Rock Joint and Rock Mass Characterisation for Nuclear Waste Repositories

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ABSTRACT

The NGI methods of characterising rock joints and rock mass have been utilised extensively for several potential nuclear waste repositories around the world. Many kilometers of drill core has been logged for the potential repository sites in England and Sweden. In addition to the standard rock mechanical laboratory testing of joints, coupled shear flow testing (CSFT) has been performed on natural rock joints for obtaining a magnitude of joint conducting apertures. This is important for estimating the permeability of the rock mass. The objectives of the CSFT tests were to produce site specific data, albeit on small scale samples, so that the effects of normal and shear stress changes, closure and shearing, can be evaluated and compared with the patterns of behaviour predicted by numerical modelling of the disturbed zone. A laboratory test program, using the CSFT apparatus, was designed to investigate the penetrative potential of a grout using different water cement ratios on joints having different joint roughness (JRC) and joint conducting aperture in different stress conditions (total normal stress, joint water pressure and grouting pres-sure). The tests revealed that joints with a conducting aperture (e) as small as approximately 25 microns can be grouted using a stable mixture of superfine cement, water and super plasticizer (dispersing agent). The minimum physical aperture (E) that can be grouted corresponds to approximately four times the cement's maximum grain size.

Rock reinforcement designs of the repositories have been derived from the Q-system statistics. Numerical modelling using UDEC-BB has been carried out for predicting the behaviour of the rock. The purpose of nu-merical modelling was to investigate the stability of rock caverns and in particular the rock reinforcements (predicting bolt loads and rock deformations), the extent of the disturbed zone (joint shearing and hydraulic aperture) with respect to cavern orientation, the effect of pillar widths, and the effect of cavern sequence.

For some specific sites, 3D modelling was used to calculate seepage into tunnels and in deposition holes, using the computer program NAPSAC. This type of calculation demands a realistic joint network and joint transmissivity data in a large scale, e.g. from several hydraulic borehole tests. The modelling results are used to give an overall indication for the direction of deposition tunnels based on the modeled fracture network. In addition, the water inflow criteria in deposition holes was checked using NAPSAC to estimate the loss of deposition holes which need to be abandoned due to high leakage.

Keywords: Rock joint; Coupled analysis; Tunnel; Nuclear waste; Stress analysis

1. INTRODUCTION

The NGI methods of characterising rock joints (using JRC, JCS and $_r$) and characterising rock mass (using the Q-system of Barton et al., 1974) have formed the basis for quantitative information for site characterisation at Sellafield for the UK radioactive waste repository and for the potential repository site in Sweden. Special geotechnical logging charts were developed for recording and presenting key engineering geological parameters including the data required for rock mass classification purposes (Q-system). This PC based chart, wherein the data is shown by means of histograms, has allowed the data logged from different areas around a project site to be manipulated and combined and thereby setting up input data files for numerical modelling of critical sections of an underground excavation. Advanced rock mechanical testing of joints which include coupled shear flow conductivity tests (CSFT) were performed on natural joints from sedimentary and volcanic rocks. The CSFT testing apparatus, which has been designed by NGI, has helped to derive the experimental data needed to quantify the effect of joint deformation on conductivity (Makurat et al., 1990). In addition, rock joint sealing experiments were conducted using this CSFT apparatus (Bhasin et al., 2002). Rock reinforcement designs were evaluated using the Norwegian Method of Tunnelling (NMT) concepts (Barton et al., 1992).

2. JOINT CHARACTERISATION

2.1 **Joint Shear Strength Parameters**

Index tests to determine JRC (tilt tests, pull tests and profiling), JCS (Schmidt hammer tests), $_r$ (tilt tests, pull tests and Schmidt hammer) have been carried out on joints recovered in the drill cores from the planned repository sites in England and Sweden. The NGI methods of tilt testing and Schmidt hammer testing are described in detail by Barton and Choubey (1977) and by Barton and Bandis (1990.

The original form of the non-linear «JRC - JCS» criterion for predicting the shear strength of rock joints (Barton and Choubey, 1977) is written as:

$$\tau = \sigma_n \tan\left[JRC \log\left(\frac{JCS}{\sigma_n} \right) + \Phi_r \right] \qquad (1)$$

where σ_n = effective normal stress and τ = shear stress.

The residual friction angle ϕ_r for unfilled joints is determined from Schmidt hammer and tilt tests using the following equation (Barton and Choubey, 1977):

$$\Phi_r = (\Phi_b - 20^o) + 20\left(\frac{r}{R} \right) \qquad (2)$$

The parameter ϕ_b is termed the basic friction angle for flat, sawn, but unpolished, weathered surfaces of the rock in question

Computerised data using the geotechnical core logging charts has enabled visualisation of both the lateral and depth variation of the various parameters for the jointed rock masses at the sites.

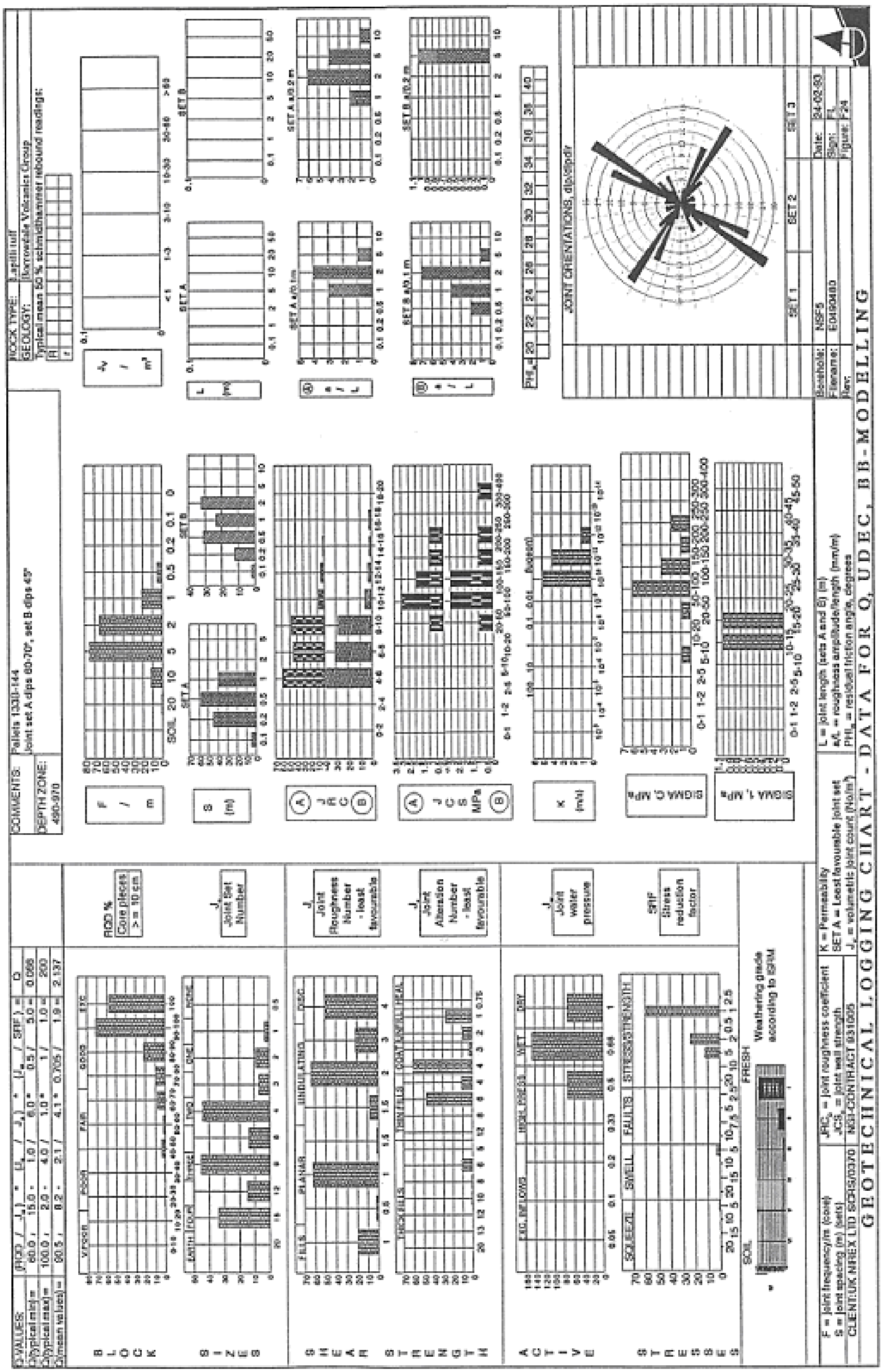
2.1 Geotechnical Logging Chart

The recording and presenting of key geotechnical parameters including the data required for rock mass classification purposes was developed at NGI for systemising the data logged from a repository (Fig. 1). The basis of the engineering geological data is the six different parameters in the Q-method: RQD (rock quality designation), J_n (joint set number), J_r (joint roughness number), J_a (joint alteration number), J_w (joint water reduction factor) and SRF (stress reduction factor). Histograms for these parameters are shown in Fig. 1. The Q-system parameters, along with other important engineering geological data, form a set of information required for the design and modelling of underground structures. The Q-system parameters occupy the left-hand side of Fig. 1. This chart is arranged in a special manner for convenience in field mapping, in core logging and in subsequent use of the information. In the middle section of the chart there are histograms for joint frequency F, joint spacing S, joint roughness coefficient JRC, joint wall strength JCS, permeability K, rock strength and rock stress. On the right side of the chart, there are histograms showing Schmidt hammer readings R, r, volumetric joint count J_v, joint length L, joint roughness amplitudes a/L, residual friction angle and joint orientation. This method of recording the six Q-system parameters and other geotechnical information during field work for small or large areas has been found to be very useful. Incorporating all the information in a PC-based spreadsheet (Excel) makes it possible to see the variation in the different parameters through the cavern. Hence, data from different areas may be manipulated and combined. The geotechnical chart contains information for setting up input data files for numerical modelling of critical sections of the cavern. A description of all the above parameters which are considered to be of importance when performing field mapping and core drilling is given by Bhasin (1994).

2.2 Depth Logs

Based upon the data recorded in the geotechnical logging charts, depth logs were prepared for the various boreholes. Figure 2 shows an example of a Q-value depth log for one of the logged drill core.

Similarly, the depth variation of other parameters such as permeability, porosity, unconfined compression strength and the six individual Q-system parameters could also be visualized.


GEOTECHNICAL LOGGING CHART - DATA FOR Q, UDEC, BB-MODELLING
JOINT ORIENTATIONS, dip/dipdir

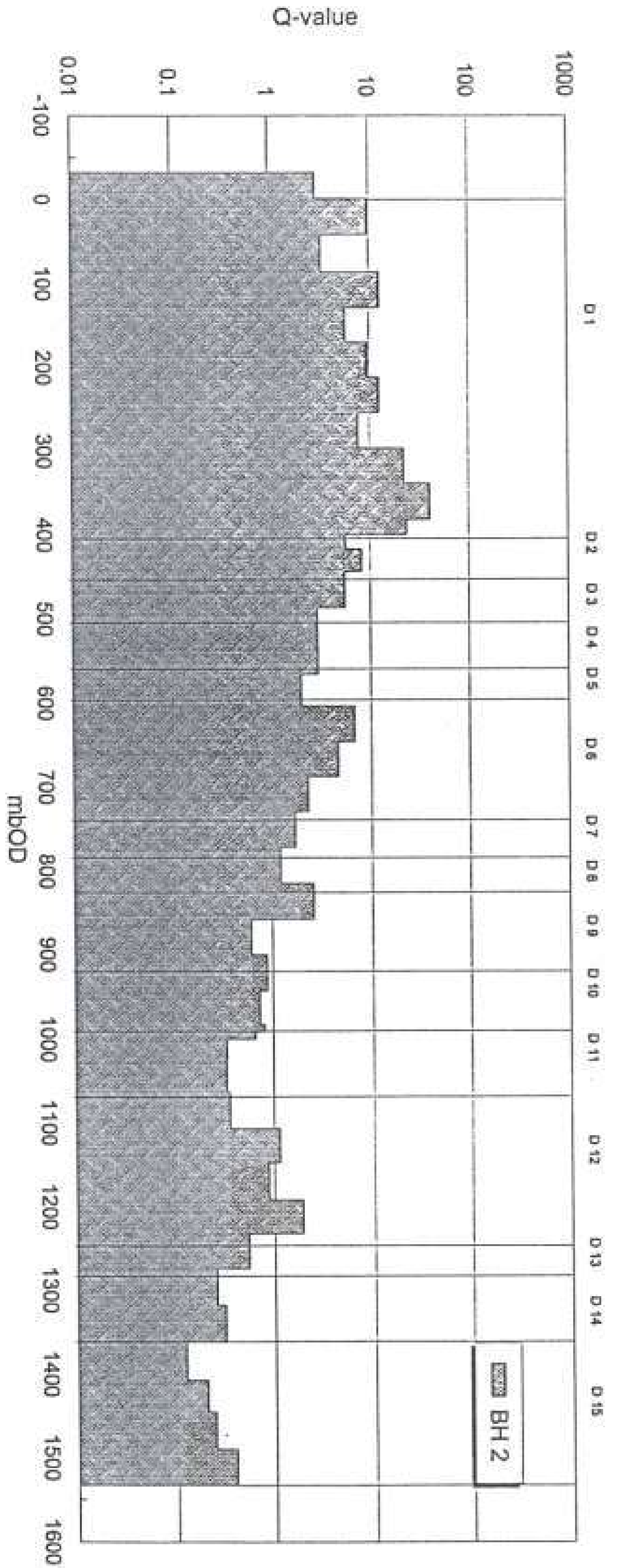


Fig. 2 - Q- value depth log for a borehole

During the logging of drill cores the Q-system parameters J_w (joint water reduction factor) and SRF (stress reduction factor) were initially estimated based on the characteristics of the cores. These parameter values were later revised based on the results obtained from the rock mechanical tests conducted in the field and in the laboratory. The J_w-values were revised based on the permeability tests carried out in the field. NGI's experience from Lugeon testing of boreholes in projects related to underground construction works were utilized for revising the J_w-values (Fig. 3).

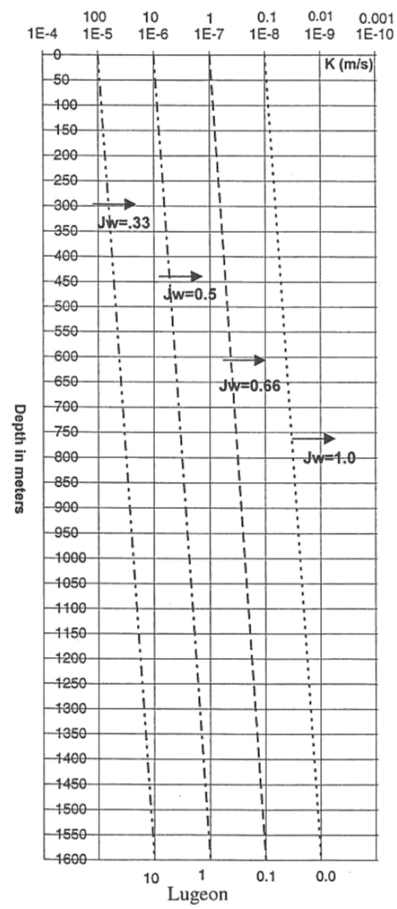


Fig. 3 - Relation between J_w and measured permeability and depth (Bashin et al., 1999)

In this figure the results from permeability tests conducted at various depth intervals were plotted for estimating the J_w values.

3. EXPERIMENTAL DETERMINATION OF ROCK JOINT CONDUCTING APERTURE

3.1 Coupled shear flow testing

Concerning nuclear waste repository safety, a key aspect is the confidence of being able to successfully seal underground excavations and demonstrate methods of reducing the permeability of adjacent rock by sealing joints and fissures. Therefore, one of NGI's rock mechanical testing programme from core logging activities comprised experimental determination of rock joint conducting apertures through CSFT tests.

Figure 4 shows NGI's biaxial cell which is primarily used for coupled shear flow temperature testing of natural joints. Samples for the CSFT tests are selected so that the joint passes approximately through the middle of cast epoxy block. The joint samples used for the tests were approximately 90mm in diameter and 150mm in length. Detailed data from characterisation of individual samples are given in Table 1. The methods of characterisation were based on tilt testing and Schmidt hammer testing of jointed samples. The sample consists of two parts. Each part of the sample was cast into a reinforced epoxy block and then mounted into the apparatus with flat jacks acting on each of the four sides as shown in Figure 4.

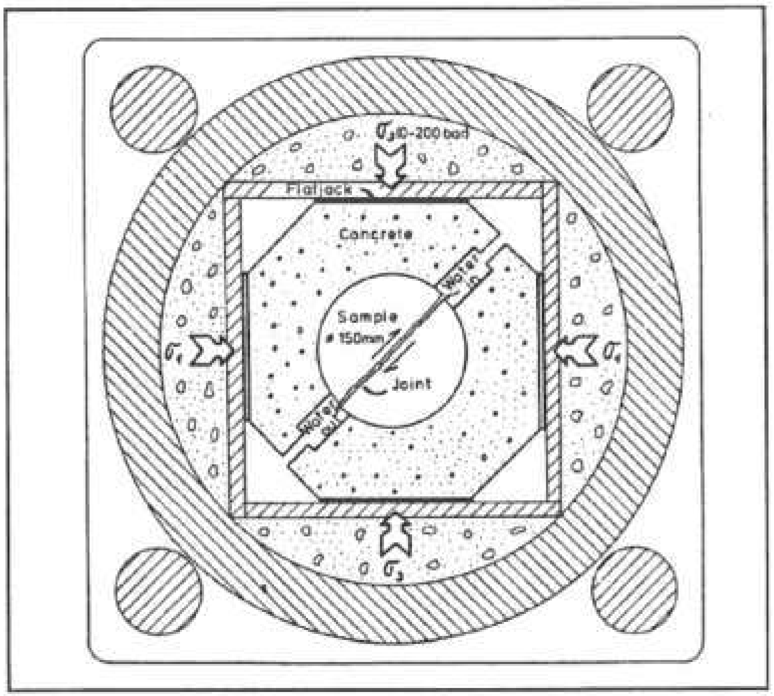


Fig.4 - NGI's biaxial apparatus for CSFT testing of rock joints

Table 1 - Characterisation data of individual samples for the CSFT tests

No.	Rock type	Depth (m)	Joint dip (°)	JCS (MPa)	JRC	ϕ_r (°)
1	Sandstone	106.72	85	60.7	4.18	28.2
2	Sandstone	106.45	86	67.2	4.38	26.4
3	Ignimbrite	503.57	70	87.1	5.44	28.7
4	Sandstone	1032.35	73	86.8	3.97	24.8
5	Tuff	609.92	75	264.5	4.22	25.3
6	Tuff	691.71	59	*	3.0	*
7	Tuff	805.54	72	*	4.0	*

* Preliminary tests not conducted due to delicate infills

By applying the same oil pressure to all four flat jacks, only normal stress is applied over the joint. This stage is referred to as "normal stress alone" or consolidation stage. The shear displacement along the joint is very small during this stage as minor seating adjustments/interlocking is taking place. The "normal stress alone" stage is followed by a shear stage where shear displacement (3 mm) along the joint is created by reducing the oil pressure in two opposite flatjacks and increasing it in the other, so that the normal stress acting along the joint remains approximately constant. Displacements normal to and along the joint are measured during all stages of the test. Fluid conductivity is measured by measuring the amount of fluid that passes through the joint (in a horizontal direction) under a constant pressure head.

Table 2 summarises the effect of normal and shear displacements in the joint conducting apertures for the tests performed.

Table 2 - Magnitudes of joint conducting apertures after consolidation and shearing

No.	Depth (m)	Normal stress alone stage		Shearing stage		
		σ_n (MPa)	Joint conductive aperture on 3rd cycle (μm)	Shear displacement (mm)	σ_n (MPa)	Joint conductive aperture after shearing (μm)
1	106.72	26	6	2.8	8.78	47
2	106.45	26	16	2.3	13.27	25
3	503.57	24	125	2.8	14.98	75
4	1032.35	26	170	3.8	17.89	114
5	609.92	31	8	3.1	9.73	13
6	691.71	30	6	3.0	17.35	40
7	805.54	30	5	5.5	16.61	15

The waste disposal in a cavern results in unavoidable disturbance of the rock mass surrounding the cavern. Small amounts of shear displacements (1-2mm) can cause dilation of jointing resulting in nearly two orders of magnitude increase in conductivity (Barton, 1982). Since the flow through a joint can be assumed proportional to the cube of its hydraulic aperture, the joint flow dominates the permeability of jointed rock masses and therefore special attention must be paid to the aperture and its change. The flow through a joint can be converted to theoretical smooth wall conducting aperture (e) using the following equation:

$$e = \sqrt[3]{\frac{Q.12.\nu}{g.w.i}} \qquad (3)$$

where

e = conducting aperture assuming parallel plate flow (m),

w = width of flow path (m),

ν = kinematic viscosity (m^2/s),

The change in aperture can result from both normal stress and shear displacement. The mechanical aperture (E) is usually larger than the corresponding smooth wall conducting aperture (e) because of the roughness of the joint (Barton et al., 1982):

$$e = \frac{E^2}{JRC^{2.5}} \tag{4}$$

where JRC= Joint roughness coefficient. This equation is only valid for normal stress conditions.

4. ROCK JOINT SEALING EXPERIMENTS

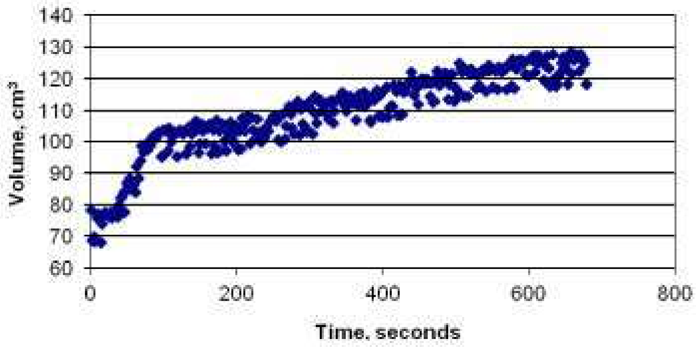
The rock joint sealing experiments for nuclear waste repository projects using the CSFT apparatus have been described in detail by Bhasin et al, 2002. This laboratory test programme was designed to investigate the penetrative potential of a grout using different water/cement ratios on joints having different joint roughness (JRC) and joint conducting aperture in different stress conditions (total normal stress, joint water pressure and grouting pressure). In the rock joint sealing experiments an ultra fine cement grout, in which 98% of the material was finer than 12 microns, has been used to study the penetrative potential of grout mixes with different water cement ratios. The grout mixture comprised of cement (Spinor A), tap water and dispersing agent (Mighty 150).

The results indicate that joints with a conducting aperture (e) as small as approximately 25 microns can be grouted using a stable mixture of superfine cement, water and a super plasticizer (dispersing agent). The penetration capacity of a specific cement grout depends, in addition to the joint's characteristics, on the maximum grain size, the water/cement ratio and the injection pressure used. The tests reveal that the minimum physical aperture (E) that can be grouted corresponds to approximately four times the cement's maximum grain size.

During the tests, the rate of grout flow and the injection pressure versus time were recorded automatically. Typical results of these recordings are shown in Fig. 5.

Figure 6 shows the result of the grouting for a sample with 80-100 micron joint opening. Approximately 50 % of the area is covered by grout material. This indicates that even a 80-100 micron joint is not 100% effectively grouted using the super fine (Spinor) cement. The effective hydraulic aperture has been reduced to about one fourth. If one assumes parallel plate flow (laminar), the hydraulic conductivity has been reduced by 64 times.

The numerical modelling of disturbed zone effects when excavating access tunnels and low and intermediate level caverns provide estimates of joint apertures before and after excavation of the tunnel caverns. The apertures are affected by normal stress changes, shearing, dilation and even tensile opening. NGI's BB model (Barton-Bandis) simulates stress and size dependent coupling of shear stress, displacement, dilation and conductivity thus enabling the prediction of jointed rock mass behaviour. The magnitude of joint conducting apertures obtained through the CSFT tests have been found to be in close agreement with those predicted by the discrete element modelling of disturbed zone when excavating access tunnels in low and intermediate waste storage caverns.



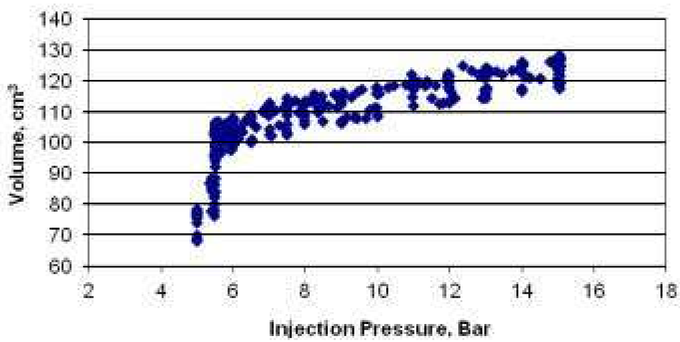


Fig. 5 - Injection test for sample 1 with water/cement ratio = 0.6, total stress = 20 bar (2MPa)

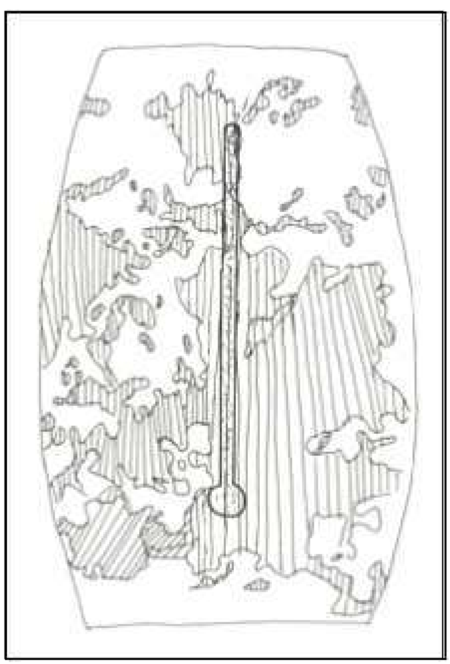


Fig. 6 - Grouted area of a sample after testing

5. NUMERICAL MODELLING OF UNDERGROUND OPENINGS AND ESTIMATE OF ROCK SUPPORT REQUIREMENTS

For the jointed rock masses found at the repository sites, modelling of the rock mass as a discontinuum is the only realistic method to simulate what could happen during tunnel, shaft and cavern excavation. NGI has used the discontinuum modelling based on the integrated use of the empirical Q-system, and on Cundall's (1980) Universal Distinct Element Code (UDEC) with the Barton-Bandis (BB) joint model incorporated (UDEC-BB).

The Q-system of tunnel support design provides recommendations for rock bolt spacing and thickness of fibre reinforced (or in some cases, unreinforced) shotcrete. Numerical modelling is utilized in NMT designs made by NGI for helping to understand the potential failure modes thereby improving on the basic empirical design. For the Sellafield repository design studies, modelling was carried out to obtain a better understanding of the stability of the caverns, the rock reinforcement requirements, the extent of the disturbed zone around the cavern areas and the effect of various pillar and crown pillar dimensions. Figures 7 and 8 show examples of the hydraulic apertures and the bolt loadings around the periphery of a cavern of size 26×16 m.

For the Swedish Nuclear Fuel and Waste Management Company (SKB), NGI carried out a preliminary estimate for the rock support in rock caverns, tunnels and shafts for a deep repository at a depth of 500 meters (see Fig. 9). The facilities is planned to consist of 8 rock caverns, and roughly 60 km of deposition tunnels and 6000 deposition holes. This estimate of the rock support requirements was based on the Q-value rock classification and the rock support design support of Grimstad et al. (2003) (Fig. 10). The values of the parameters are mainly determined from the logging of drill cores. The first four parameters of the Q-system RQD, J_n, J_r and J_a can be directly obtained from core logs whereas the joint water factor J_w and stress reduction factor SRF are estimated by using water conductivity data and the ratio between uniaxial compressive strength and in situ stresses.

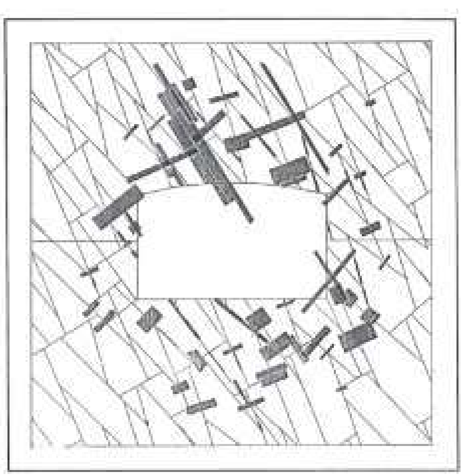


Fig. 7 - Thick lines indicating the magnitude of hydraulic apertures around the opening, maximum aperture = 2.3 mm

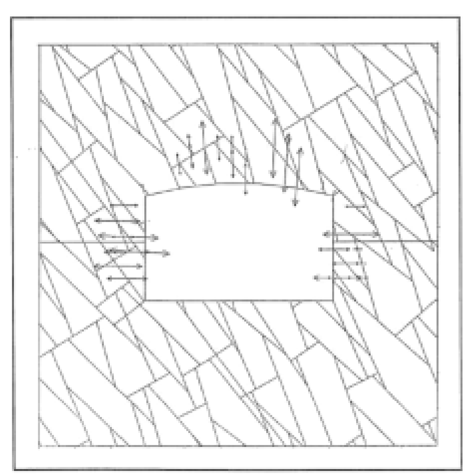


Fig.8 - Arrows indicating the magnitude of bolt loading around the cavern, maximum bolt load = 40 tons

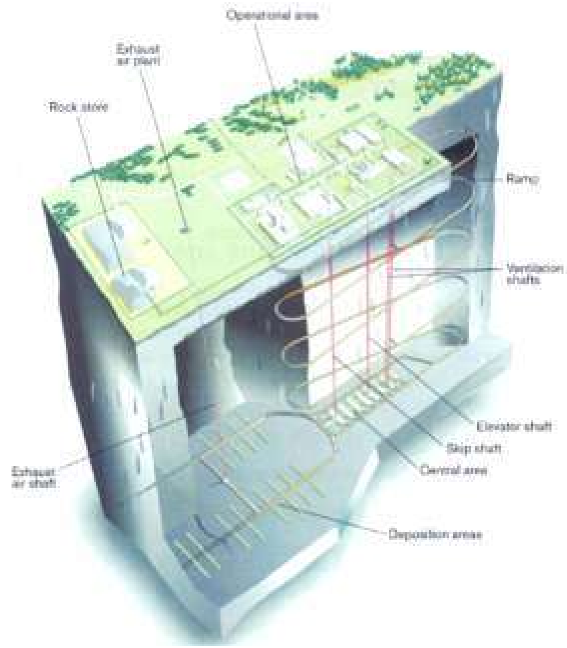


Fig. 9 - 3D illustrations of surface and underground facilities in Laxemar, Sweden (SKB, 2006)

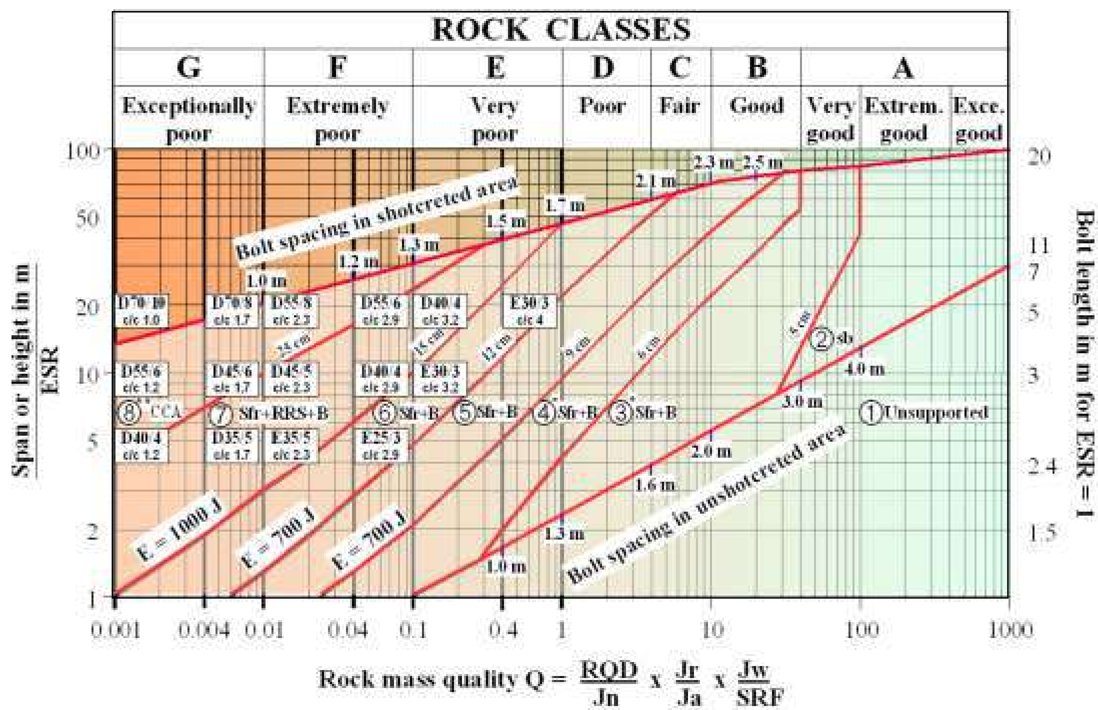


Fig. 10 - Design of rock support (Grimstad et al., 2002)

The total quantity of bolts in the complete facility shown in Fig. 9 was calculated to be between 145,000 and 189,000 pcs, of which approximately 102,000 to 133,000 pcs are in deposition tunnels. The total amount of fibre reinforced shotcrete is calculated between 12,000 and 19,000 m^3. Only 400 to 2,000 m^3 fibre reinforced shotcrete is calculated in deposition tunnels, instead wire mesh is proposed as rock support. The wire mesh is estimated in deposition tunnels to be between 219,000 and 293,000 m^2. A small amount of approximately 20 m^3 unreinforced shotcrete is calculated in the other tunnels/rock caverns.

Further, NGI has performed an investigation for SKB to find; 1) the optimal direction of the deposition tunnels, and 2) the loss of deposition holes with respect to the seepage and amount of unstable rock wedges. The seepage calculations where performed using a discrete fracture network (DFN) model and the analyses was carried out using *NAPSAC* (Version 9.0) software (www.connectflow.com). NAPSAC is a finite element software program for modelling ground water flow and transport in fractured rock. A discrete fracture network approach is used to model ground water flow through fractured rock. Such an approach portrays fractures and their connectivity in a more realistic model than the conventional models. Basically, DFN are stochastic models of fracture architecture that incorporate statistical scaling rules derived from analysis of fracture length, height, spacing, orientation and aperture.

For the direction of the deposition tunnels, a model size 400 m x 300 m x 500 m (height x width x length) was used. In the centre of this model a 300 m long deposition tunnel was simulated to calculate the seepage into the tunnel with regard to the direction of the tunnel. The numerical DFN-model was based on available data from a HydroDFN given by SKB. A HydroDFN can be defined as a fracture network which is constructed based on a classification of water conducting feature (WCF). This model is based on hydraulic data from several deep boreholes at the Laxemar site and includes fracture transmissivity and fracture length. For modelling purpose, the model area has been divided into an inner and an outer region. In the inner region (H 30 m x B 20 m x L 500 m), i. e. surrounding the tunnel, all joints that are described in HydroDFN are generated. An example is shown in Fig. 11. In the outer region, only joints with a length of more than 10 m are generated. The connectivity of a HydroDFN is significantly influenced by both size and intensity of fractures, The consequence of this is that the joint intensity, for joints larger than 10 m, has to be adjusted in the analysis in order to match a specified intensity which is termed as P_{32}in the HydroDFN model. A P_{32} intensity is defined as the total fracture area divided by the volume of model region. The direction of the deposition tunnel has been varied between 0°, 30°, 60°, 90°, 120° and 150° in relation to the largest horizontal in situ main stress at, N132° according to SKB.

For calculation of the loss of deposition holes with respect to seepage, the directions of the deposition tunnels where used. In the DFN modelling, the same 300 m-long deposition tunnel was used but with 38 deposition holes. Some results from the calculations are showed in Fig 12. The calculations showed approximately 1.4-6.1% loss of deposition holes for an inflow criteria q > 10 l/min.

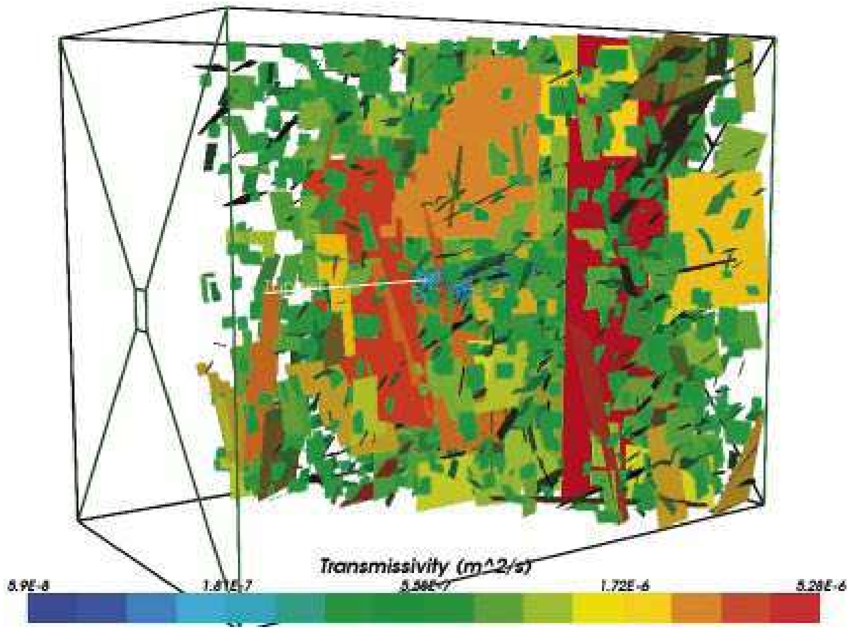


Fig. 11 - An example of a section through a realisation from the DFN model. The blue joints in centre are short fractures (SKB, 2006)

Fig. 12 - An example of a number of joints that intersect the 38 deposition holes from an NAPSAC generation (SKB, 2006)

The calculations for the volume of potential unstable wedges was performed for the same directions as for the seepage calculations. The analyses was carried out by using the program UNWEDGE (www.Rocscience.com)). The results of the wedge analyses indicates that the loss could be about 5% if we choose the volume criterion with wedges larger than 0.15 m^3. However, it is worth mentioning that a review of the results from drilling deposition holes in Äspö underground laboratory in Sweden, which is in the proximity of current study area, indicated a zero per cent loss. This indicates that the analysis from Unwedge are quite conservative and should be interpreted with caution for future design. An example of a calculation using UNWEDGE for the current stydy is showed in Fig. 13.

6. CONCLUSIONS

NGI's method of characterising joints and characterising rock masses using a highly illustrative and systematic method for recording and presenting geotechnical data has been described. The method serves as a check list for important parameters, and allows the all important variability of rock masses to be recorded and taken into account in design.

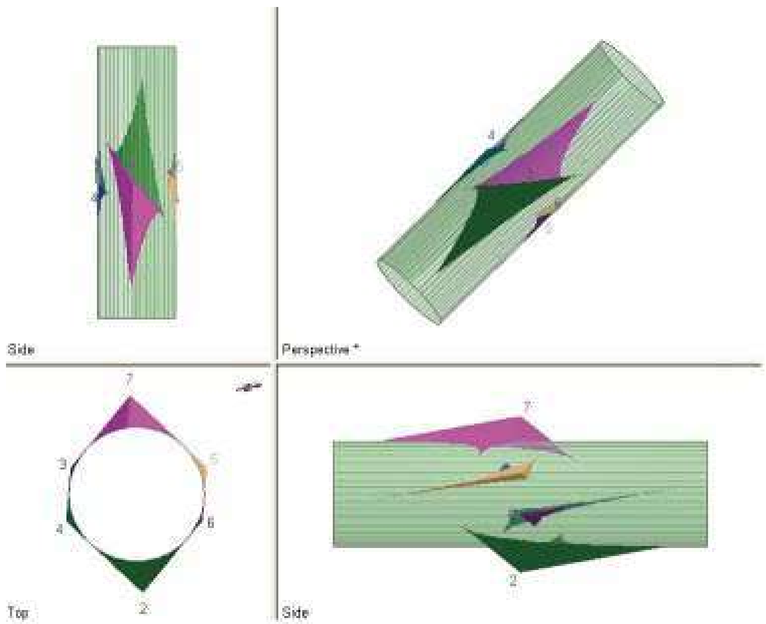


Fig. 13 - An example of a calculation with *UNWEDGE* (SKB, 2006)

An important element in NGI's rock mechanical testing programme from corelogging activities at Sellafield comprised of experimental tests for determination of rock joint conducting apertures through CSFT tests. The magnitude of joint conducting apertures obtained through the CSFT tests can be compared to those predicted by the discrete element modelling of the disturbed zone. Rock joint sealing experiments were performed using the CSFT appartatus for demonstrating methods for reducing the permeability of the rock mass. The results from these experiments indicate that a conducting aperture as small as approximately 25 microns can be grouted using a superfine cement

Discontinuum numerical modelling of the jointed rock mass surrounding underground structures using UDEC-BB has been used to predict the rock mass behaviour and to assist in the selection of the optimum cavern orientation, excavation sequence, optimum geometry and rock support of the underground excavations. NMT support procedures using the Q-system can be applied to support design and for estimating the total rock support requirements of a repository. In special cases, numerical discrete fracture network modelling can be used for optimising the orientation of underground structures and for predicting the seepage through the rock mass.

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