole and can be determined as the product of a reduction parameter γ and the number of water conductive discs :

$$N_g = \gamma \cdot N$$

The penetration length is calculated from b_g and λ , where λ is a parameter for the curve way in the disc, accordingly the quotient between the curve way and the radial way for the penetration.

The parameter κ is described as the quotient between the channel aperture b_g and the hydraulic aperture from water loss measurement :

$$\kappa = b_g / b$$

Connections

The penetration length for the grout in the disc system can be expressed as :

$$I_{III} = \frac{\Delta P \cdot b_g}{2 \cdot \tau_0 \cdot \lambda}$$

The grout volume for the disc system along the borehole can be determined as :

$$V_{\text{hole, cal}} = I_{III}^2 \cdot \bar{b} \cdot \pi \cdot \beta \cdot N_g$$

From the water loss measurements the transmissivity, T , can be expressed as :

$$T = \frac{Nb^3 \cdot \rho_w \cdot g}{12 \cdot \mu_w}$$

From the transmissivity and determination of the five different geology parameters (γ , κ , λ , θ , β) the predicted grout volume can be estimated as :

$$V_{\text{hole, cal}} = \left(\frac{\Delta P}{2 \cdot \tau_0} \right)^2 \cdot \frac{12 \cdot T \cdot \mu_w}{\rho_w \cdot g} \cdot \frac{\gamma \kappa^2 \cdot \theta \cdot \beta \cdot \pi}{\lambda^2}$$

From the estimated number of conducted fractures and the parameter κ , the fracture aperture and thus the penetration length for the grouting can also be determined.

3 DOCUMENTATION OF THE GROUTING

3.1 Geology and hydrogeology

The bedrock at the Äspö Hard Rock Laboratory is dominated by a granite which in mineralogical composition varies from real granite to a more diorite variety. The rock quality has in general been good or fair according to the RMR classification system. The tunnel direction is N-S.

The main hydraulic structures are single fractures, faults and fracture zones. The single water-bearing fractures form a steeply dipping NW-SE striking set. The faults dip gently with the same NW-SE strike (Munier and Hermanson, 1994). Two of the major fracture zones have resulted in very comprehensive grouting activities. The first zone was a 45 m wide zone composed of a number of subzones and the second zone was highly water-

bearing and the most intense part was highly fractured and partly clay-altered. The rock quality was poor in both zones.

Transmissivity has been recorded in the probe holes drilled every 20 m. In general, values in the range of $0.5-1.0 \cdot 10^{-7} \text{ m}^2/\text{s}$ have been recorded, but in the two major zones, values have been recorded in the range of $2\,000-15\,000 \cdot 10^{-7} \text{ m}^2/\text{s}$.

3.2 Grout properties

Several different mix designs have been investigated in order to find suitable combinations. The investigation focused on the rheological properties, i.e. yield stress and plastic viscosity, of the different fresh cement grouts. Rheology plays an important role in the grouting; for example; the yield stress will limit the penetration length. In general, suspensions possess a very complicated rheological behaviour. This complexity is due to various physical and chemical interactions from both the fluid phase and the solid phase. Such complex behaviour introduces problems in the measurement and evaluation of the rheological properties (Håkansson, 1993).

The rheological properties and the effect of different manipulations were evaluated by using a Brookfield Rheoset rotational viscometer. The measurement could continuously be carried out in the same fluid sample, enabling the change with time to be observed.

Firstly, a grout called Stabilo grout was used; this grout has been used extensively in grouting works in Sweden. Later a highly accelerated grout with calcium chloride was developed to better withstand the higher water pressure encountered in the tunnel.

Stabilo grout

The mix design with grouting cement, bentonite, plasticizer and silicate provide the desired rheological properties. In order to improve the penetration in finer fractures, the grouting cement was changed to a finer cement, Mikrodur P (Micro). The mixes are shown below (table 1), where Std was the standard mix design, Styv was a stiffer mix and Acc has an accelerating effect on the setting time.

It was confirmed that the temperature has an effect on the properties and the tests were carried out at low temperature, about the same as the rock $8-10^\circ\text{C}$. Low temperature will decrease both the yield stress and the plastic viscosity and slow down the reaction rate.

Table 1 The tested mixes for Stabilo grout

Name	Std	Styv	Acc	Micro
Cement	Degerham	Degerham	Degerham	Microdur
W/C	1.3	1.0	1.0	2.0
Bentonite [%]	2.0	3.0	2.0	2.0
Plasticizer [%]	1.0	2.0	1.0	1.0
Silicate [%] (solution)	3.0 (40)	4.0 (40)	3.0 (40)	2.0 (40)

Stabilo grout results

The result are shown in figure 4. The following observations have been made : a decrease in the W/C ratio increased both the yield stress and the viscosity; the bentonite influenced the grout by increasing both the yield stress and the viscosity due to the ability of

the bentonite to absorb water; the plasticizer separated the individual cement grains and made them less prone to form agglomerates. In ordinary cement grouts the plasticizer will clearly give lower yield stress and lower viscosity. When silicate was used, it increased the yield stress. Grout with both plasticizer and silicate increased the yield stress (due to the silicate).

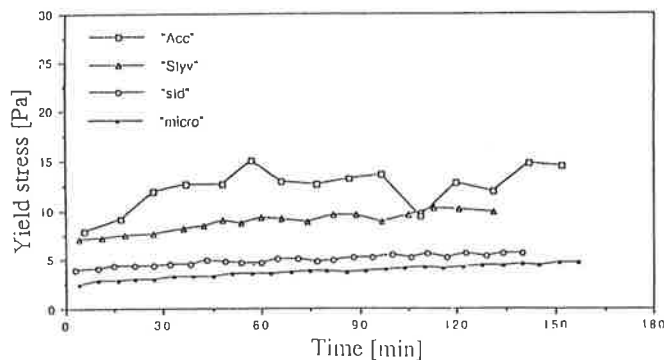


Figure 4 Investigation on Stabilo grout, for the yield value

Calcium chloride grout

In order to meet the conditions in the tunnel, new grouts had to be designed and tested. The new mix used calcium chloride to achieve a more accelerated grout with the following compositions : Degerhamn grouting cement W/C 1.0 or 0.8 and 15.0 % chloride.

Calcium chloride results

The results are shown in figure 5. From the study it is clear that by adding calcium chloride, the setting time can be altered. The grout was very sensitive to disturbance during the first part of the hardening process. As long as the mixing was going on the hardening process was delayed and the grout had rheological properties similar to ordinary cement grout but with a higher yield stress.

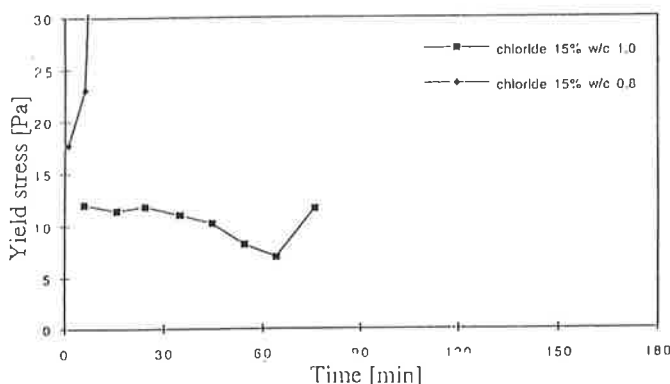


Figure 5 Investigation on calcium chloride grout, for the yield

3.3 Grouting performed

A total of 136 groutings have been carried out along the tunnel where 52 of these groutings, or nearly 40 %, are related to two fracture zones. The total grouted volume is 384 000 l, excluding bore hole fillings, where the two zones have consumed 120 000 l, or about 30 %, of the grouted volume (table 2). A total of 21 950 m grout holes have been drilled for the grouting.

Extraordinary grouting was performed for the two fracture zones. Table 2 demonstrates that the fracture zone II has a high average grout volume. The high volumes of grout in the zones are a result of the high degree of regroutings performed at the zones

Table 2 Grout volume and hole length

	Fracture zone I	Fracture zone II	Remaining parts of the tunnel
Grout volume [l]	50 000	70 000	264 000
Drill hole length [m]	3050	7060	11 840
Grout length of the tunnel [m]	31	30	1160
Average grout volume per meter tunnel [l/m]	1610	2330	230
Drill length per meter of tunnel [m/m]	98	235	10

The grouting was mainly concentrated in four different sections along the tunnel (figure 6). The grout volume in each section varied from 10 % to 25 % of the total volume for the whole tunnel. The first two sections are the two major fracture zones, section 970 m to 1000 m and 1270 m to 1360 m respectively. The third section is section 1360 m to 1390 m which is in typical tunnel geology but near the fracture zone II and it has a high transmissivity. The fourth section is section 2010 m to 2160 m which is a long section with several minor fracture zones and fracture swarms (Munier and Hermanson, 1994).

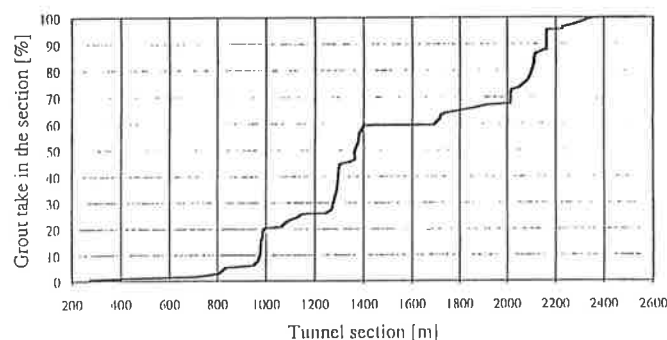


Figure 6 Distribution of grout take along the tunnel

4 Application of the models

4.1 General

The water loss measurements were carried out in all the boreholes before the grouting started. The grouting of the first hole will influence the transmissivity for the rest of the hole in the fan. Because of this influence only the first grouted borehole in each fan could be used in the analysis. Of the total 136 fans, 44 have been used for the analysis.

Pressure and grout mix have often been varied during the grouting of a hole. The pressure and yield value used in the calculation have been determined as the mean value for the hole. The number of fractures has been estimated based on the geological mapping (Stanfors et al, 1993, 1994).

In order to calculate the grout takes, the described models need some geological data. For model II the value of the spreading angle has been supposed to vary with the rock quality, which gives a value between 0.2 - 2.4 rad. The lowest value is valid for the poorer rock because in poorer quality only a few fracture leads water and the rest fractures are filled with gouge. For model III the values given in table 3 has been used.

Table 3 Estimation of geology parameters in model III

Consideration	Parameter	Value
Number of fractures	γ	0.05 - 0.40 : Depending on rock quality and type of fractures
Channel aperture	κ	4.0
Average aperture	θ	1.0
Grouted path	λ	1.5
Secondary fracture plane	β	Number of fracture sets divided by two

It was observed that poor rock had many fractures which were more or less conductive but just a few of the fractures could be grouted, this yielded a smaller γ value. The good rock have few fractures but many of them were highly conductive and could take grout, hence the γ value is higher.

4.2 Grout take

The calculated volumes have been compared to measured values for the three models. The ratio between the calculated and measured grout volume has been determined. The models are complete if the ratio is 1.0. Table 4 shows the average and median ratio for all fans. The models can be said to be reliable if the ratio is in the range of 0.5 to 2.0, table 4 shows the percentage of ratio in this range. A regression analysis has been carried out with the measured and calculated volumes as variables. The correlation coefficient r is shown in table 4. The comparison between calculated and measured grout take can also be seen in figures 7-9, where the lines represent the limits for reliability are shown.

Table 4 Volume comparison between the models

Model	Average ratio	Median ratio	Reliability	The r value (the ideal value is 1.0)
I	1.76	0.71	36 %	0.62
II	2.07	0.58	30 %	0.56
III	1.80	0.90	59 %	0.71

The average is higher than the median, because some holes have a much higher calculated value than measured. Accordingly the median is more useful to compare. Model III gives a better result than model I and II for all comparisons.

4.3 Grout penetration

The penetration length can be calculated on the basis of the measured grout take. The calculation for model I is simple, since only the number of fractures has to be estimated. For model II the results from water loss measurement have been used so that the actual value of the spreading angle can be calculated. The situation is similar for model III. The result from the water loss measurement has been used and the ratio between the channel aperture and for the hydraulic aperture has been calculated. The remaining parameters have been estimated according to table 3.

For the grout penetration the acceptable value is between 5 to 15 m which was the actual span measured in the three water-bearing zones at the tunnel. The results of the calculated penetration are shown in table 5.

Table 5 Penetration length comparison between the models

Model	Mean length [m]	Median length [m]	Acceptable
I	5.8	4.9	45 %
II	13.0	2.8	16 %
III	14.9	11.8	59 %

4.4 Comparison between the models

The ability of the models to give an acceptable prediction is expressed as the ratio between the number the acceptable outcomes and the total number. The results are given in table 6.

Table 6 Comparison between the models

Model	Grout volume [%]	Penetration length [%]
I	36	45
II	30	16
III	59	59

Model I

This model is the simplest one and gives a moderate result for both grout volume and penetration length.

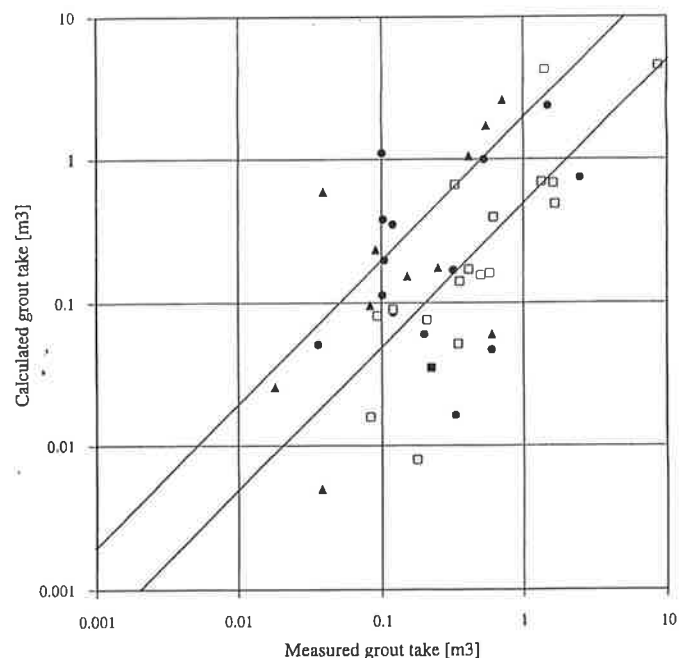


Figure 7 Comparison between calculated and measured grout take for model I

Model II

This model is sensitive to the input data and therefore sometimes gives the wrong results. Minor errors in measured grout take and water loss can give large errors in the spreading angle and thus in the penetration length.

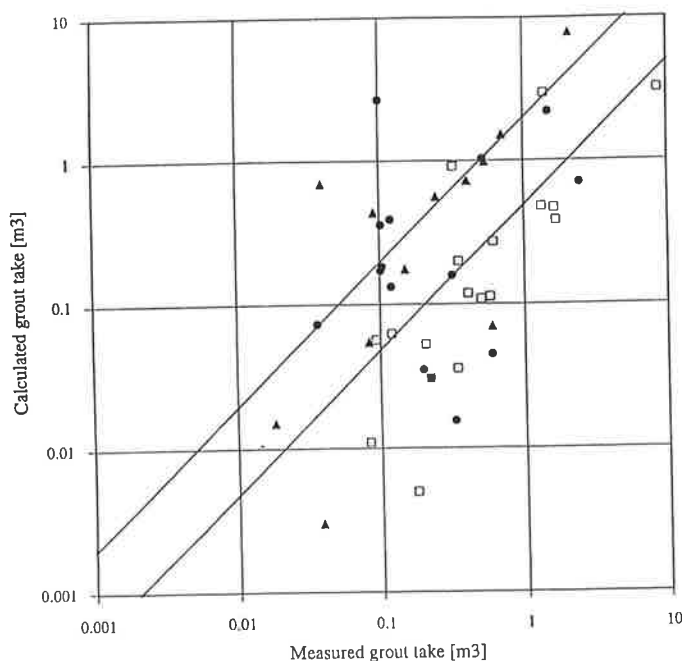


Figure 8 Comparison between calculated and measured grout take for model II

Model III

This model gives the best results but is the most complex one and needs a good knowledge of geology to be used. One important factor is that the model distinguishes between grout and water take in the fractures.

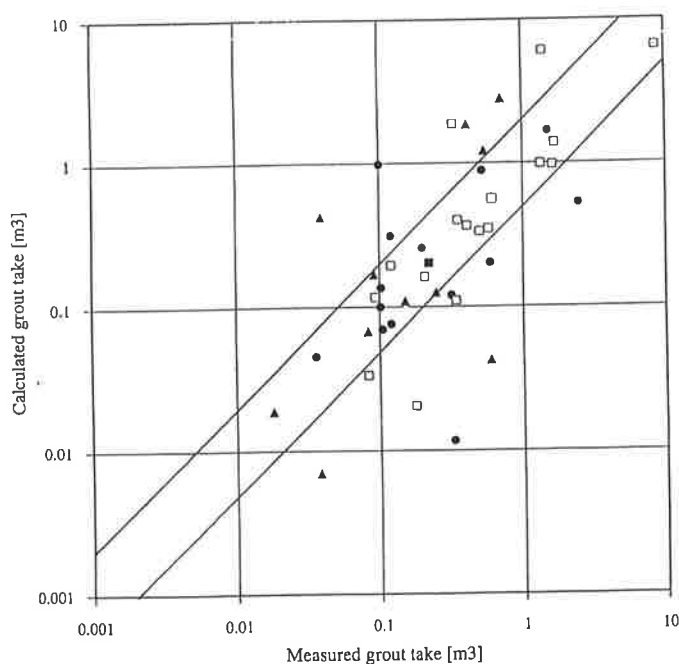


Figure 9 Comparison between calculated and measured grout take for model III

5 CONCLUSION

The theoretical models point to three main categories of factors : grouting technology (grouting pressure and grout properties), hydrogeology (transmissivity and fractures porosity) and geology (number of fractures, fracture aperture and geometry).

For all the models, the yield value of the grout is a very important parameter and a small error in the measurement and estimations of this value can result in a large error in the calculation of grout take.

The grout take is directly dependent on the transmissivity value. The estimation of the transmissivity is dependent on the evaluation method and what is measured - the first borehole or the whole fan.

The knowledge of how the geology influences the spreading of grout in rock is poor. In model III different geological parameters have been used with good experience but more investigations will be needed to make a classification system for grouting.

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Shotcrete as a tunnel lining – A laboratory study

Le béton projeté comme revêtement de tunnel – Une étude de laboratoire Spritzbeton als Tunnelauskleidung – Eine Laboruntersuchung

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ABSTRACT: This paper presents a laboratory study concerning shotcrete as a tunnel lining applied on irregular tunnel surfaces. The problem concerned is first presented. A description of the model test is given and major findings from the study are presented. The main conclusion gained from the study is that the irregularities of tunnel surfaces are unlikely to have negative effects on the general safety of a tunnel.

RÉSUMÉ: Ce document présente une étude laboratorique en ce qui concerne béton de spray comme un revêtement du tunnel, appliqué sur des surfaces irrégulières du tunnel. D'abord on introduit le problème. Ensuite une description de l'essai du modèle est présentée. Des résultats majeurs de l'étude sont exposés et la conclusion principale est la suivante: Des irrégularités des surfaces d'un tunnel n'ont pas (ou peu probable) d'effets sur la sécurité générale du tunnel.

ZUSAMMENFASSUNG: Der vorliegende Bericht beschreibt eine Laborstudie bezüglich der Anwendung von Spritzbeton zur Tunnelwandverstärkung an unebenen Oberflächen. Zuerst wird das Problem dargestellt. Danach werden die Modellversuche beschrieben und die wichtigsten Ergebnisse beschrieben. Der Hauptsatz dieser Studie ist, daß es unwahrscheinlich ist, daß Unebenheiten an der Tunneloberfläche einen negativen Einfluß auf die allgemeine Sicherheit des Tunnels haben.

1 INTRODUCTION

When a tunnel is excavated by the drill and blast method, tunnel surfaces are usually uneven and wavy, sometimes there is substantial overbreaking due to the nature of the rock mass or unfavourable blasting performance. The unevenness or irregularities of the tunnel surfaces make tunnel sections deviate, sometimes significantly, from designed sections which are often of perfectly circular, elliptical, horse-shoe shaped or rectangular in shape. When shotcrete is used as tunnel lining, the shotcrete will follow the irregularities of the constructed tunnel. Because tunnel support analysis and design are performed based on those perfect design sections, the following questions are often raised in tunnelling practice:

- i. how much does a shotcrete lining applied in a tunnel behave differently from the design?
- ii. is it necessary to fill the overbreakings with shotcrete?
- iii. is it necessary to have a smooth finished tunnel surface to meet the condition of design?

Answering those questions has significant importance to an economical and fast tunnelling performance, since filling overbreakings with shotcrete is costly and time-consuming. By careful blasting the roughness of the tunnel surface can be reduced. But it can never be guaranteed that overbreaking can be avoided. Moreover, smooth blasting in tunnels can be time-consuming and expensive.

In the past, some work concerning this problem in hard rock have been done (e.g. Holmgren, 1975; Fernandez, et al, 1976), whereas little has been found in the literature with reference to weak rock. This paper, with focus on weak rock, presents a study on the interaction between a shotcrete lining and uneven tunnel surfaces by laboratory model testing. The model testing is aimed at gaining overall information on the behaviour of shotcrete linings applied on wavy tunnel surfaces. Numerical calculations are also carried out in order to study the rock behaviour around an uneven tunnel by means of ground reaction curve concept. The study was done at the Division of Soil and Rock Mechanics, Royal Institute of Technology, Sweden.

2 SCALE FACTOR

In order to relate the results of a model test to an actual tunnelling practice, similitude laws must be derived. This is done by dimensional analysis based on Buckingham's π -theorem (e.g. see Langhaar, 1951). For the purposes of the model test of this study, the tunnel lining problem is treated as a static and non-body force problem. Therefore velocity, acceleration and density are not taken into account. For such a problem, the following conclusion have been drawn by the dimensional analysis (Chang, 1994),

For a static, non-body-force tunnel lining problem, a complete similarity between a model and prototype can be achieved, if the material properties and load conditions of the model are identical to those of the prototype, and provided that the geometrical scale is satisfied.

In reality, it is impossible to find a model material identical to the rock mass in which a tunnel is driven. However, the above statement tells us that if the model materials are chosen so that properties of the materials are fairly close to that of a practical case, and the geometrical scale is satisfied, the results obtained from the model tests can be well related to the tunnel by similitude laws.

3 DESCRIPTION OF THE MODEL TESTS

The dimension of the model is 1.6m x 1.6m x 0.5m and the radius of the "tunnel" is chosen as 0.4 m. The rock material is made of cement, bentonite, sand and water. The material properties were intensively tested by Holmberg (1992) with uniaxial and triaxial tests. The mechanical properties of the model material are given in Table 1, which shows that the model material is equivalent to rock class IV (poor rock) in accordance with the Geomechanics Classification for Rock Mass Rating (RMR). Therefore, the test results are applicable to poor rock conditions according to the statement given above.

Table 1. Properties of the rock material.

Modulus of elasticity	80 - 100 MPa
Poisson's ratio	0.2
Uniaxial strength	0.7 - 0.9 MPa
Friction angle	15° - 20°
Cohesion	0.2 - 0.3 MPa
Dilation angle	20°

The lining material consists basically of standard cement, sand and water (w/c ratio is 0.4), while steel fibres of DRAMIX 30/50 are incorporated 3% by weight for some models. The thickness of the lining is 1 cm except one model.

The model is loaded by six jacks from two sides (three on each), providing hydrostatic load-conditions. Using the axisymmetrical principle, the "tunnel" is located at the corner in order to have an easy access for placing lining and convergence meters inside the tunnel. Instrumentation consists of devices for measuring the radial deformations of the tunnel and the oil pressure in the loading system.

Totally, seven models as listed in Table 2 are tested. Three models with a circular tunnel, two of which are lined, are tested and they provide a base for comparison. Model No.2 had a too thick lining and its results are extracted from the comparison. Two types of irregular surfaces shown in Figure 1 were tested. Model 7 is a supplemental tests due to that model 6 had improper contact at the lining ends to the loading frame so that the lining cracked at the contacts at relatively low load.

Table 2. Seven model tests performed in the laboratory investigation.

Number	Type of Test	Comments
1	Circular, unlined	
2	Circular, lined (2cm)	Lining was too thick
3	Circular, lined (1cm)	
4	Simply waved, lined (1cm)	
5	Simply waved, lined SFR (1cm)	
6	Doubly waved, lined SFR (1cm)	Lining cracked at low load
7	Doubly waved, lined SFR (1cm)	

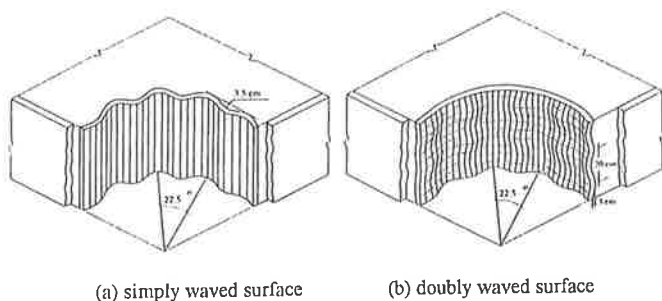


Figure 1. Two types of waved surfaces tested in the model testing.

4 MAJOR FINDINGS AND DISCUSSION

Major findings from the model tests and a short discussion are given here. For details, please refer to Chang (1994).

The effects of the irregularities are investigated by comparing the results from the models with different tunnel surfaces. In the cases where the tunnel surface is simply waved (model 4 and 5), the load-deformation curves are quite similar to that of the circular tunnel without lining (model 1). This indicates that the simply waved lining has little support effect concerning the stiffness and strength of the model.

Figure 2 shows the comparison of the doubly waved models (model 6 and 7) with the lined circular model (model 3) together with the curves obtained from an analytical solution where the residual strength σ_{cr} of the lining is considered (Chang, 1994). It can be seen that the doubly waved models have almost the same "stiffness and strength" as the circular one, but somewhat higher

residual strength. This indicates that the irregularities of a tunnel surface will unlikely affect the general behaviour of a shotcrete lining. The reason for the higher residual strength of the doubly waved models is likely due to the contribution of the steel fibres used in the linings. Another reason could be that the irregularities of the tunnel surface provide somewhat resistance to the lining sliding along the tunnel surfaces after the failure of the lining, as illustrated in Figure 3.

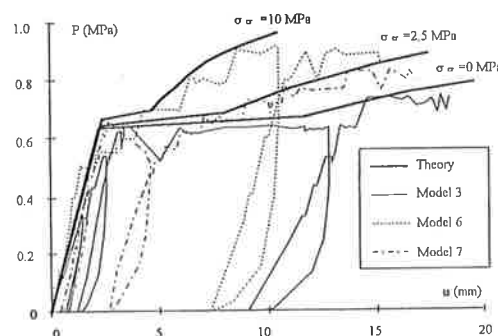


Figure 2. Comparison of doubly waved and circular models with theory

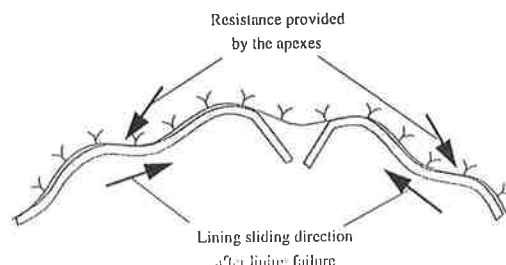


Figure 3. Irregularities provide resistance to sliding of lining along tunnel surface.

The observations from the model tests show that tensile cracks are present at the apexes of the irregularities at quite low load level. Even so, the doubly waved linings still have adequate load bearing capacity. The observations indicate that such local cracking is unlikely to affect the general safety of a shotcrete lining in the short term. But in practice it will affect the durability of the lining in the long term.

As the load increases, the cracking at the apexes develops in the lining and eventually the linings begin to split from the rock at the apexes. This indicates that the apexes are the weak points of the bond between the lining and rock, so that they must be bolted in order to enhance the bond. At the general failure of the lining, the simply waved models show a gentle failure, whereas the circular and doubly waved models show a brittle failure accompanied with large deformation, even the linings in the doubly waved models are steel fibre reinforced.

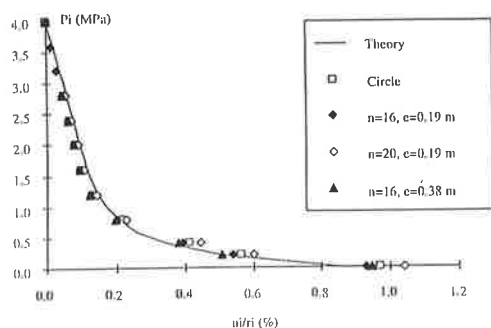
The analysis of the measured load-deformation curves from the model tests shows that the lined tunnel will deform as much as the unlined tunnel after the brittle general failure of the lining. There is a deformation leap from the lined deformation to the unlined deformation accompanied with high deformation rate, which in turn may result in a high risk of progressive tunnel failure. The theoretical studies done by the authors show that the residual strength of a shotcrete lining has substantial importance in reduction of the risk of the progressive tunnel failure (Stille et al, 1990; Chang, 1994).

After the linings have broke down, the observations show that rock pieces at the apexes behind the linings are squeezed out as the load is being maintained. This indicates that the rock is

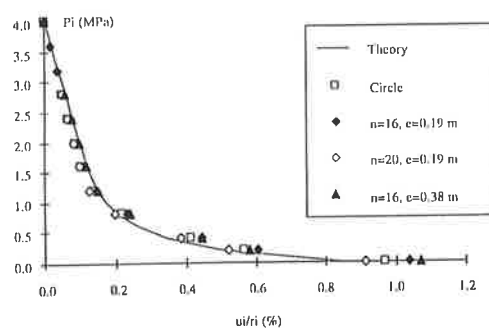
adjusting itself and forming an arch. The stress analysis done by numerical calculations shows that there is a stress concentration at the recesses. So plastic zone will start to exist at the recesses and propagate as the load increases. It can be imagined that the plastic zones starting from the recesses will eventually get coalesced and consequently the apexes between the recesses will be squeezed out.

Concerning the influence of irregularities on the behaviour of the rock mass, numerical calculation is carried out to obtain ground reaction curves of tunnels with four different cross sections. The results are given in Figure 4 which show that the ground reaction curves of the irregular tunnels are all similar to that of the circular tunnel and the analytical solution. This suggests that ground reaction curves are unlikely influenced by the irregularities. It should also be pointed out that the difference in deformation at recesses and apexes is almost negligible. The numerical results also indicate that the extension of plastic zones are unlikely to be affected by the irregularities.

Numerical calculations are also done to study the structural behaviour of flat and waved plates. The results show that a doubly waved plate has a higher bending stiffness and capacity, but lower compressive stiffness and capacity than a flat plate.



(a) GRC at recesses



(b) GRC at apexes

Figure 4. Ground reaction curves obtained from numerical calculations and compared with analytical solution (n = number of irregularities, c = amplitude of irregularities).

5 CONCLUSION

The study shows that the irregularities of a tunnel surface are unlikely to have negative effects on the safety of a tunnel lined with a shotcrete lining. Whereas the study indicates that the irregularities may contribute to the residual strength of the lining system, which in turn will reduce the risk of progressive tunnel failure when the strength of the lining is reached.

The findings in the study have significant importance in tunnelling engineering. From safety point of view, it is unnecessary to specify strict requirements on the roughness of tunnel surfaces, otherwise it will result in an uneconomical tunnel construction.

Placing more shotcrete at the recesses is also a trivial solution, which just costs more. The model tests indicate that apexes rather than recesses are the weak points of the lining system. Therefore, it is more important to ensure that the shotcrete layer covering the apexes has sufficient thickness. Bolts are suggested to be placed at the apexes to enhance the bond between the shotcrete lining and the rock. Nevertheless, attention must be paid to the local cracking at the apexes, which may affect the long term durability of the lining.

6 ACKNOWLEDGEMENT

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