

Quantitative Description of Rock Masses for the Design of NMT Reinforcement

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ABSTRACT

The different merits of TBM, and drill-and-blast tunnelling are compared, together with the support design philosophies of NATM (analyse-monitor) and NMT (analytical-empirical). Details of the NMT method are given, including the investigation, design, execution and contractual aspects. Improved methods have been developed for interpreting seismic data, where the velocity - Q-value relationship is modified by depth and rock strength and porosity. Extensive recent data on tunnel convergence and Q-values for tunnels of different size indicate a simple relationship between span, Q-value and convergence, which can be used to assist in confirmation of support class when tunnel logging. The method can also be used in back-analyses to estimate stress ratios. A simple relationship between RMR and Q allows stand-up time to be estimated, which can be useful in assessing TMB problems.

INTRODUCTION

Slow development, evolution and occasional revolution could be used to describe the developments made in the last 100 years of tunnelling. It may be reasonable to claim that the invention and development of the TBM, the road header, the hydraulic drill, rock bolts and shotcrete have each revolutionised the practice of tunnelling. Within each class there have been important evolutions, such as earth pressure balance (EPB) machines, rock bolts with plastic sheaths (CT) and shotcrete with fiber reinforcement S(fr), to name just a few.

Methods of tunnel design have also developed slowly, but there has been evolution and occasional revolution here also. The use of empirical design methods has evolved following slow developments, and the use of displacement monitoring likewise. Possibly we would be correct in describing discontinuum modelling as a revolution in relation to earlier continuum modelling.

In parallel with tunnelling methods (e.g. TBM, roadheader or drill-and-blast) and tunnel design (e.g. empirical or analytical or instrumental) there seem to have developed some fairly distinct schools of tunnelling which utilise different principles. Each get the job done but different speeds of construction (m/week) and different costs (\$/m) are an inevitable consequence.

TBM OPTIONS

A TBM may be the fastest and cheapest method, it may also be the fastest and most expensive method. There are also unfortunate examples of it being both the slowest and most expensive method! Net penetration rates can vary by a factor of at least 10, and completed km/year by much greater differences, due to unsuitable choices, geological accidents and so on.

The correctly designed TBM tunnel succeeds in soft rock by a certain level of overdesign of the support (e.g. concrete segments or cast concrete) which are sufficient for all ground conditions but represent overdesign in the better range of qualities. Mechanisation and speed of construction demand this "over-design", and, on a long tunnel, success (= low cost and fast completion) may be almost guaranteed.

A hard-rock TBM may succeed for different reasons, foremost of which may be the reduced or negligible support requirements, and high utilisation of the machine. This is especially relevant in hydropower projects, but - the rock must be of a suitable character to realise the benefits of such TBM tunnelling.

When a large diameter TBM gives rather slow completion and requires significant internal concrete structures, as when used as a road tunnel, it is difficult to see that the resulting high cost is defensible in relation the drill-and blast alternatives.

DRILL-AND-BLAST OPTIONS

The drill-and-blast alternatives also have their potential for quite high speed and very low cost. They may also incur slower speeds of completion and higher cost, if inappropriate (e.g. non-mechanised) methods are used and if a final nominal concrete lining is required. Use of predictive empirical methods (e.g. the Q-system) and acceptance of shotcrete and rock bolts as final lining (e.g. NMT) are likely to give the fastest tunnelling and the lowest costs, but design must be reliable, with numerical checks, and possible adjustment of the empirical solutions.

ROAD-HEADER OPTIONS

Soft rock (e.g. sandstone and shale) and hard soils (e.g. marl and fissured clays) can be readily excavated by roadheader. The usually slow progress and greater costs are a function of the real need (or assumed need) to excavate and support in multiple drifts and a real need (or assumed need) to complete the tunnel with a cast concrete lining, possibly with a membrane.

Reliance on monitoring to modify the design in response to behaviour of the temporary support (usually referred to as NATM) can help to reduce the costs of the temporary support. However it should be carefully noted that a zero deformation or convergence rate is not a guarantee of safety. Empirical methods for choosing suitable support may be a safer approach, provided the empirical designs have been thoroughly analysed. The inevitable final cast concrete lining, which may or may not be needed, usually destroys the potential economy of an NATM tunnel. Recently a final lining of fiber reinforced shotcrete has been accepted for fissured London clay. This is an important evolution, nearly a revolution.

CHANGING FROM S(mr) to S(fr)

The evolution of S(fr) as final tunnel and cavern lining occurred 20 years ago in Norway. In fact Norway's first Ph.D investigation of the properties of wet process fiber reinforced shotcrete dates from 1981 (Opsahl).

The choice of S(fr) and corrosion protected rock bolts by our designers, and the acceptance of S(fr) + B by Owners, has meant that quite a high level of experience has been built up. This

has included the evolution of the Q-system to help choose appropriate quantities of these new methods (Grimstad et al. 1986, Grimstad and Barton 1993), and the use of Cundall's UDEC (and sometimes 3DEC) to check the empirical designs.

The development of the Q-system in 1974 to assist in tunnel support selection was useful for selecting appropriate quantities of mesh reinforced shotcrete and rock bolts. In most countries, B+S(mr) is still the most frequently adopted method of preliminary support both in hard rock tunnels and in many soft rock tunnels. In some countries B+S(mr) is also accepted as final support in certain categories of hard rock tunnels.

The commercial development of fiber reinforced shotcrete in the 1970's presented tunnellers with the possibility to eliminate the labour and time-consuming cycles: shotcrete-mesh fixing-shotcrete that are involved in S(mr). It is therefore surprising that the S(fr) technology has not spread faster, both as temporary support and permanent support.

The capital cost of efficient shotcrete robots which can be used on numerous projects by successful contractors, and the higher unit price of S(fr) as a material, are each outweighed by the speed and safety of application, and the smaller quantities of S(fr) that are needed. Hard rock tunnels in heavily jointed media with frequent clay-fillings and overbreak can be treated with S(fr) (and rock bolts) in a fraction of the time needed for S(mr) (and rock bolts). Shotcrete design errors causing large rebound, and continued use of dry-process shotcrete, are perhaps two of the main reasons for the relatively slow, though accelerating use of S(fr) robots world-wide.

The logic of acceptance of B+S(mr) or B+S(fr) as final support of large caverns, sometimes in very poor rock conditions (e.g. Barton 1994) and the frequent non-acceptance of B+S(mr) or B+S(fr) as final support in much smaller tunnels by Owners in the same countries, is as illogical as it is costly. Hydropower caverns are after all full of machinery and are also the place of work for several people.

HIGH TECH, LOW COST TUNNELLING WITH NMT

The use of S(fr) as final lining and the application of the Q-system for selecting the final support are two of the key components of NMT (see Fig. 1). However, without a flexible contract system these methods could not be used to their full advantage (i.e. cutting time and costs but ensuring safety). A further necessity is that the support components have high quality so that their functional life time is assured.

Design aspects, contractual aspects and excavation and support techniques that are utilised in NMT are listed below. Many of the methods are widely used by tunnellers in many countries, in the same way that many of the components of NATM are widely used by others. Despite the commonality there are usually great differences between the final NMT product and the final NATM product. The differences usually follow through to the final cost. However the technical differences are less marked on occasion, as for example when S(fr) is accepted as final lining in an NATM-based project, or when monitoring is used as part of the follow-up in NMT "design-as-you-drive", due to soft ground sections, or due to exceptional spans.

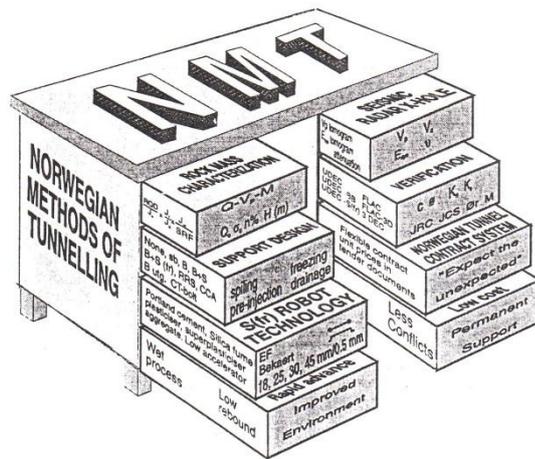


Fig. 1 Some of the key elements of NMT

I. Design methods used in NMT

1. Preliminary design is based on field mapping, drill core logging and seismic interpretations using newly developed V_p - Q relationships.
2. Rock mass quality is described by the Q -value (Barton et al. 1974; Grimstad and Barton, 1993; Barton and Grimstad, 1994).
3. Final support class is selected during tunnel construction based on tunnel logging and use of the Q -system support recommendations. These are outlined in Fig. 2.
4. Numerical verification of one or more of the various permanent support classes is performed in special cases, using the distinct element (jointed) two-dimensional UDEC-BB or three-dimensional 3DEC computer codes.

II. Contractual aspects of NMT

1. The Owner pays in principle for technically correct support
2. The Contractor is compensated *via* the unit prices quoted in the successful tender.
3. The Owner bears more risk that the Contractor thereby reducing prices.
4. Needed support is based on the agreed Q -value, and may vary frequently.

III. Support methods utilised in NMT

1. Excavation, usually drill and blast, is tailored to the rock conditions. NMT is also applied in tunnels excavated by road header or hydraulic breaker outside Norway.
2. The temporary support such as B or B+S(fr) is approved as part of the permanent support. In poor conditions, pre-grouting, spiling and use of rib reinforced shotcrete arches up to the face may be used. Cast concrete may also be needed as temporary support in some cases, cast against an articulated shield.
3. The permanent support class is chosen during tunnel advance, and will depend on the rock conditions which are systematically logged. Deformation measurements will usually be used in very and extremely poor rock as confirmation of the support class. However, it may be dangerous to assume that a zero rate of deformation signals stable conditions.
4. In general an NMT designed tunnel is drained. Insulated, pre-cast concrete panels for water (and frost) control may be used when needed in the case of road or rail tunnels. These can be assembled at approximately 1km per month.
5. The permanent rock support usually consists of high quality wet process, fiber reinforced shotcrete applied by high capacity robot, and fully grouted, corrosion protected rock bolts. These may be supplemented by rib-reinforced shotcrete (RRS) when very poor conditions are encountered.
6. Concrete lined sections will be used through fault zones, swelling clay and very weak rock that may squeeze. When the overburden and rock conditions combine to give high SRF estimates (see Table 1), both the temporary support, which will suffer significant deformation, and the final concrete lining, will obviously need careful design.
7. The use of nominal thickness, final cast concrete linings for appearance or due to tradition is discouraged due to cost, scheduling and lack of loading when Q-system designed B+S(fr) or cast concrete for assumed loading levels is already in place.
8. "Design as you drive" or "in situ selection of support", presupposes anticipation and designs for the full range of rock conditions. Tunnelling and support costs in the range of US\$ 5000 to US\$10000 per metre are normal in Norway for two-to-three lane highway tunnels using these NMT principles. Consistently poor conditions with tunnelling progress delayed by necessary heavy support will obviously cause these prices to be exceeded.

UPDATING Q-SYSTEM SUPPORT CHART

On the basis of more than 1050 new case records studied by NGI colleague Grimstad, (Grimstad and Barton, 1993) the Q-system support recommendations were recently formally updated to incorporate steel fiber reinforced shotcrete S(fr), in place of the more time consuming mesh reinforced S(mr). Use of the latter was discontinued in the early 1980's in Norway, following commercial application of S(fr) from about 1978. The new design chart is shown in Fig. 2. Bolt types and capacities, and shotcrete design including fracture energy, accelerator, and fiber type and length will depend on the detailed requirements of each case, and are not given.

TABLE 1 Updated Q-parameter ratings (see SFR) for use with Fig. 2 support categories. (Barton and Grimstad, 1994).

1. Rock Quality Designation		RQD
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100

Note: i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
 ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

2. Joint Set Number		J_n
A	Massive, no or few joints	0.5 - 1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15
J	Crushed rock, earthlike	20

Note: i) For intersections, use $(3.0 \times J_n)$
 ii) For portals, use $2.0 \times J_n$

3. Joint Roughness Number		J_r
a) Rock-wall contact, and b) rock-wall contact before 10 cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5

Note: i) Descriptions refer to small scale features and intermediate scale features, in that order.
 c) No rock-wall contact when sheared

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

5. Joint Water Reduction Factor		approx water pres. (atm)	J_w
A	Dry excavations or minor inflow, i.e., <5 l/min locally	<1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1-2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5-10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5-10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>10	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>10	0.1-0.05

Note: i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
 ii) Special problems caused by ice formation are not considered.

6. Stress Reduction Factor		SRF
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq 50m$)	5
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m)	2.5
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E	Single shear zones in competent rock (clay-free) (depth of excavation $\leq 50m$)	5.0
F	Single shear zones in competent rock (clay-free) (depth of excavation > 50m)	2.5
G	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5.0

Note: i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

b) Competent rock, rock stress problems		σ_1/σ_2	σ_2/σ_3	SRF
H	Low stress, near surface, open joints	>200	<0.01	2.5
J	Medium stress, favourable stress condition	200-10	0.01-0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after >1 hour in massive rock	5-3	0.5-0.65	5-20
M	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	<2	>1	200-400

Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_2 \leq 10$, reduce σ_1/σ_2 to 0.75 σ_1/σ_2 . When $\sigma_1/\sigma_2 > 10$, reduce σ_1/σ_2 to 0.5 σ_1/σ_2 , where σ_1 = unconfined compression strength, σ_2 and σ_3 are the major and minor principal stresses, and σ_2 = maximum tangential stress (estimated from elastic theory).
 iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure		σ_1/σ_2	SRF
O	Mild squeezing rock pressure	1-5	5-10
P	Heavy squeezing rock pressure	>5	10-20

Note: iv) Cases of squeezing rock may occur for depth $H > 350 Q^{1/2}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $q = 0.7 r Q^{1/2}$ (MPa) where r = rock density in kN/m³ (Singh, 1993).

d) Swelling rock: chemical swelling activity depending on presence of water		SRF
R	Mild swelling rock pressure	5-10
S	Heavy swelling rock pressure	10-15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, r (where $r = \sigma_1 \tan^2 (45^\circ + \phi/2)$). Choose the most likely feature to allow failure to initiate.

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

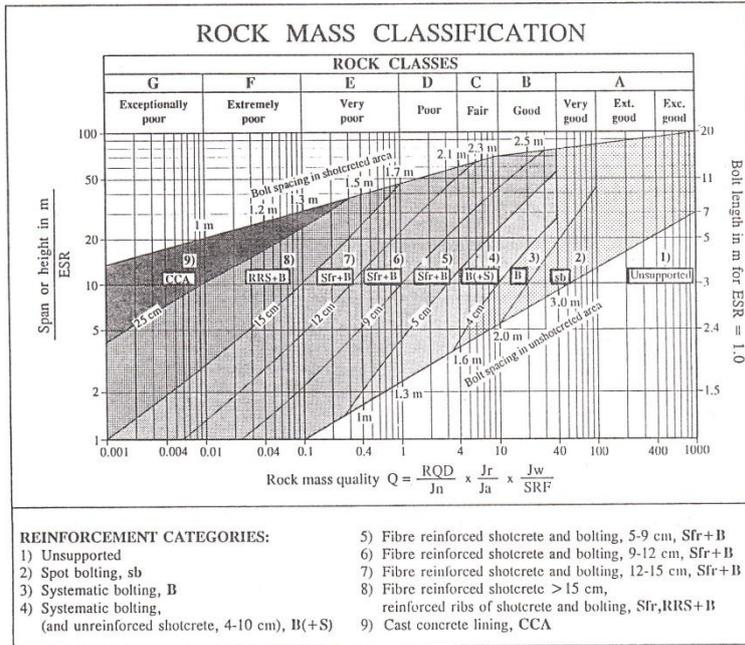


Fig. 2 Updated Q-system designs for permanent support of tunnels and caverns. (Grimstad and Barton, 1993).

Insufficient space in the support chart is available for describing the various temporary support measures that will precede #9; CCA (cast concrete arches). However multiple stage excavation, spiling (or even forepoling) and instalment in (stages) of lattice girders or rib reinforced shotcrete (RRS) will usually be required prior to casting of the final concrete lining and invert behind the steel shuttering. In cases of heavy leakage, the water must be tackled first with pre-injection or cement grout, or two component expanding grout. If water control by grouting is not possible, local drainage measures might still allow S(fr) to be utilized as part of the temporary support measures.

Use of Fig. 2 and Table 1 for obtaining guidance on support needs, requires the selection of appropriate ESR values from Table 2. These are used to moderate the magnitude of the SPAN used in Fig. 2.

Type of Excavation	ESR
A Temporary mine openings, etc.	ca. 2-5?
B Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers	1.6-2.0
C Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels	1.2-1.3
D Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	0.9-1.1
E Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels	0.5-0.8

TABLE 2 Summary of recommended ESR values (updated) for selecting safety level

QUANTITATIVE DESCRIPTION AND LOGGING OF ROCK MASSES BEFORE AND DURING TUNNELLING

In the foregoing we have summarised basic practical elements of the NMT method, with emphasis on support methods. We have looked at the end result of a design process. Design methods will form the remainder of this paper. In describing the design process we will look at the following aspects:

1. Logging of core and outcrops
2. Utilising seismic velocities to extrapolate Q-values
3. Numerical modelling for design verification
4. Logging during tunnel construction
5. Interpretation of convergence monitoring
6. Interpretation of stand-up time using Q and RMR

1. Logging of core and outcrops

The variability of rock masses suggests that a statistical method will give the most clear indication of conditions. Instead of giving a ready-made computer output of logged conditions, an original field chart is reproduced in Fig. 3. This gives the result of core logging (BH 2) in gneiss from a depth of 13 meters (= soil cover) to 51 meters. Each core box contained 3 meters of core, and the boxes have been numbered 1 to 14.

Q-parameters were logged twice in each box of core. For example box 7 (depth 28-31 m) has two recordings of RQD (70-80, 90-100) two recordings of J_n (two sets in each case), etc. The end result is sets of histograms that show the overall range of quality at a glance, while conditions at any depth can also be extracted from the histograms by the core box reference number.

The same system can be used in outcrop mapping, by using a numbering system referring to outcrops marked on a contour or geological map of the site. Photographs of the same outcrops would obviously be numbered likewise, giving a readily traceable documentation.

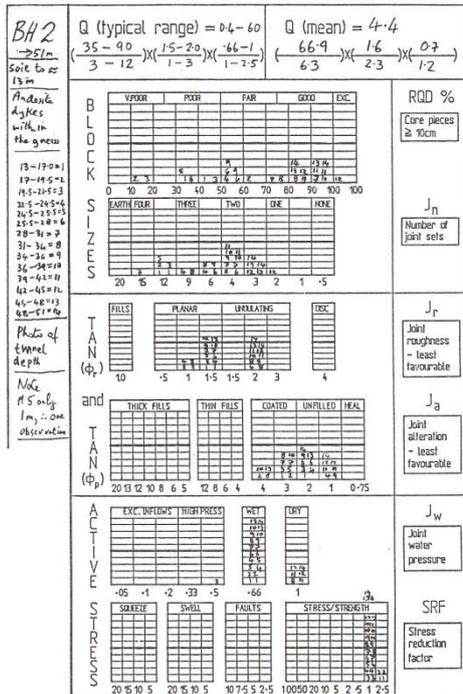


Fig. 3 Example of core logging, showing overall variability and depth - related variability.

Utilising seismic velocities to extrapolate Q-values

Limitations in the number of boreholes caused by tunnel length (or depth) and limitations in the number of outcrops, may call for the use of seismic refraction surveys. Such surveys may also be useful for locating boreholes where most information is required, and in some cases for locating pairs of holes at say 50 m spacing for seismic cross-hole tomography.

A fairly wide reaching survey of seismic velocity measurements (V_p) and Q-value estimates of drillcore from the same sites, resulted in the following simple relationship for hard rock sites at shallow depth:

$$V_p = 3.5 + \log_{10} Q \quad (1)$$

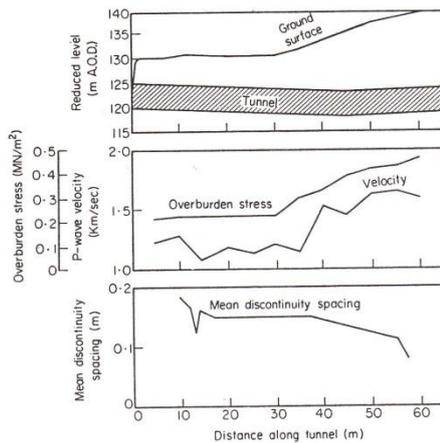


Fig. 4 Seismic survey in jointed chalk (Chinnor, UK) show increased velocity (V_p) with depth, despite increased joint spacing at depth (Hudson et al. 1980).

Although this relationship works well under the above conditions it does not take care of soft porous rocks, nor the important effects of stress level or depth.

Fig. 4 is an example of a low velocity (1.1-1.7 km/s) porous chalk which shows significantly increased velocity as the depth increases to 25m, despite an increased frequency of jointing. The reason is presumably due to stress increase, but a reducing J_a value of the joints (less weathering at depth) might also explain such a result.

At the Gjøvik cavern site in Norway, NGI recorded at seismic velocity increase of up to 2 km/s between pairs of boreholes drilled to about 60 m depth. This was despite the virtually unchanged joint frequency at depth (Barton et al. 1994). Again the explanation is most likely to be stress-related, in particular due to the high horizontal stresses (3 to 5 MPa) measured in the same boreholes.

In Fig. 5 an attempt has been made to account for the approximate effects of depth (H , meters), uniaxial strength (σ_c , MPa) and porosity (n %) on the basic "hard rock - shallow site" relation given above. The central diagonal in Fig. 5 represents equation 1. Cases known to the author have been used in the development of this chart, which also incorporates in situ modulus of deformation data. (Barton, 1996).

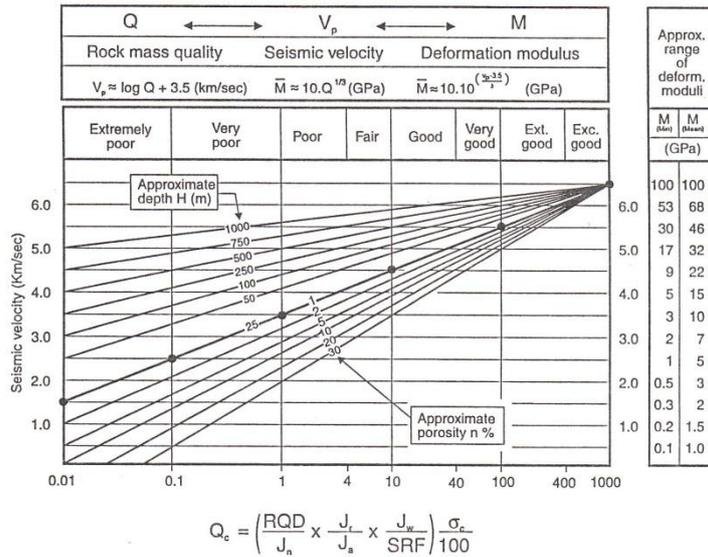


Fig. 5 Approximate, empirically based inter-relationships between Q-value, seismic velocity and modulus of deformation (Barton, 1995).

Use of this chart for extrapolating rock qualities between boreholes is fairly straight forward, but should first be limited to the same depths as the seismic survey. The logged Q-values in the upper 25 m of each borehole will be "matched" with the adjacent seismic velocity profiles. Correction of the Q-values to Q_c-values (using the ratio σ_c/100) and eventual further correction for porosity (if different from the nominal 1% typical for hard rocks) will be performed when doing this Q_c-V_p matching. (Correction may also be needed).

Subsequently, when evaluating seismic data measured between boreholes at greater depth, the approximately expected increases in V_p for the same rock quality can be read off the chart by looking at the depth correction (H, meters). Interestingly there is some evidence that sub-sea (or sub-lake) velocities from sea-bed (or lake-bed) velocity measurements are also increased by the equivalent water load, in the case of impermeable rock masses, since total stresses may be operating and the increased stress helps to increase the stiffness of the joints.

3. Numerical modelling for design verification

Important structures such as hydropower caverns, desilting chambers, or tunnels of very large sections in poor rock conditions with high stress, may each require checks of the empirical designs. The empirical Q-system support will be checked for overloading of the shotcrete or bolting. On occasion, bolt locations or bolt lengths may be seen to benefit from an adjustment, or shotcrete may need to be thicker in part of a profile due to anisotropic loading.

Fig. 6 shows a suggested range of Q-values for which discontinuum modelling will be more appropriate than continuum modelling ($Q \approx 0.1-100$). Unless a planned excavation is extremely large, it would normally not be necessary to investigate behaviour numerically at the upper end of this scale.

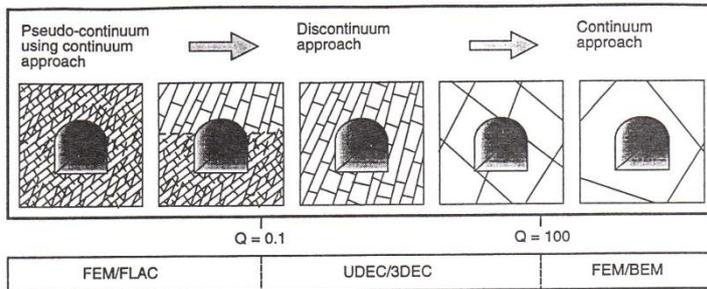


Fig. 6 Schematic diagram suggesting the range of application of discontinuum modelling (UDEC and 3DEC) in relation to the Q-value.

Necessary input data for UDEC or 3DEC modelling can be obtained from index testing of joints in drill core (to obtain JRC, JCS and ϕ , of principal joint sets following Barton and Choubey, 1977) and from field mapping. In addition to block size and large scale roughness, the deformation modulus of the rock mass (M , from Fig. 5) and estimates of the frictional strength of filled discontinuities are needed. The latter can be crudely estimated from $\tan^{-1}(J_r/J_n)$ (Barton, 1995).

Thanks to Itasca and NGI development of a non-linear joint behaviour subroutine in UDEC and realistic modelling of shotcrete (UDEC-BB and UDEC-S(fr) respectively), it is now possible to gain detailed insight into the potential behaviour of jointed rock around underground openings, and how the bolting and fiber reinforced shotcrete interact. The example of UDEC-BB/S(fr) modelling shown in Fig. 7 for the case of a sub-sea tunnel in poor quality shales and greywackés, indicates just two of the many graphical representations of shotcrete loading (axial load) and rock shotcrete interface conditions (i.e. bond failure) that can be obtained by realistic variation of assumed parameters. The code is so realistic that the shotcrete falls off the modelled tunnel wall, if the bond (and c , ϕ) are set too low - as happens in practice sometimes. More details of this shotcrete modelling technique are given by Chryssanthakis et al. (1997).

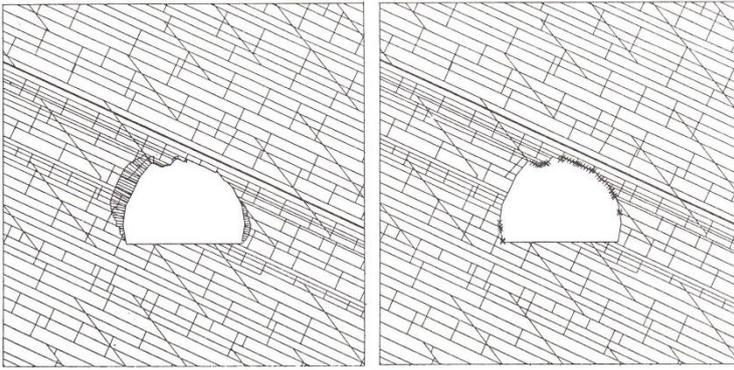


Fig. 7 Examples of UDEC-BB/S(fr) modelling of a tunnel in poor quality greywacke and shale. (Monsen, NGI 1997).

4. Logging during tunnel construction

Follow-up logging and updating of the support class prognoses is a very important part of NMT, and a very important part of high tech, low cost-tunnelling. It is at this stage that the decisions are made concerning how many m^3 of S(fr) should be applied by the shotcreting robot, and in how many layers it will be applied. A first sacrificial layer that may be damaged due to deformation will nevertheless give protection for the bolting operation, after which higher quality, stiffer shotcrete can be applied.

Fig. 8 shows two forms of tunnel and cavern log. The left hand figure gives principle rock mass structure (in summary), records temporary shotcrete use, and recommends permanent support (B+S(mr) from 1978 case record). The right hand figure shows the Q-parameter statistics for the top heading of a large cavern (Gjøvik log by Bhasin, Barton et al. 1994). This can be set up locality-by-locality, as in Fig. 3 for the case of borehole logging.

5. Interpretation of convergence monitoring

In 1980, Barton et al. presented tunnel convergence and cavern deformation measurements as a function of span and Q-value. The data was plotted in terms of Q/SPAN (on a logarithmic scale) against convergence or deformation (also on a logarithmic scale). The general trend of the data, following updating with Gjøvik cavern deformation measurements, was as shown in Fig. 9 (top), most data lying between the AA and CC lines. At this time (1994) it was arguable that a steeper trend could have been predicted.

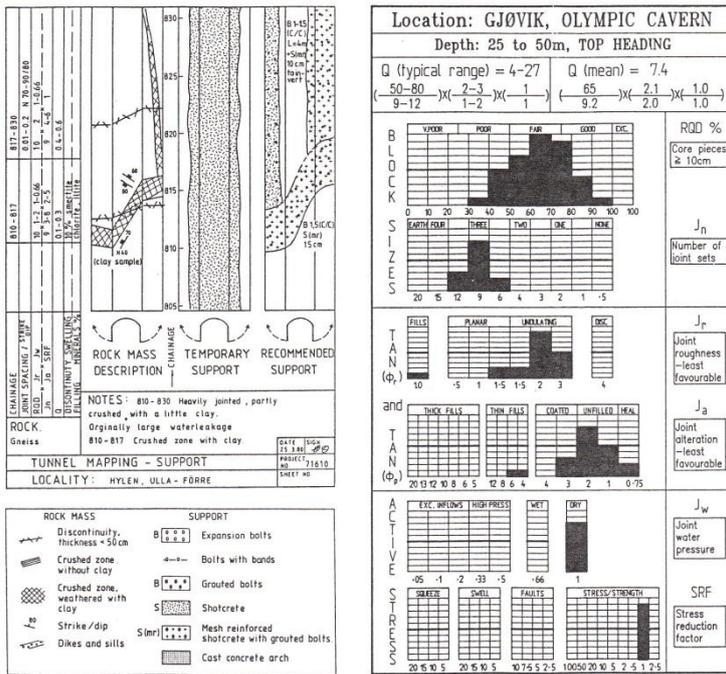


Fig. 8 Example of tunnel and cavern logging to obtain spatial information (left) and overall statistics of the Q-parameters (right) (NGI 1978, 1991).

Recently, thanks to numerous case records collected by Chen and Guo (1996) at the National Taiwan Institute of Technology, the central BB trend has been confirmed, spanning a range of $\Delta \approx 0.3 \text{ mm}$ to 700 mm . In 1997, the author discovered that the central trend line BB has the following remarkably simple form:

$$\Delta(\text{mm}) \approx \frac{\text{SPAN}(m)}{Q} \quad (2)$$

In consistent units this therefore becomes

$$\Delta = \frac{\text{SPAN}}{1000Q} \quad (3)$$

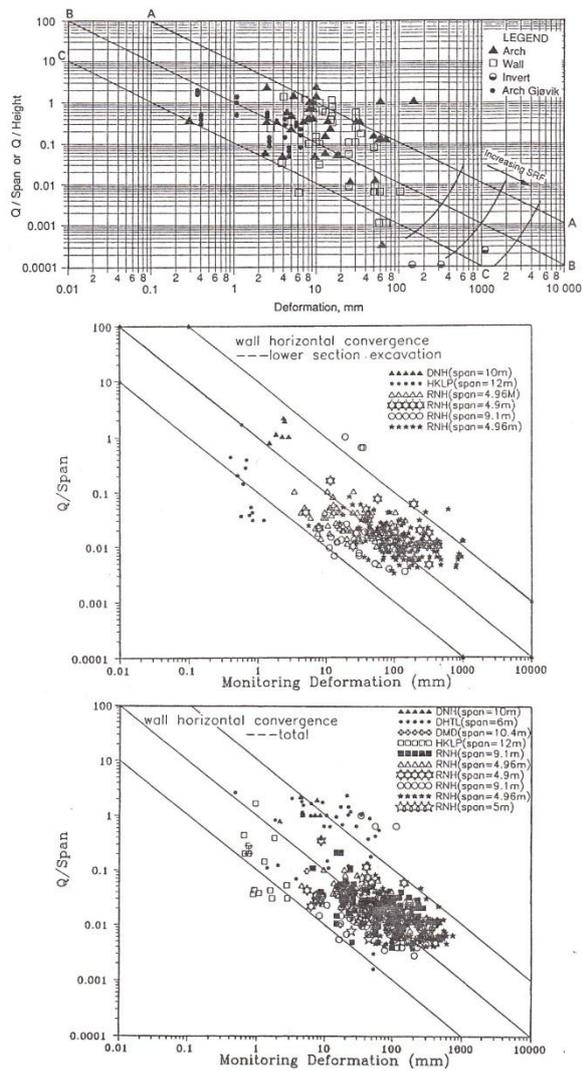


Fig. 9 Span/Q versus deformation. Barton et al. (1994), Chen and Kuo (1997).
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The large spread of data on either side of this trend line (both smaller and larger Δ) has several potential causes including over-support or under-support, shallow or deep siting of the tunnel, and varying rock moduli and/or compression strength.

If we ignore the largely unknown factors of over-or under-support, we can perhaps explain much of the range of data with the simple devices of depth (increased Δ with increased depth) and compression strength (increased Δ with reduced σ_c). A direct proportionality of Δ with these two parameters gives too strong a response, and the following dimensionless form was therefore tried following equation 3:

This equations was first developed by adding the ratio H/σ_c . The addition of density (γH) to make the ratio dimensionless, results in the form given below.

$$\Delta_v \approx \frac{SPAN}{100 \cdot Q} \sqrt{\frac{\sigma_v}{\sigma_c}} \quad (4)$$

where σ_v is the vertical stress, σ_c is the uniaxial strength and Δ_v is the vertical deformation of the arch.

For example the Gjøvik cavern with span of about 60 m (60,000 mm) showed a total "radial" arch deformation of about 6 to 8 mm, and had a mean Q-value of about 10. the average depth was 40 m (say $\sigma_v = 1$ MPa) and the average uniaxial strength $\sigma_c \approx 75$ MPa (tectonised gneiss) substituting in equation 4 we get:

$$7mm \approx \frac{60,000}{100 \cdot 10} \sqrt{\frac{1}{75}}$$

The right hand side of this approximation is calculated to be 6.9 mm. Normally the agreement would not of course be as close as this. The influence of horizontal stress on Δ_v is also unknown in this case.

We will define Δ as the absolute radial deformation. The above Gjøvik cavern arch deformation is the absolute value (instrumentation was installed from the surface prior to excavation). When measuring convergence in a tunnel, at least half of the "elastic" deformation has usually already occurred at the face. Subsequent deformation is however often larger than the theoretical remaining 30-50 %, due to non-elastic effects, so it may be reasonable to assume that the absolute radial deformation is of similar magnitude to the convergence (wall-to-wall).

It is logical to assume that the same form of equation can be used for wall displacements in relation to horizontal stress level. Using the total height of a cavern, and the relevant horizontal stress component (σ_h) perpendicular to the cavern wall, we can write:

$$\Delta_h \approx \frac{HEIGHT}{100 \cdot Q} \sqrt{\frac{\sigma_h}{\sigma_c}} \quad (5)$$

We can test these two equations against the recently constructed Nathpa Jhakri power house. The following measured data has been used in numerical UDEC-BB) modelling of this cavern, reported by Chryssanthakis et al. (1996).

Span ≈ 20 m $\sigma_v \approx 6$ MPa $\sigma_c \approx 35$ MPa
 Height ≈ 50 m $\sigma_h \approx 4$ MPa (perpendicular to wall) $Q \approx 3$

Measured deformations (where MPBX are installed) are approximately 25 mm in the arch and up to approximately 50-55 mm in the walls, though there is significant variability here. Equations 4 and 5 give the following excellent estimates:

$$\Delta_v \approx \frac{20,000}{100 \cdot 3} \sqrt{\frac{6}{35}} \approx 28 \text{ mm}$$

$$\Delta_h \approx \frac{50,000}{100 \cdot 3} \sqrt{\frac{4}{35}} \approx 56 \text{ mm}$$

Some words of caution are in order here. The data base does not include soft ground tunnels with exceptionally low compression strengths. Nor does it include exceptionally low local Q-values where collapse would occur unless exceptional support measures were installed. The data shown in Fig. 9 in fact lies between the limits of SPAN/Q (with span in meters) equal to 0.5 to 250. Outside this range and even close to these limits where data is sparser, equations 2, 3, 4 and 5 should not be applied.

It is important, and also of interest to observe that equations 4 and 5 only work in a realistic manner when the stress level is significantly lower than the compression strength. Such cases represent moderate levels of reinforcement and support, and this support moves with the rock mass keeping it "together" until stress-redistribution is complete. This is especially true in large caverns. If on the other hand equations 4 and 5 are tested in high stress, squeezing environments, in fact outside the top end of the SPAN/Q data range, then huge deformations are predicted, as if the tunnel was closing. In practice the support in such cases (e.g. cast concrete, circular steel arches, forepoling, spiling etc.) completely changes the character of the ground, preventing the "natural" deformation of several meters magnitude (i.e. tunnel closure!) It can be speculated that the large predicted deformations will in practice sometimes represent actual collapses, and the need for re-support, possibly with re-excavation and new support if tolerances are reduced too much by the continued squeezing. It is not easy to assign a deformation magnitude on a case where collapse occurs.

Although one should proceed with caution in such matters, there may be grounds for suggesting that in cases well within the SPAN/Q data range, a measured convergence (or approximate absolute radial deformation) could be used for estimating the value of K_o . From equations 4 and 5 we find the following simple approximation:

$$K_o = \frac{\sigma_h}{\sigma_v} \approx \left(\frac{\Delta_h}{\Delta_v} \right)^2 \left(\frac{SPAN}{HEIGHT} \right)^2 \quad (6)$$

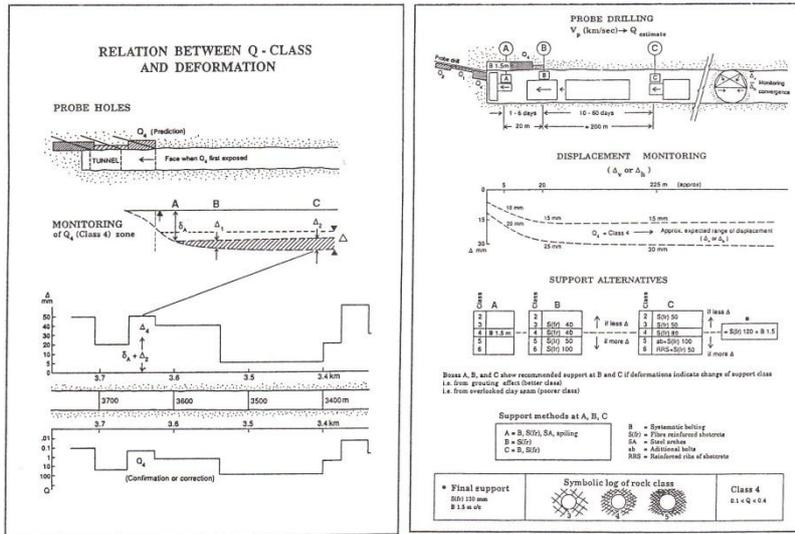


Fig. 10 Idealised use of follow-up logging (Q), deformation (or convergence) monitoring and seismic measurements in probeholes, for confirming the rock class and support class and deciding how to phase the application of support in a large TBM tunnel, (Barton, 1996).

With Nathpa Jhakri power house predictions given previously we have:

$$K_o \approx \left(\frac{56}{28}\right)^2 \left(\frac{20}{50}\right)^2 \approx 0.64$$

Measured values used previously were $\sigma_n \approx 4$ MPa, $\sigma_v \approx 6$ MPa, i.e. almost the correct result is shown.

We are now able to utilize with more confidence monitoring data (Δ_v , Δ_h) as a confirmation or correction of the assumed rock class and support class. This can be done as indicated in Fig. 10, by comparing Q-logging at the face (or behind a TBM tail shield) with the convergence data. If probe holes are used with down-hole seismic measurements (VSP-style) then a further check is possible using the V_p -Q-relationship shown in Fig. 5.

Fig. 10 (right) shows how the assumed “class 4” support (S(fr) 120 mm + B 1.5 c/c) could be applied close to the face and further behind the face, if direct tunnel wall Q-logging and convergence-measurement-magnitudes each confirmed that the first estimate was correct. If the early estimate proved incorrect, due to Q-logging of the newly exposed rock and evidence from unexpected convergence, then a correction to the planned additional support would be made at drilling/shotcreting stations B and C. It is emphasised that use of probeholes (percussion drilled, to reduce time during the maintenance shift) is an essential tool for avoiding a stuck TBM - of which there are many around the world at any one time.

6. Interpretation of stand-up time using Q and RMR

The RMR method of rock mass classification developed by Bieniawski (e.g. 1989) appears to have been focused most strongly on the stand-up time of excavations (especially from the mining industry), where support (the so-called “roof span”) is a key operational parameter. Although there have been several changes in RMR parameter ratings since 1973, it may be worthwhile to access Bieniawski’s data base, in case the information is of help in interpreting stand-up time problems encountered with TBM. In such tunnels, there are always several meters of advance (sometimes up to two TBM diameters) before support can be installed (i.e. circular ribs, bolts, shotcrete, or PCC element linings, Sharp et al. 1996)

A stand-up time that is less than the time it takes to bore anything for about 3 to 20 meters (depending on TBM diameter) may therefore mean excessive deformation, and pressure on the tail shield and trailing fingers. In the worst cases, when Q-values are as low as say 0,10, the rock may cave at the face and block the head of the TBM, collapsing further when the TBM is pulled back. Stand-up time may be very critical in projects where the TBM solution is incorrect, due to excessively poor rock conditions.

Although the RMR and Q-systems of rock mass classification utilise some different parameters, experience shows that there are some broad similarities when the two are compared, for instance in a detailed core logging exercise. Fig. 11 shows the results of one such exercise, and many more can be studied in the literature and at project sites. Of course when high stress or low strength become important variables, the systems tend to diverge, as the SRF term in the Q-value (representing the ratio σ_c/σ_1) does not appear in the RMR, and σ_c does not occur directly in the Q-system. However, the term Q_c :

$$Q_c = Q \times \frac{\sigma_c}{100} \quad (7)$$

used when correlating Q with V_p and M (Fig. 5) may correlate with RMR better over certain areas, but not when the stress level is a major issue. The number of joint sets will also cause differences to appear in the two systems.

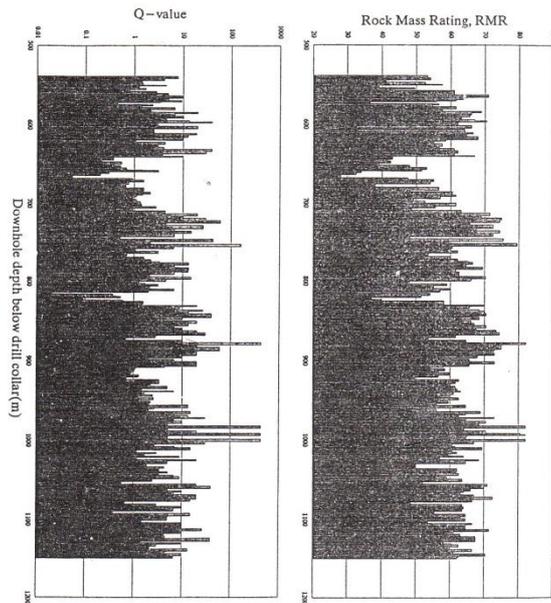


Fig. 11 Comparison of Q-log and RMR-log in the same 600 meters of drill core (NGI, 1993).

An analysis of available Q-RMR comparisons in the literature and the addition of hundreds of additional data points from NGI's recent comparisons (e.g. Fig. 11) indicates that the following is a useful working relation. It has the advantage of being easy to use away from a calculator, since it is based on \log_{10} rather than natural logs.

$$\text{RMR} \approx 50 + 15 \log_{10} Q \quad (8)$$

This relation has been used to superimpose approximate Q-values on the Bianiawski (1989) stand-up time chart. The result is shown in Fig. 12. For example, weak heavily sheared phyllite with a Q-value as low as 0.01 is, according to Fig. 12, likely to collapse before a large TBM has advanced enough (minimum 4 m) to secure the rock in question with circular steel arches. This agrees with some recent experience.

CONCLUSIONS

1. A brief review of tunnelling methods (TMB, road header and drill-and-blast) and of three tunnel design philosophies (empirical/analytical - NMT, observational-NATM, "over-

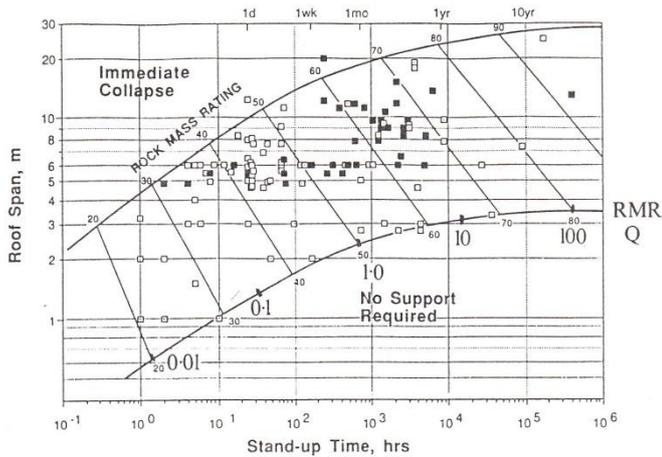


Fig. 12 Bieniawski (1989) stand-up time data as a function of roof span (= unsupported advance of tunnel). Approximate Q-values have been superimposed using equation 8.

design"-TBM) reveals the advantages of each method under appropriate conditions. The most significant ratio in tunnelling is (meters/month/cost) x (safety).

2. Choice of NMT to minimise costs and speed advance in jointed rock is strongly linked to the use of high tech, low labour methods such as robot application of S(fr) and reliance on special corrosion protected (CT) rock bolts.
3. Acceptance of B+S(fr) as final reinforcement and lining of NMT tunnels represents such a large cost saving that a modest investment in geological investigations (i.e. mapping, drilling, seismic, Q-logging) seems justified. Empirical, Q-value based designs for reinforcement and support should also be checked by discontinuum modelling (e.g. UDEC-BB, UDEC-S(fr)) when conditions or tunnel dimensions are challenging enough to make such documentation important.
4. Methods have been demonstrated in the paper for efficient quantitative logging, for interpreting seismic measurements in more detail than is usual, and for interpreting convergence or deformation measurements as part of the design-as-you-drive method. Convergence magnitudes help to confirm rock class estimates, and the designed support. However, zero rates of convergence on their own do not guarantee stability. Quantitative classification of conditions and well proven designs are more reliable than reliance on zero rates of deformation.

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