

GEOTECHNICAL DESIGN

Figure 1. Synthesis of Q-system tunnel support recommendations updated to include fibre-reinforced shotcrete (after Grimstad et al 1986).

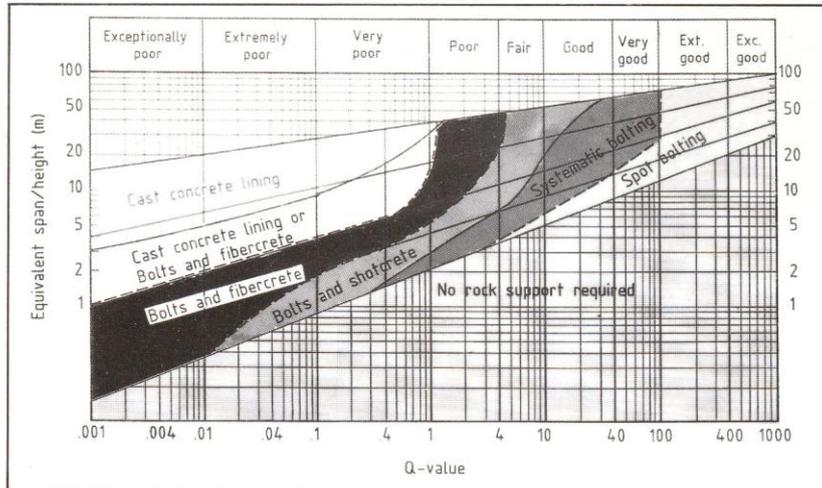
The rapid development of Norway's hydroelectric potential during the 1970s and early 80s, and major road tunnels during recent years, has had a significant influence on tunnel design work undertaken by the Norwegian Geotechnical Institute (NGI). Field investigations for some 1,200 km of tunnels, support design for 900 km and construction supervision for 600 km (both drill-and-blast and tunnel-boring machine) have set their mark on both the level of experience and on the design methods developed at NGI.

Norwegian tunnels are beset by diverse problems during drive, such as occasional major water inflows, stress-induced slabbing and rock-bursting, unstable clay-bearing jointed rock with notable joint persistence, major faulting and zones of severe swelling clay. This great variability is reflected in the huge numerical range of rock qualities (from 0.001 to 1000), described in the NGI Q-system now used worldwide. Figure 1 shows a recent (1986) update and a new feature - 'S(fr)'.

Q-SYSTEM B AND S(FR) REINFORCEMENT

Rock bolts and shotcrete as tunnel support (the B + S method) have been used in very many countries for several decades, but few would dispute the pioneering work performed in Scandinavia in the developments made with these products. In particular, robotically-applied, wet-process, fibre-reinforced shotcrete (S(fr)) has caused a revolution in support of difficult ground and has completely superseded the use of mesh-reinforced shotcrete (S(mr)) in Norway. As a result of this, a Q-system chart (Figure 1) developed by Grimstad et al (1986) already incorporated this product 5 years ago, following some six to eight years of excellent experience with S(fr) both in Norway and Sweden.

'B + S(fr)', signifying systematic bolting and fibre-reinforced shotcrete, is a flexible combination seldom matched by NATM support methods which often involve mesh-reinforced shotcrete, but can result in high labour costs and cause a "shadow" effect under spraying.



Moreover, the initially unreinforced shotcrete gives poor protection to mesh-fixing personnel.

In poor ground, or in a major excavation such as the 60m span Olympic ice-hockey cavern to be described later, it is usual to check the performance of the B + S(fr) support by convergence measurements or by MPBX extensometer installation. B + S(fr) has been used for at least a decade and gives superior advance rates and personnel safety. It is also the major component of final rock support in large caverns and tunnels through difficult ground.

ROCK MASS VARIABILITY

To those familiar with the Q-system method of rock-mass classification² the following six numbers (selected from hundreds of thousands of alternative combinations) communicate a significant amount of information on the quality (or otherwise) of the rock mass:

$$Q = \frac{80}{6} \times \frac{2}{3} \times \frac{0.66}{1} = 6 \quad (1)$$

$$\text{where } Q = \left(\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \right)$$

(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, SRF = stress reduction factor.) These numbers represent a valid description of the rock mass at a given location in a tunnel, and are associated with a specific need for tunnel reinforcement, for example B (1.5m c/c) + S(fr) 5 cm for a 15-m span road tunnel.

When surface mapping, or logging drill core, or when recording large amounts of geotechnical data in an advancing tunnel, it has been found convenient to record Q-system data in histogram form such as in Figure 2. This gives a good indication of rock-mass variability, and early

data can be combined with subsequent data, and manipulated in PC-based spreadsheet format. In the case of the ice-hockey cavern cited earlier, sets of histograms were produced from preliminary mapping in existing, nearby excavations, and subsequently combined with the results of Q-logging of 250-m of drill core³. This data base provided cavern support designers with preliminary indications of rock reinforcement needs. The system has since been used for mapping the distribution of Q-values in the arch of the huge cavern and confirming the prognoses obtained from geophysical studies. These studies are described later.

PREDICTING ADVANCE RATES

Experience in using the Q-system within NGI's group of engineering geologists is very extensive due to 600 km of rock-reinforcement supervision and design, including more than 150 km of hard-rock TBM tunnels. An interesting synthesis of experience with two drill-and-blast road tunnels of 10-m and 14-m spans driven in the early and late eighties respectively, is shown in Figure 3. The diagram shows the tunnel drive rate in m/week/advancing face as a function of the Q-value and the corresponding rock reinforcement or temporary support method. The extreme range of conditions encountered, from swelling siltstones ($Q = 0.002$) to massive basalts ($Q = 80$), also involved advance through at least a dozen rock types in the two tunnels combined.

The more rapid advance with B + S(fr) compared to cast concrete is clearly seen in Figure 3. The driving rate represents progress with both excavation and temporary support of the full tunnel cross-sectional areas of 70 and 90 m². Some 60% to 100% of the final support quantities are incorporated in the temporary

support, since cast-concrete is seldom used as a final lining. Concrete-lined sections had been completed during driving, when Q-values less than about 0.1 were encountered. Additional bolting and shotcreting is often completed soon after tunnel break-through, so the advance rates shown are quite representative for the finished structures.

USE OF Q-SYSTEM IN TBM TUNNELS

An unusual access tunnel for the Svartisen hydropower project was driven 3 km in Precambrian granites by drill-and-blast methods, and 4.4 km in Cambro-Silurian meta-sediments by TBM. It was subsequently handed over from Statkraft to the Public Roads Administration (Vegvesenet) and enlarged by drill-and-blast into a horse-shoe cross-section as sketched in Figure 4. The overburden exceeded 750m over at least 2 km and reached 1000m in one location.

As a result of this unusual progression of excavation method, one of NGI's senior engineering geologists⁴ was able to make a direct comparison between his Q-system mapping of the TBM tunnel followed by his subsequent mapping of the drill and blasted enlarged tunnel along exactly the same chainage.

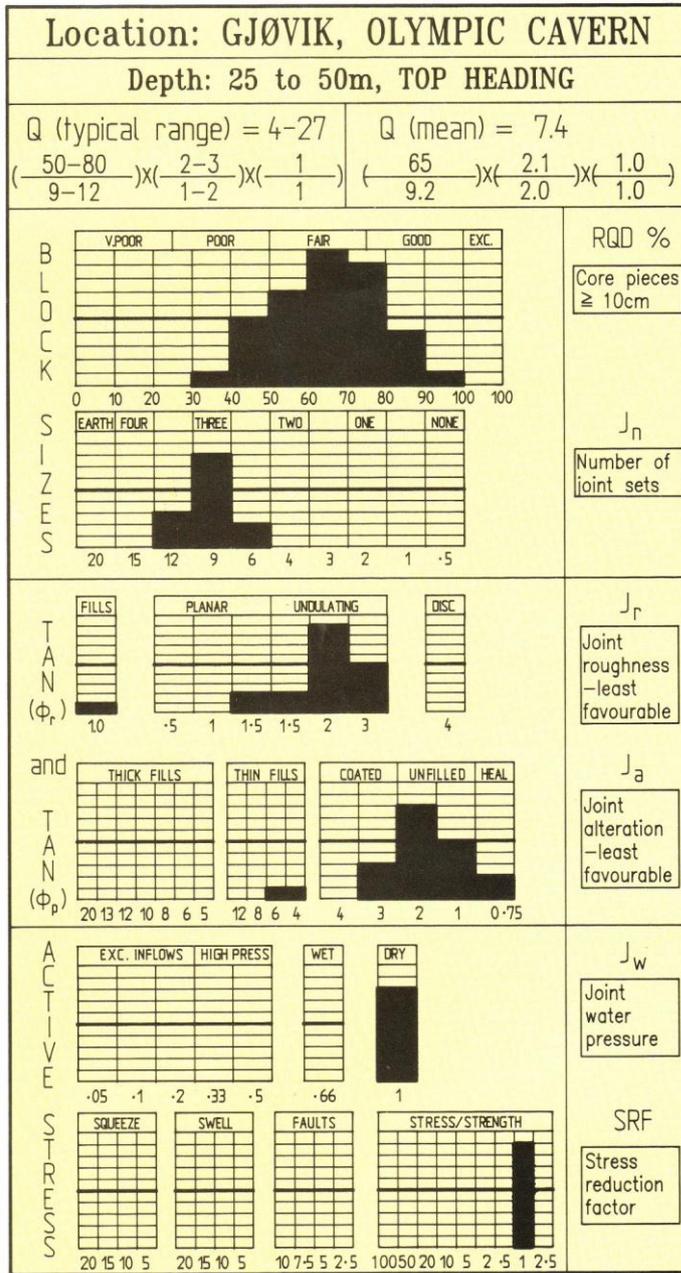
It is of course generally expected that in the same rock, a TBM tunnel will need less rock support than when drilled and blasted. A direct comparison was possible in the 4.4 km driven through marble, micaceous gneiss and meta-sandstone. Q-system mapping of the TBM tunnel before enlargement gave an average Q-value of 20.5, while it was found to be 18.5 after enlargement by blasting. Significant reductions in Q-values occurred only in relatively short sections where the values were between 4 and 30. Blasting seemed to have only slight effect in sections with TBM Q-values less than 4 (i.e., rock already affected by stress, clay or water) or where TBM Q-values were greater than 30 (massive rock requiring no support anyway).

Predicted permanent rock support needs for the TBM-related rock conditions amounted to the following for 4,400 m of tunnel: 1,500 bolts, 1,120 m² shotcrete (5 to 10 cm thick).

Predicted permanent rock support needs (and those finally used by the contractor) for the hybrid tunnel depicted in Figure 4 amounted to somewhat greater quantities of support: 2,315 bolts, 2,030m² shotcrete (5 to 10 cm thick)

The tendency to map a lower value of RQD due to blast damage, and sometimes to miss a low J_a value (clay coated joints) in the case of the smooth TBM tunnel, were two typical areas of discrepancy. The stress term SRF also indicated some differences between the two cases. Although the circular TBM excavation is usually favourable for stability, it can also represent the factor that causes higher stresses close to the tunnel wall, and possibly greater slabbing problems than with the drilled and blasted tunnel. In general, however, the TBM-mapped Q-values were 1.5 to 3.0 times higher than the drill-and-blast mapped Q-values in the sections with TBM mapped values in the range 4 to 30.

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CROSS-HOLE SEISMIC TOMOGRAPHY

Urban tunnelling through difficult fault zones with low cover, or the approach of a major fault zone mid-way beneath a deep fjord are two typical tunnelling scenarios that call for more

Figure 2. Systematic recording of Q-system data in the pilot heading of the 60m span Olympic ice-hockey cavern (Løset and Bhasin, 1991).

information on the rock mass. With good warning well ahead of the face, a tunnel

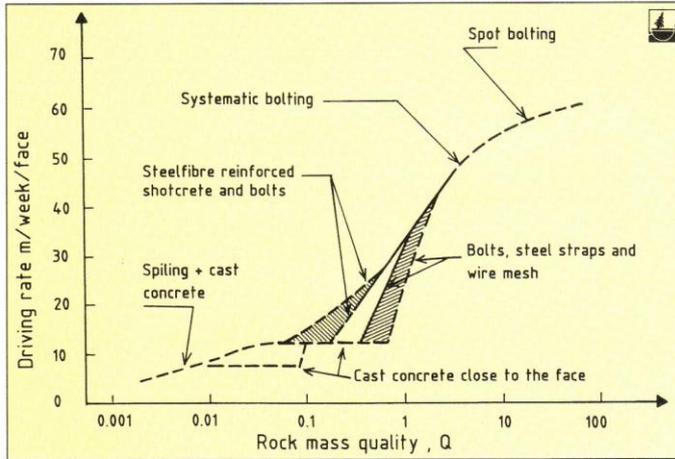


Figure 3. Tunnel driving rate including completed temporary support for 70 m² and 90 m² (10 m to 14 m span) road tunnels (based on Grimstad 1981).

contractor can plan his strategy, mobilize equipment and minimize risk. In other cases he may avoid costly over-reaction and unnecessary delays. In the case of large caverns where choice of location also exists, improved knowledge of the internal structure of the rock mass can save considerable sums in rock support if a location within higher quality rock can be found.

NGI's tunnel geophysics group have performed a dozen or more jobs within these three categories in the past five years. Other jobs have included quarry surveys, waste repository surveys, dam foundation surveys of karstic phenomena in marbles, and mapping of near-surface mining drifts in chalk. In all cases, the geological information achievable from boreholes is greatly enhanced with resulting improved cost effectiveness.

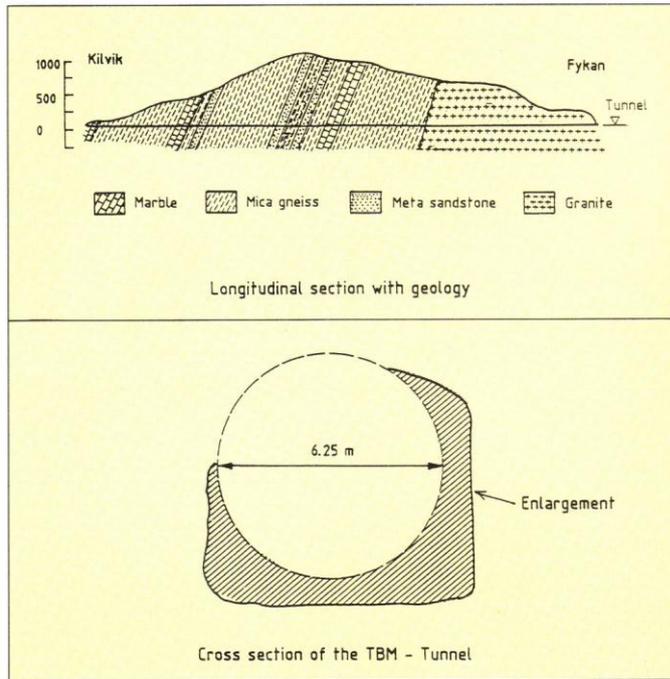
In the case of the Fjellinjen twin motorway tunnels beneath Oslo⁵, cross-hole seismic tomography was performed for a total of ten profiles, first using pairs of boreholes drilled from the surface, and subsequently using pairs of probe holes ahead of the tunnel face as the 13 m span tunnels approached a major fault containing crushed alum shale. The contractor elected ground freezing for one of the tunnels as a result of this geophysical information.

In the case of the Hvaler sub-sea tunnel, a string of hydrophones was placed on the sea bed as receivers. A pilot borehole was drilled 75 m ahead of the tunnel face for successive positioning of the signal source. Tomographic presentation of the results gave the contractor a graphic picture of the gradual narrowing of the vertical fault zone with increasing depth from the sea bed. Surface refraction surveys and probe drilling from a drilling ship had given a false impression of the width of the feature.

In the case of the sub-sea Maursund surveys, work was performed in cooperation with Vibrometric. Strings of triaxial accelerometers were set out on the sea bed, and the signal source was moved to successive positions down a 290-m long deviated borehole drilled from the shore⁶. The survey indicated the presence of two steeply-dipping weakness zones stretching from the sea bed to below the planned tunnel trace. The first one starting at 105 m was some 20 m

wide and again was more marked at the sea bed than at tunnel depth. Such information has given planners more reliable information for choosing the optimal tunnel level.

Figure 4. Svartisen road tunnel geology, overburden and final cross-section after drill and blast enlargement of TBM "pilot" tunnel (Løset, 1991).



JOINTING AND SEISMIC VELOCITY IN ROCK CAVERN

One of the most recent applications of cross-hole seismic tomography by NGI has been the survey of the site for the 60m span Olympic ice-hockey cavern at Gjøvik. This is under construction and is approaching full span; it will be used in the 1991 Winter Games in Norway. Figure 5 illustrates the results of the exploratory cross-hole seismic measurements that were performed between two pairs of boreholes along the future cavern axis and along a section perpendicular to the axis. Both the source (1 gm detonators) and the receivers (a string of hydrophones) were placed at 2.5-m intervals down the boreholes.

Detailed comparison between seismic velocity and the local RQD and joint frequency (measured along the drill core) demonstrated good correlation in the shortest vertical hole (No. 1, 45-m long). Velocity around 4000 m/s at 15-m depth corresponded to RQD = 60% and approximately 10 joints/m. At 40-m depth, velocities of about 5000 m/s corresponded to RQD = 90% and 2 joints/m. The latter corresponded to expected mid-cavern wall conditions.

Figure 6 shows a velocity-joint frequency analysis for vertical hole No.3 which was 63-m deep, and reached 5 m under the planned cavern invert. The velocity-depth gradient indicates a more or less linear reduction from 5500 m/s at the level of the planned cavern invert, to 4500 m/s in the arch and to 4000 m/s 10 m or more above the arch. Corresponding reduction in RQD or increases in joint-frequency

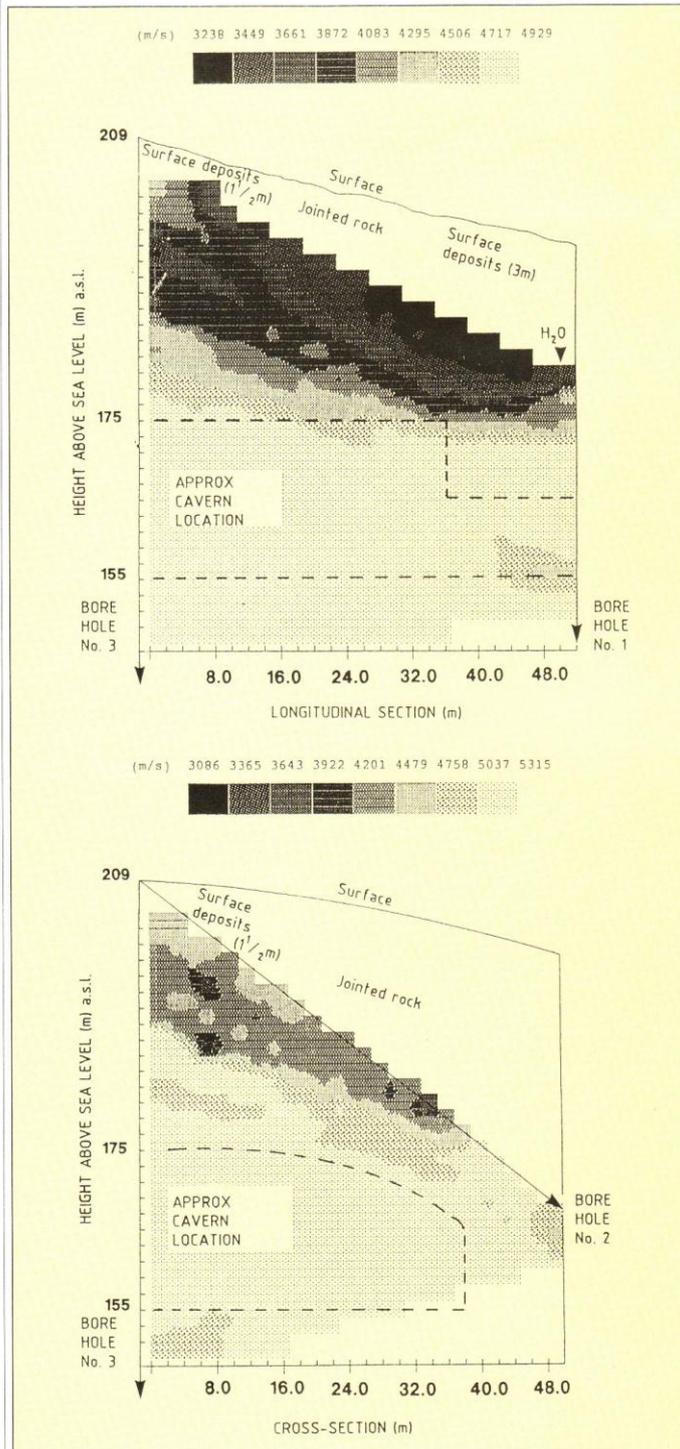


Figure 5. Cross-hole seismic tomography for longitudinal and perpendicular cross-sections through the Olympic ice-hockey rock cavern site.

are not evident in Figure 6, and therefore suggest a good correlation with the expected decrease in rock stress as the surface is approached. Joint closure with stress is a particular feature of the joint model used in the distinct element code UDEC-BB, reported later.

CORRELATING ROCK QUALITY AND SEISMIC VELOCITY

A feature of the results that has indicated good correlation between the prognosis and the excavated conditions is the reduced velocity and rock quality predicted at the ends of the caverns. Q-system mapping (Figure 2) indicated mean Q-values reducing from between 13 and 20 in the central areas to about 5 at the East end and between 2 and 5 at the West end.

Detailed comparisons of the recently-mapped Q values in the cavern arch and P-wave velocity distributions obtained from the tomography indicate (for these jointed gneisses) the following approximate ranges:

$$Q = 5 \text{ to } 15 \quad V_p = 3900 \text{ to } 4500 \text{ m/s}$$

$$Q = 20 \text{ to } 30 \quad V_p = 4700 \text{ to } 5200 \text{ m/s}$$

An approximately linear relationship: $V_p = 50 Q + 3600$ (m/s) is indicated from these preliminary results over this limited range of rock qualities. Implications for future use are that tunnel or cavern support might be designed to some level of accuracy based on careful calibration of seismic surveys against rock mass classification.

Combination of the above data with Q-system application at the Yellow River Xiaolangdi dam site and with the huge amount of *in situ* testing performed by the Yellow River Water and Hydroelectric Power Development Corporation (YRCC, MWR) indicates that the following may be a useful first approximation over a wide spectrum of rock qualities including fault zone breccia, clay inter-bedded sandstones, siltstones, thin and thickly bedded sandstones, moderately and heavily jointed gneiss:

$$V_p = 1000 \log Q + 3500 \text{ (m/s)} \quad (2)$$

$$Q = 10^{\frac{V_p - 3500}{1000}}$$

A simple easy-to-remember form of these results is shown in Table I.

Vp (m/sec)	500	1500	2500	3500	4500	5500	6500
Q	0.001	0.01	0.1	1	10	100	1000

MODELLING JOINTED ROCK BEHAVIOUR AND TUNNEL PERFORMANCE

The most important development in the modelling of rock masses has been the code

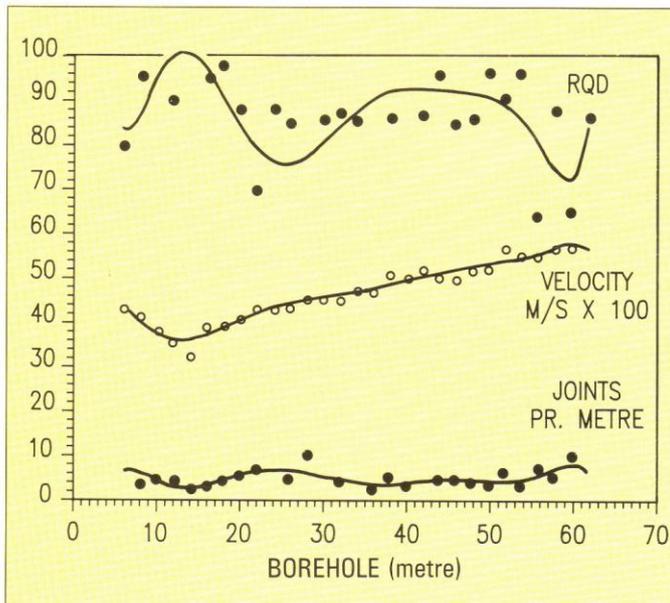


Figure 6. Example of P-wave velocity sensitivity to stress increase with depth, when RQD and joint frequency are not changing with depth; Olympic ice-hockey rock cavern site.

UDEC, developed by Cundall in 1980⁷. This finally provided an alternative to continuum modelling of rock masses, and also allowed for the deformation of the intact rock between the joints. The tendency for unfavourable joints or faults to shear or open as a result of nearby excavation could finally be represented. Improvements since 1980 also allow joint

apertures to be modelled for fully-coupled modelling of fluid flow.

NGI's engineering geologists are currently engaged in several major projects both in Norway and abroad, where acquisition of Q-system data is integrated with the extra mapping needed for discrete element UDEC modelling. In essence, the Q-system reinforcement

recommendations are checked by performing UDEC analyses. Rock bolting and anchoring are modelled using the Lorig (1985) sub-routine⁸ for representing fully-grouted reinforcement. Joint properties are described both by the Q-system parameters J_r and J_a (for approximate description of roughness and degree of clay filling) and by the Barton and Bandis (1990) joint constitutive laws⁹ which are incorporated in the NGI version of UDEC; this is termed UDEC-BB¹⁰.

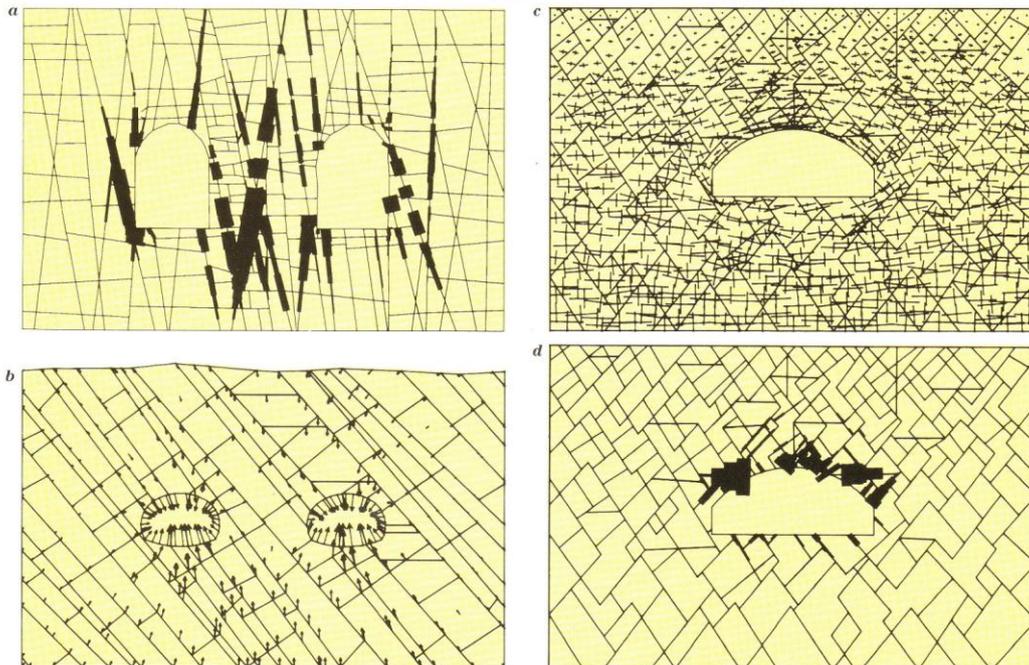
Input data for the BB model is obtained by simple index tests such as joint roughness measurement, Schmidt hammer tests, etc. This is obtained in parallel with Q-system estimates of rock mass deformation modulus. Since the mean value of modulus is given by the approximation $E = 25 \log Q^{11}$, combining this expression with equation 2 suggests that rock mass deformation modulus can be estimated from:

$$E \text{ (mean)} = \frac{(V_p - 3500)^4}{40} \text{ (GPa)}$$

for values of P-wave velocity in excess of 3500 m/sec.

Figure 7. Examples of UDEC-BB discontinuum modelling:

- a) 20 mm shearing of faulted rock between caverns of 15 m span
- b) 6.2 mm deformation of tunnel invert in bolted twin tunnels of 13 m span
- c) satisfactory stress redistribution above 60 m span cavern
- d) maximum 3.2 mm joint shearing for 60 m span cavern.



NGI rock mechanics engineers have applied UDEC-BB to a wide range of problems since 1985. Those related to tunnelling or caverns include the following:

- pressurized gas storage in jointed rock formations
- cryogenic storage (-160°C) in jointed rock formations
- bolting studies for twin motorway tunnels
- anchor cavern studies for suspension bridges
- TBM tunnels in soft rocks with liners
- damage zone modelling for nuclear waste repositories
- coupled stress-flow modelling of test tunnels
- design studies for 60-m cavern with and without bolting
- TBM access tunnels for nuclear waste repositories

Examples of UDEC-BB application to tunnel and cavern problems are illustrated in Figure 7. Of particular interest is the single 60 m span ice-hockey cavern model shown in diagram C. The high levels of horizontal stress (2 to 4 MPa at 30-m to 50-m depth) and the rough, interlocked jointing are causing predicted maximum deformations of only 2 mm to 5 mm. This is despite the mean RQD values of only about 70% (see Figure 2). When going to press, the top heading of 36-m span had caused only 2 mm to 3 mm of deformation, very close to the integrated Q-UDC-BB predictions.

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