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Chapter 8: Design of Tunnel Support

8 Design of Tunnel Support

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8.1 Introduction

Central Europe has long traditions in tunnel construction. A number of large tunnel projects were constructed in the nineteenth century (including the St. Gotthard, Arlberg and Simplon railway tunnels) during which adverse tunnelling features, many of them unexpected, were encountered, causing enormous construction challenges and problems. The squeezing effect often encountered in weak ground has been a constant problem in Alpine tunnelling. The need for improved tunnel excavation and support techniques has, therefore, existed for decades. Several pioneers have made important field observations which have contributed to the development of tunnelling in weak ground.

The New Austrian Method (NATM) made use of earlier experience gained from tunnelling over several decades, but took advantage of the new support technology that was made available in the late '50s and '60s. To divide this new method from earlier tunnelling practice – where heavy rock supporting works were installed – it was called the New Austrian Tunnelling Method. Officially, the NATM was introduced by Rabcewicz at the 13th Geocolloquium in Salzburg 1962. This new trend in Austrian tunnelling soon gained national attention. International recognition was achieved through a paper in *Water Power* (Rabcewicz, 1964). Since then numerous tunnel projects in Central Europe and also in many other countries have been excavated and supported using NATM principles.

Parallel to the launching of NATM in Austria and Central Europe, great advances were made in excavation by the drill and blast method in Scandinavia. Development from pneumatic to hydraulic drilling machines, and from hand-held borers to hydraulically operated, mobile jumbos made large contributions to the rate of tunnel excavation. At the same time the development of rock support methods such as rock bolts, cast-in-place concreting techniques, and especially the spraying of shotcrete have pushed forward the technique of tunnelling in jointed rocks.

From numerous tunnel and underground excavations, mainly for hydropower exploitation, the Norwegians have made important advances in the development of tunnelling techniques. Annually, about 100 km of tunnels have been excavated since the beginning of the '60s. The need for design methods based on a numerical classification of the rock mass through which the tunnel would pass was realized in the development of the Q-system (Barton *et al.*, 1974), which incorporated a large amount of tunnelling experience. Since its presentation in 1974 the Q-system has been applied increasingly in tunnelling projects worldwide.

The main contribution to tunnel excavation efficiency and cost cutting in the last 15 years has been the development of sprayed concrete (shotcrete). The Norwegians have been in the forefront in improving this method for rock support, especially through their development of the wet spraying method and later in refining fibre-reinforced shotcrete. As is discussed in Section 8.3, the application of this method in place of the dry method and mesh reinforced shotcrete, has effectively increased the rate of tunnel excavation, improved the tunnel working environment, and kept tunnelling costs stable, even producing a reducing trend. The technique, which incorporates the Q-system, is now termed the Norwegian Method of Tunnelling (NMT).

8.2 The New Austrian Tunnelling Method (NATM)

Introduction

It is important to note that the new Austrian tunnelling method (NATM) has been developed for tunnelling in *weak or squeezing ground*. Such ground requires the use of structural supports, either to re-establish equilibrium or to limit displacements around the tunnel. The rock or soil material itself may be soft or hard.

The method was developed by L. von Rabcewicz, L. Müller. and F. Pacher between 1957 and 1964. It is essentially an empirical approach, evolved from practical tunnelling experience. Rabcewicz (1965) stated the goals of the NATM as:

"To provide safe and economic support in tunnels excavated in materials incapable of supporting themselves, e.g. crushed rock, debris, and even soil. Support is achieved by mobilizing whatever humble strength the rock or earth possesses."

"To use surface stabilization by a thin auxiliary shotcrete lining, suitably reinforced by rockbolting and closed as soon as possible by an invert.".

Later, other important features were introduced and incorporated into the NATM, such as contractual arrangements, excavation procedures and more advanced design methods. It is basically a 'build as you go' approach based on monitoring, backed by theoretical considerations.

The NATM has sometimes been assumed to be synonymous with the use of shotcrete during tunnel construction, probably because this rock supporting method is often applied in connection with the NATM. This is wrong; in practice the NATM involves the whole sequence of weak rock tunnelling from investigation during design, engineering and contracting to construction and monitoring. Consequently, an overview of the NATM will include most features involved in the execution of a tunnelling project and tends to be a comprehensive work on tunnelling. In this chapter, therefore, its main elements are described briefly. Important features of the NATM such as the accumulation and active use of construction experience are not included.

The main design principles in the NATM

A basic principle in the NATM is to take advantage of the load-bearing capacity of weak rocks. This is achieved by utilizing the property rock masses have to dilate or bulk as they yield. During this process the high ground stresses close to the tunnel dissipate and the surrounding rock mass is transformed from a loading body into a

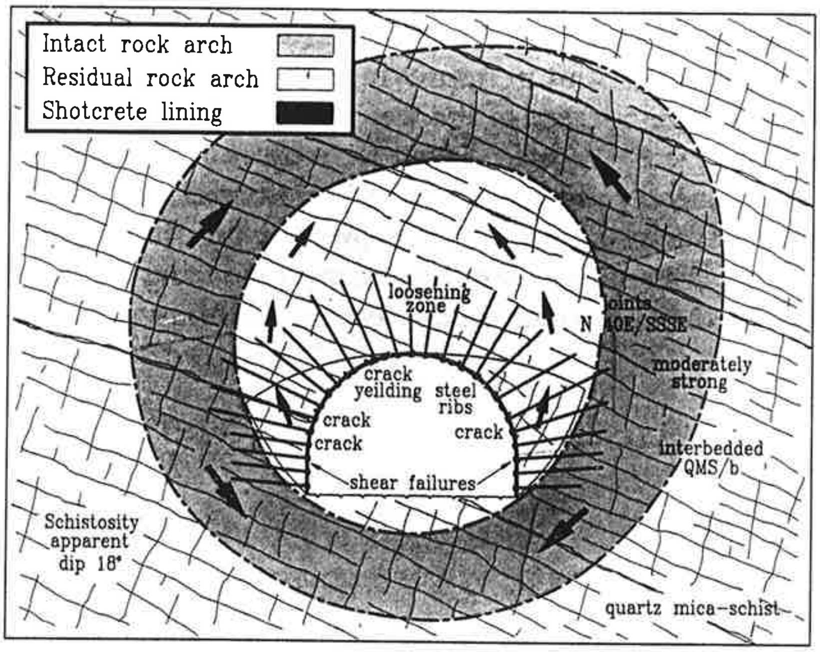

Figure 8.1 *The various zones around a tunnel in weak ground (after Hagenhofer, 1990)*

load-carrying element (Figure 8.1). Only a reduced support is therefore needed to confine the unstable ground close to the tunnel.

This principle is achieved practically by allowing the rock masses around the underground opening to deform in a controlled way. The rock support has therefore mainly a confining function to stabilize the rock masses that deform. As a

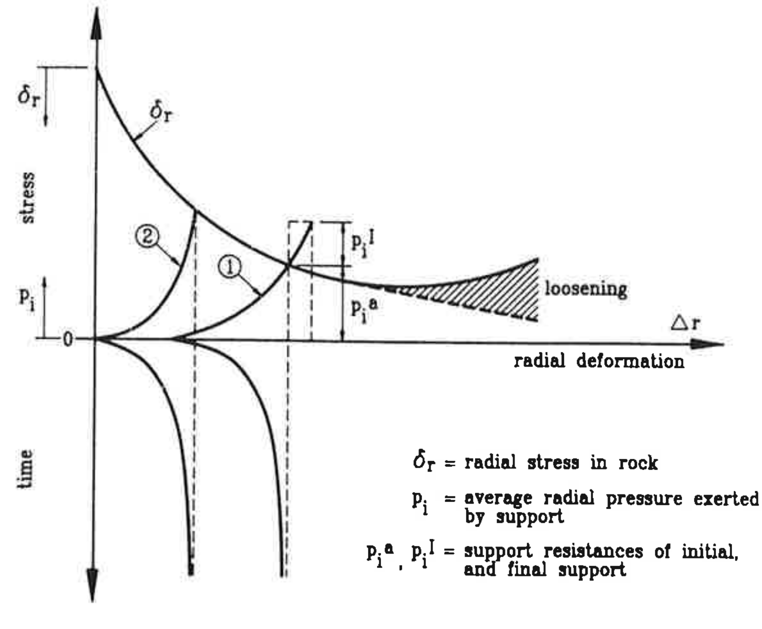

Figure 8.2 *An example of a ground support interaction diagram or Fenner-Pacher curve (after Brown, 1981). The numbers 1 and 2 show two different support systems installed at different times. The importance of stiffness and timing of the support is described in the text.*

consequence, the support must have suitable load-deformation characteristics and be installed at the right time. This approach requires a knowledge of the inter-relationships between ground deformation and load as well as between support deformation and load.

For this purpose the Fenner-Pacher curve is often applied in the design of rock support. This ground response curve (Figure 8.2) shows the rock/support interaction and the deformations with time. It provides a tool to optimize rock support, to help determine a favourable time for installation and an appropriate stiffness. There are, however, several limitations connected with response curves caused by simplifications and problems in generating relevant input data on rock mass characteristics.

Rabcewicz (1965) stresses the importance of a *deformable* rock support which should be neither too stiff, nor too flexible. A stiff rock support will be carrying a larger load because the rock mass around the opening has not had the possibility to deform enough to bring the stress peak further into the surrounding rocks (see Figure 8.2). Conversely, if the support is too flexible, the deformation may become too large and unsafe conditions may arise. Generally, this requires a support system consisting of systematic rock bolting and shotcrete. Whatever support system is used, it is essential that it is placed and remains in intimate contact with the surrounding ground and deforms with it.

The ground response curve in Figure 8.2 indicates also that the *timing* of rock support installation is a further important factor for a favourable mobilization of the inherent strength of the rock mass. If the rock support is installed too early, a heavier support is required to carry the resulting rock mobilized. An installation made too late may cause deformations of the rock masses surrounding the tunnel resulting in loosening and failures. It is however difficult to predict the time factor and its variations during tunnelling, even for experienced rock mechanics and tunnelling engineers. The use of monitoring and stress measurements in the tunnel during construction (see later) is therefore an important characteristic of the NATM.

In Austrian tunnelling practice, the ground is described behaviourally and allocated a ground class, based on field observations. The qualitative ground description used is associated, rather inconsistently, with excavation techniques together with principles and timing of standard support requirements. Although there are guidelines in the qualitative NATM classification, the ground class is mainly determined from individual observations by the engineering geologist. Brosch (1986) does not know of any Austrian experience with the common international classification systems (i.e. the RMR and Q systems). This rather unsystematic use of geo-data is a drawback which limits communication between people involved in tunnelling and the further development of the NATM.

Rock support and excavation principles

As a part of the NATM 'the dual-lining support' (initial and final support) for tunnels was introduced. This is the concept of letting the rock masses surrounding the tunnel and the initial support deform before the final or permanent support is applied. The two stages of rock support have been described by Rabcewicz and Golser (1973) as:

(i) The initial support–often carried out as an outer lining designed to stabilize the

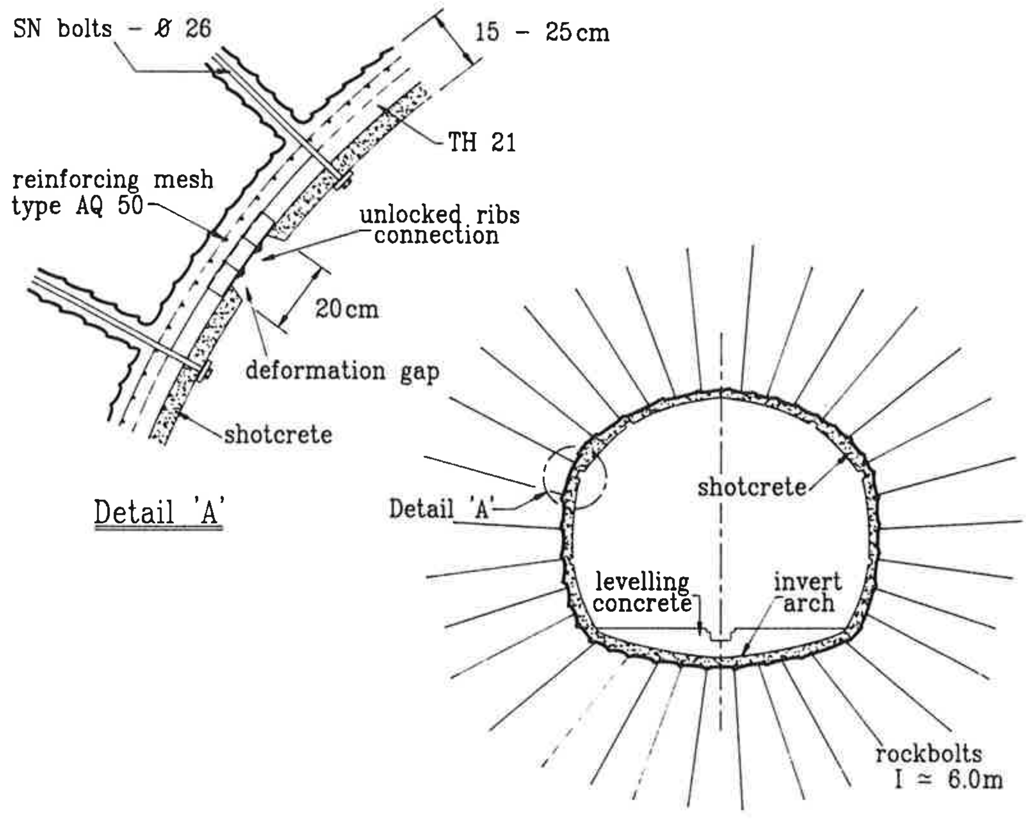

Figure 8.3 *Where large deformations take place in the tunnel walls it is important to have a highly flexible lining to interact with the deformations. The figure shows an example from the Tauern tunnel where the initial lining of shotcrete and rock bolts was designed with longitudinal joints to meet this requirement (adapted from Amberg and Cristini, 1986).*

rocks during excavation. It consists mainly of shotcrete, systematically bolted and reinforced by additional steel ribs if required. In addition, a closing of the invert is carried out in very weak ground; and

(ii) *The final support*–often carried out as a concrete lining. It is generally not installed until the deformations of the initial support have reached an acceptable, decreasing trend.

The initial support can partly or completely represent the total support required. The final lining may be necessary for structural reasons:

(i) when the initial lining is stressed beyond its elastic limit, or

(ii) when squeezing or swelling from time-dependent loads will exceed the bearing capacity of the initial lining.

A second lining may also be required for waterproofing. The dimensioning of the final support is made from assessments based on results from monitoring of stresses in the initial support element and/or of deformations of the tunnel surface and the ground surrounding the tunnel. In strongly squeezing ground the support should be designed to absorb large movements, as shown in Figure 8.3.

The practical execution of NATM involves the close coordination of the tunnel excavation technique and rock supporting works, and these two operations must be planned and designed according to the ground conditions. In very weak ground it

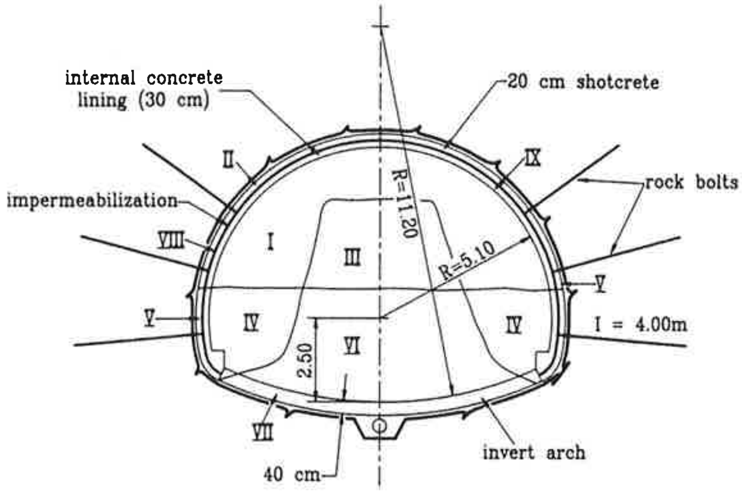

Figure 8.4 *A sequential tunnel excavation from a highway tunnel close to Florence, excavated in clayey schists (adapted from Amberg and Cristini, 1986)*

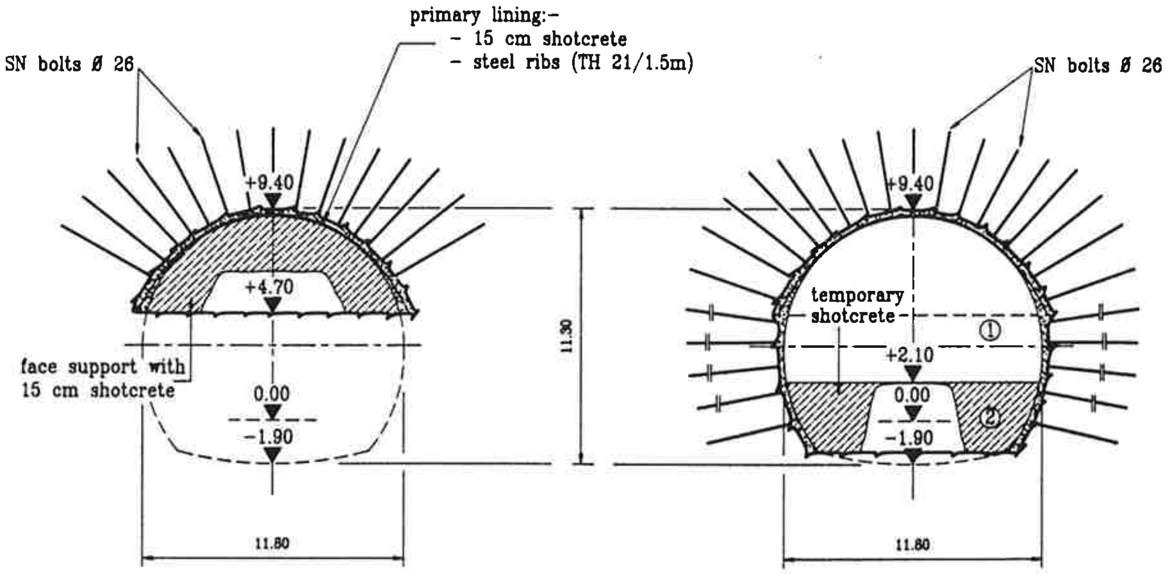

Figure 8.5 *Excavation and initial (primary) support from the Tauern road tunnel in a part consisting of strongly foliated chloritic phyllite alternating with quartz-rich phyllite (from Amberg and Cristini, 1986)*

might be necessary for reasons of stability to excavate smaller parts of the tunnel cross section, for example an upper heading and benching or alternatively the arch, the core and the invert arch of the tunnel section may be appropriate solutions (see Figures 8.4 and 8.5).

The excavation and support sequences (Figure 8.4) are as follows:
(i) Excavation of arch (I) leaving a central core in order to shore the walls and the arch. Immediately after excavation, execution of a first layer of reinforced shotcrete, placement of light ribs and a further increase of the shotcrete thickness until final value (II);
(ii) Excavation of central core (III) and side trenches by stage (IV), prestrengthening with a mesh reinforced shotcrete lining and steel ribs (V);
(iii) Excavation of the trench (VI), execution of the invert arch (VII); and
(iv) Waterproofong membrane (VIII) and final concrete lining (IX).

Monitoring of deformation and stresses

In order to investigate the behaviour of the ground during and after excavation, correct application of NATM is based on systematic in-situ measurements primarily of deformations and stresses (see Figure 8.6). From the progress of the deformations, it is possible to recognize early enough if an unacceptable trend appears and to act accordingly. Thus, monitoring of tunnelling in weak ground is not research, but an essential means of knowing whether the tunnel construction is proceeding satisfactorily.

The instruments are installed in sections along the tunnel when the initial support is placed. In addition to hazard control, the information from the measurements is related to the characteristics of the ground and the size of the opening. When interpreted, it is possible to adapt the type and dimensions as well as the timing of rock support to the actual ground conditions encountered during the excavation. In this context, it is obviously very important to have an effective calculation model which permits a quick interpretation of the data, and the best possible representation of reality.

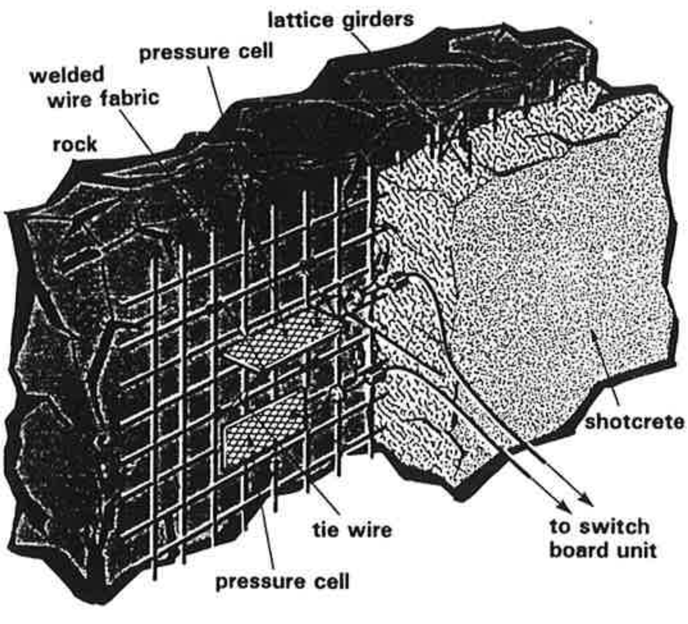

Figure 8.6 *Example of instrumentation of stresses in the shotcrete (adapted from Martin, 1984)*

Experience with the NATM

The NATM has been applied successfully in a large number of tunnels in many parts of the world, some of which were constructed in poor and difficult ground conditions. Compared to traditional tunnelling considerable cost savings have often been gained in addition to shorter construction times. NATM has, however, also experienced unpleasant downfalls (Wallis, 1987, 1988).

The versatility and adaptability of the method have been demonstrated from its basic principles and from the flexibility of using shotcrete and rock bolts both for initial and final support. A significant advantage of using shotcrete is the facility to adjust its thickness to the actual rock mass condition, i.e. by the application of further shotcrete layers.

Where NATM is used in conjunction with drill and blast or mechanical excavation, the flexibility of these methods can be fully utilized. Various sizes of tunnels and excavation sequences, for example for pilot headings and benching, can be made. The main benefit of NATM is for tunnels constructed in weak ground, i.e. materials that have a lower strength than the rock stresses they are exposed to. Tunnels excavated in stable and slightly loosening rock will generally benefit little from application of the NATM.

8.3 Norwegian Method of Tunnelling

Introduction

Although NATM has been used with good results in numerous soft ground tunnels, it may fall short of requirements and be too expensive when tunnelling in harder, jointed rock that require drilling and blasting. The reasons for choosing an alternative to NATM in such cases is that overbreak is more easily and effectively treated with fibre-reinforced shotcrete, *S(fr)*, than with mesh reinforced shotcrete, *S(mr)*. There is great wastage of shotcrete when voids (overbreak) have to be filled to cover the less flexible mesh reinforcement. In addition, jointed ground that causes overbreak can be effectively characterized so the required thickness of *S(fr)* and bolt spacing can be estimated accurately.

Following the development of wet process fibre-reinforced shotcrete in Norway at the end of the '70s, *S(mr)* was gradually replaced by *S(fr)*, and the less flexible mesh reinforced variety has virtually been eliminated over the last 15 years. The combination of systematic bolting, *B*, and wet process, robotically applied fibre-reinforced shotcrete provide the essential physical components of the Norwegian Method of Tunnelling (NMT) in jointed rock. The third component that puts NMT design on a more secure footing than rule-of-thumb estimating is the use of the Q-system of rock mass characterization (Barton *et al.*, 1974). This, in contrast to the descriptive rock mass classes used in NATM, provides a numerical rating of the rock mass and allows appropriate selection of *S(fr)* thickness (in cm) and bolt spacing (in metres) for tunnels or caverns ranging from about 2 to 100 metres in span. The selection of rock support for NMT is based on a very large data base (1 300 cases) which is synthesized in a recently updated Q-system support chart (Grimstad and Barton, 1993). Table 8.1 summarizes other essential features of NMT.

Table 8.1 *Essential features of NMT (after Barton et al., 1992)*

1) Areas of usual application:
Jointed rock giving overbreak; harder end of uniaxial strength scale
(σ_c = 3 to 300 MPa)
Clay bearing zones, stress slabbing
Q = 0.001 to 40 or more

2) Usual methods of excavation:
Drill and blast, hard rock TBM, machine excavation in clay zones

3) Temporary rock reinforcement and permanent tunnel support may be any of following:
CCA, *S(fr)+RRS+B*, *B+S(fr)*, *B+S*, *B*, *S(fr)*, *S*, *sb*, (NONE)
(see key below and Figure 8.7)
- temporary reinforcement forms part of permanent support
- mesh reinforced shotcrete not used
- dry process shotcrete not used
- steel sets or lattice girders not used; *RRS+S(fr)+B* are used in clay zones and in weak, squeezing rock masses. In worst cases *CCA* is used.
- contractor chooses temporary support
- owner/consultant chooses permanent support
- final concrete linings are less frequently used, *i.e.*, *B+S(fr)* is usually the final support

4) Rock mass characterisation for:
- predicting rock mass quality
- predicting support needs
- updating of both during tunnelling (monitoring in critical cases only)

5) The NMT gives low costs and
- rapid advance rates in drill and blast tunnels
- improved safety
- improved environment

CCA = cast concrete arches, S(fr) = steel fibre reinforced shotcrete, RRS = reinforced ribs, B = systematic bolting, S = shotcrete, sb = spot bolts, NONE = no support needed.

Q-System of rock mass characterization

(i) Rationale

The vast majority of the thousands of kilometres of tunnels constructed worldwide every year do not have the benefit of performance monitoring as often used in NATM. Design decisions are nevertheless required both before and during construction. No matter how many sophisticated rock mechanics test programmes and finite element analyses are performed, design engineers will come back to the basic question: "Is this bolt spacing, shotcrete thickness, or unsupported span width reasonable in the given rock mass?"

At present one has to rely on engineering judgement, or on classification methods, where the design is based on precedent, and where a good classification method will allow one to extrapolate past designs to different rock masses and to different sizes and types of excavation. Underground excavations can be supported with some confidence, primarily because many others have been supported before them and have performed satisfactorily.

(ii) Method for Estimating Rock Mass Quality (Q)

The six parameters chosen to describe the rock mass quality (Q) are combined in the following way:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{8.1}$$

where:
RQD = rock quality designation (Deere *et al.*, 1967),
J_n = joint set number,
J_r = joint roughness number (of least favourable discontinuity or joint set),
J_a = joint alteration number (of least favourable discontinuity or joint set),
J_w = joint water reduction factor, and
SRF = stress reduction factor.

The three pairs of ratios (RQD/J_n, J_r/J_a, J_w/SRF) represent block size, inter-block shear strength, and active stress, respectively. These are fundamental geotechnical parameters. It is important to observe that the values of J_r and J_a relate to the joint set or discontinuity most likely to allow failure to initiate. The important influence of orientation relative to the tunnel axis is implicit.

Detailed descriptions of the six parameters and their numerical ratings are given in Table 8.2. The range of possible Q-values (0.001 to 1000) encompasses the whole spectrum of rock mass qualities from heavy, squeezing ground up to sound, unjointed rock. The original case records examined by Barton *et al.*, (1974), include 13 igneous rock types, 26 metamorphic rock types, and 11 sedimentary rock types. More than 80 of the case records involved clay-bearing rock. However, most commonly the joints were unfilled and the joint walls were unaltered or only slightly altered.

The original case records from Scandinavian and international tunnel and cavern projects (numbering approximately 200), have since been supplemented by 1,050 Norwegian case records from main road tunnels. The new support selection chart shown in Figure 8.7 was presented by Grimstad and Barton (1993) and is a modification of the chart developed by Grimstad *et al.* (1986), when the original 1974 support recommendations were first updated to include fibre-reinforced shotcrete. It should be noted that the tunnel or cavern span or height given on the vertical axis in Figure 8.7 is divided by the term ESR. The numerical value of ESR appropriate to given types of excavation and their safety needs is shown in Table 8.3.

(iii) Tunnel Mapping Techniques

During the 20 years in which the Q-system has been utilized for tunnel support design, a variety of methods have been used for recording and presenting the rock mass characterization data. Two common methods are presented here of which the first (Figure 8.8) is a tunnel log on which principal joints and discontinuities are recorded, together with the relevant Q-system parameters. Recommended support

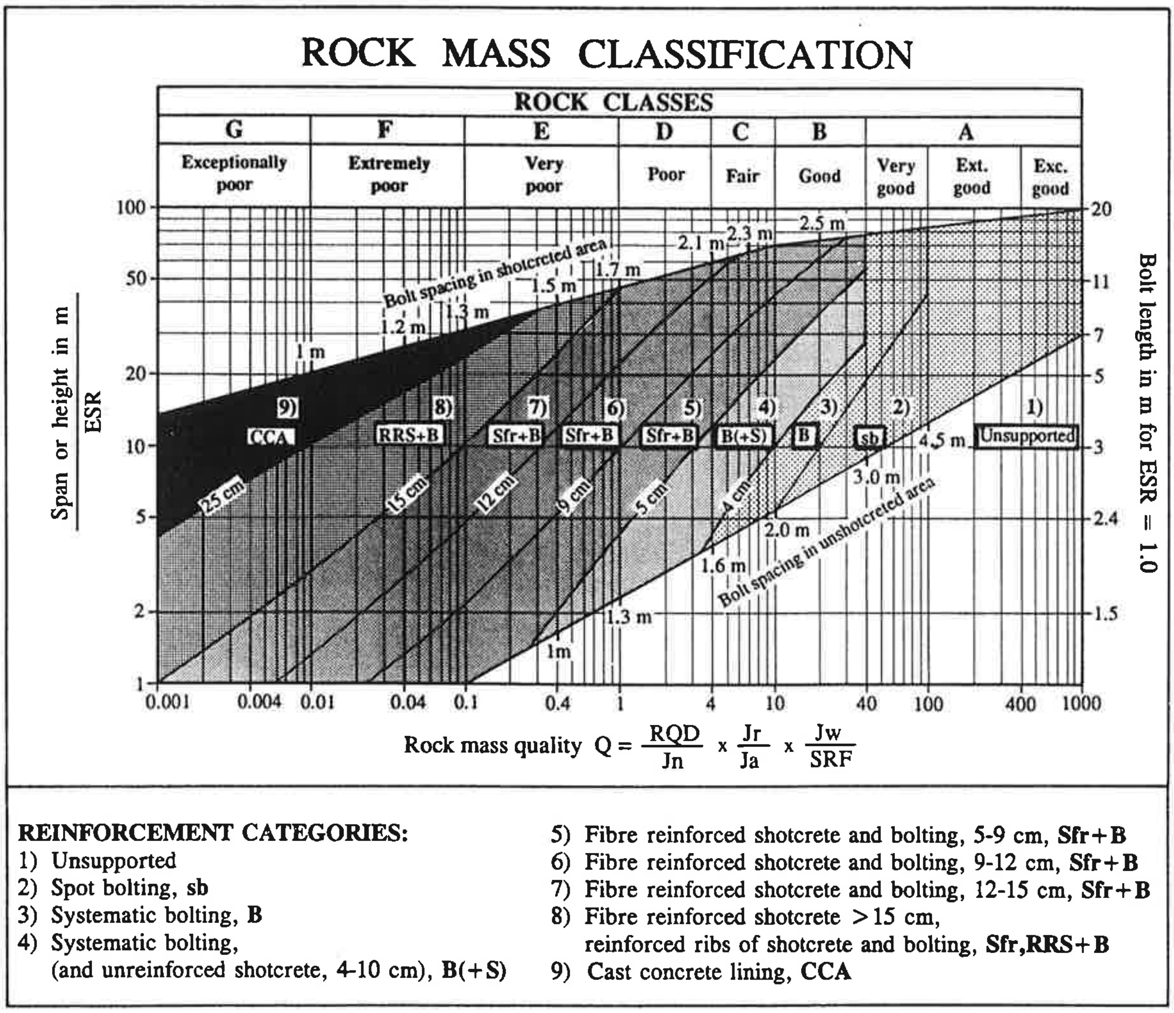

Figure 8.7 *Rock mass classification – permanent support recommendation based on Q and NMT (note extensive use of S(fr) as permanent support). (Grimstad and Barton, 1993).*

is selected, as shown in the right hand side of the log. The 1980 recommendation for support in the zone between chainage 810 and 817 consisting of *B* 1.5 m c/c and *S(mr)* of 15 cm thickness is close to the 1993 recommendation that would be obtained from application of Figure 8.7.

Type of tunnel:	headrace to hydroelectric power plant
span = 10 m	span/*ESR* = 10/1.6 = 6.25 m
height = 16.7 m	Q (of zone) = 0.1–0.3 (very poor)
Recommended support:	*B* 1.3–1.4 c/c, *S(fr)* 10–13 cm thickness

Based on the original 1974 Q-system, such zones would traditionally have been supported locally with cast concrete linings of about 30 cm thickness together with systematic bolting of about 1 m c/c. The flexibility of shotcrete, and especially that of robotically applied *S(fr)* is revolutionizing tunnel support methods, and increasingly replacing cast concrete.

Table 8.2 *The ratings of the six parameters for calculating rock mass quality, Q*

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 $\leq$ 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***	
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
ii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

Table 8.2 continued

	4. Joint Alteration Number	ϕ_r 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 <5 mm thickness)	16–24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5 mm thickness)	12–16°	8.0
J	Swelling-clay fillings, *i.e.*, montmorillonite (continuous, but <5 mm 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	water press. (kg/cm^2)	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.

Table 8.2 continued

	6. Stress Reduction Factor			*SRF*
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing ***clay*** or chemically disintegrated rock, very loose surrounding rock (any depth)			10
B	Single weakness zones containing ***clay*** or chemically disintegrated rock (depth of excavation $\leq$ 50 m)			5
C	Single weakness zones containing ***clay*** or chemically disintegrated rock (depth of excavation > 50 m)			2.5
D	Multiple shear zones in competent rock (*clay-free*), loose surrounding rock (any depth)			7.5
E	Single shear zones in competent rock (*clay-free*) (depth of excavation, $\leq$ 50 m)			5.0
F	Single shear zones in competent rock (*clay-free*) (depth of excavation, > 50 m)			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_c/σ_1	σ_θ/σ_c	*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	5–50
M	Slabbing and rock burst after a few minutes in ***massive*** rock	3–2	0.65–1	50–200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in ***massive*** rock	<2	>1	200–400
Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.75\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = 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_θ/σ_c	*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 $H > 350\ Q^{1/3}$ (Singh *et al.*, 1992). Rock mass compression strength can be estimated from $q \approx 0.7\gamma\ Q^{1/3}$ (MPa) where γ = rock density in kN/m^3 (Singh, 1993).				
d) Swelling rock: chemical swelling activity depending on presence of water				
R	Mild swelling rock pressure			5–10
S	Heavy swelling rock pressure			10–15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ (where $\tau \approx \sigma_n \tan^{-1}(J_r/J_a)$. Choose the most likely feature to allow failure to initiate.

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

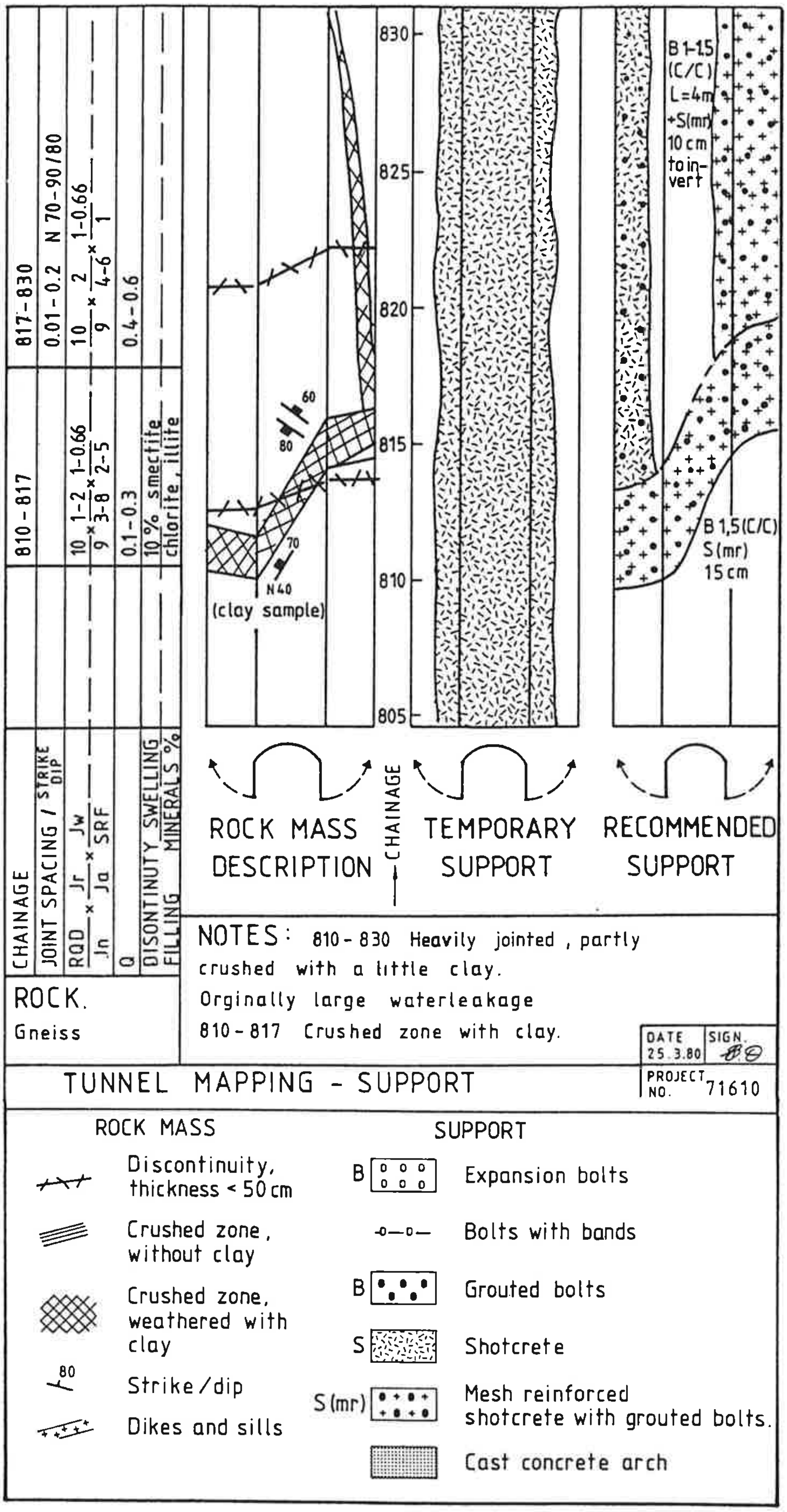

Figure 8.8 *A method of mapping tunnels for Q-system design of the rock reinforcement needs. Headrace tunnel, span 10 m, height 16.7 m (Barton et al., 1980).*

Table 8.3 *Excavation Support Ratio (ESR) for a variety of underground excavations*

	Type of Excavation	*ESR*
A	Temporary mine openings, *etc.*	*ca.* 3-5?
B	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings	1.6
C	Storage caverns, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, *etc.*	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portals, intersections	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	*ca.* 0.8?

An inescapable feature of the rock masses through which tunnels are driven, as compared to the shotcrete and concrete used for their support, is their huge variability. For this reason it may be difficult to specify just one value of Q for a given stretch of tunnel, for a length of drill core, or for a surface rock exposure. With this in mind, the statistical method of recording the Q-parameters has been developed, as shown in Figure 8.9. This figure is also a convenient reminder of the parameter ratings shown in Table 8.2. The recorded statistics for RQD, J_n, J_r, J_a, etc., for the logged location in the rock mass provide a more reliable basis for support selection than a single Q-value. A weighted average, and typical minima and maxima are usually estimated as a basis for the tunnel support decision. The example shown in Figure 8.9, taken from the mapping of the 10 m span top heading of the 62 m span Olympic Ice Hockey cavern at Gjøvik, shows variability and main quality trends equally well. Further examples of the use of this mapping method at Gjøvik are given by Bhasin *et al.* (1993).

Design of shotcrete and rock bolt support for marked weakness zones of limited thickness, with different side rock qualities, should be performed on an individual basis. A method developed by Løset is described by Grimstad and Barton (1993).

Tunnel Support Details

Application of sprayed concrete and rock bolts as tunnel reinforcement allows great flexibility in the amount of support since the sprayed concrete thickness, the spacing between rock bolts and the spacing and thickness of sprayed concrete ribs for poor ground can be varied readily to suit the rock conditions. Only in the most adverse conditions and exceptionally poor rock masses are cast concrete arches necessary (see Figure 8.7, Class 9).

(i) Spacing between rock bolts

It will be noted that the spacing between rock bolts is 20 to 40% greater when steel fibre reinforced sprayed concrete is utilized than when only rock bolts are used. The bridging effect of the sprayed concrete, particularly when fibre-reinforced, is obvious.

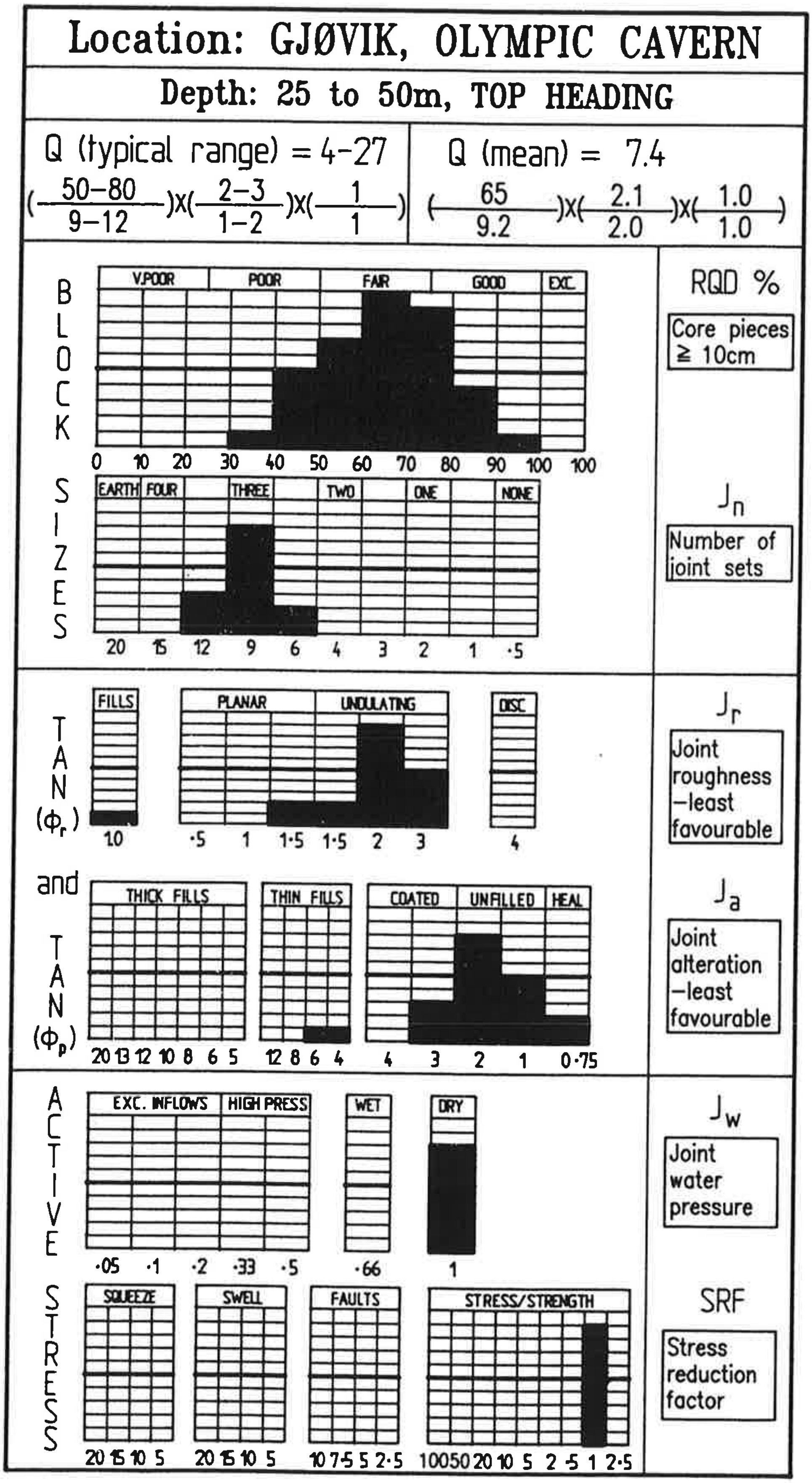

Figure 8.9 *Statistical recording of Q-system rock mass parameters (Barton, 1991)*

(ii) Length of rock bolts

The length of rock bolts used in normally-jointed rock mass, where systematic bolting is required, is given on the right side of the diagram in Figure 8.7. The length given is based on the equation:

$$L = 1.4 + 0.184 \times B \tag{8.2}$$

where L = bolt length, B = the span of the tunnel and ESR = 1. (This is a modification of L = 2.0 + 0.15 B/ESR in the original Q-system.)

In cases with steep joints running parallel to the tunnel, or when deep wedges are likely to be released, the length of the bolts often has to be longer than given by this equation, and would be chosen specially to suit the particular case to ensure sufficiently deep anchorage.

(iii) Thickness of sprayed concrete

The reinforcement Class 4 (Figure 8.7) consists of rock bolts and *unreinforced* sprayed concrete when the block size is limited (RQD/J_n < 10). The typical thickness of S will be 4 to 6 cm in smaller tunnels. However, in large excavations for power houses with significant wall height, it is customary to use up to 10 cm thickness, even when the rock quality Q is as high as 30. The reinforcement Classes 5, 6 and 7 consist of $S(fr)$ varying in thickness from 5 to 15 cm, combined with systematic rock bolts. The bolt spacings given on the upper diagonal will apply in these cases. In these classes of rock mass involving significant deformation, the advantage of selecting the appropriate toughness index for the $S(fr)$ to suit the problem needs emphasis. The same applies to the next class of support. Thin layers of $S(fr)$ are more flexible than thick layers, which act more like cast concrete. Hence the rock should be allowed to deform before spraying the final layers of $S(fr)$.

(iv) Reinforced ribs of sprayed concrete (RRS)

The reinforced ribs of sprayed concrete shown as Class 8 reinforcement in Figure 8.7 will be necessary when the usual thickness of $S(fr)$ is insufficient for bearing the load, or if the shape of the blasted opening is very irregular and a more circular shape has to be built up in order to support the rock. RRS is an extremely flexible method in which the thickness, width and spacing of the steel bar reinforced ribs can be varied according to needs.

The use of spiling (inclined bolting) ahead of the tunnel face, and monitoring of closure will generally be an advantage in these extremely poor quality rock masses which typically have Q-values in the range 0.001 to 0.1. The contrast in ground control when using RRS instead of regular shaped steel sets and blocking is fairly clear, and the total amount of concrete (and time used) is potentially greatly reduced. The traditional use of steel sets and blocking is also known to cause an increase in the effective SRF value of the rock mass due to unnecessary loosening. Final loads on the support may be increased as a result. This type of rock mass deterioration can be minimized by use of RRS.

(v) Cast concrete arches (CCA)

In exceptionally poor rock (swelling or squeezing conditions) and in larger excavations, it will be necessary to use multiple drifting, spiling, pre-injection and drainage measures, and supplement the temporary RRS (or its equivalent) with full profile cast concrete arches using steel shuttering. Depending on the amount of overbreak that has occurred prior to placement of the temporary $B+S(fr)$ or RRS, the CCA thickness is likely to vary from an average 30 cm to 1 m or more locally. A stiff invert, preferable with a convex form, will be essential in this type of squeezing or swelling ground. Analysis of the stability, and monitoring of deformations in the area temporarily supported by $B+S(fr)$ or RRS (before placement of the cast lining) is essential.

Support pressure estimation

The poorest rock mass qualities almost always cause appreciable deformation of the tunnel periphery. In these cases, it is important to provide temporary support which is flexible, but strong enough to increase stand-up time and prevent collapse, while allowing the rock mass to gain a new stress distribution. The final support can be installed based on observations. In Norway, the temporary support is almost always a part of the final support.

Selection of *type* of support, and general *thickness* of support was given in the Q-NMT design chart (Figure 8.7). Some guidance concerning maximum likely rock pressures acting on the support is provided by the following empirical equation which fitted the available case records in 1974:

$$P_{arch} = \left(\frac{20}{J_r}\right) Q^{-\frac{1}{3}} \tag{8.3}$$

where P_{arch} is the support pressure in tons/m^2.

As an example, when $Q = 1$, and J_r (joint roughness number) is equal to 1.5, the typically designed support capacity will be 13 tons/m^2. A recommendation for a given bolt spacing in Figure 8.7 can therefore be converted to select the appropriate working load for the bolts. The choice of bolt diameters is frequently 20, 25 or 32 mm, with corresponding *yield* loads of approximately 13, 20 and 32 tons respectively, when steel quality of 500 N/mm^2 is used.

In the case of thick RRS or CCA linings, where structural support is provided by the consistently positive (i.e., non-negative) radius of the concrete or shotcrete arch, then the theory of thin-walled cylinders can be applied to help check the required thickness of concrete, assuming only compressive loading. An appropriate working stress for the concrete is needed. A more accurate solution can be obtained with numerical modelling.

8.4 Conclusion

The NATM support design philosophy (Rabcewicz, 1964/65) has been used on numerous occasions for weak ground tunnelling, in general with great success. NATM, which is most appropriate for weak ground, is based on a descriptive behaviouristic ground classification (often involving about 6 classes), appropriate selection of temporary support based on these ground classes, monitoring of deformation, and application of additional support where required from the measured deformations. The soundness of an active design approach, sometimes called design-as-you-go (or more correctly, design-as-you-monitor), has been demonstrated by major cost savings compared to conventional, inflexible design approaches. However, it would be unfair to the NATM concept, and incorrect, to refer to all tunnels that have shotcrete and rock bolting as being "driven by NATM", as appears to be occurring in some quarters.

A newer concept, the NMT (Norwegian Method of Tunnelling), which is most appropriate for drill-and-blast tunnels in jointed rock which tends to overbreak, is frequently based on a quantitative (numerical) rock mass classification such as the Q-system (Barton *et al.*, 1974), appropriate use of temporary reinforcement such as rock bolting and fibre reinforced shotcrete, and supplementary reinforcement and support according to the engineering geologist's Q-based permanent support design. The tunnel span and the purpose of the excavation also figure in this selection of final support.

Despite the success of NATM in many weak ground tunnels, it clearly cannot be the best or cheapest method for tunnels in extensively jointed, harder rock masses that are drilled and blasted (as opposed to machine excavated). Extensive overbreak frequently causes mesh-reinforced shotcrete and lattice girders to be impractical, time consuming, possibly unsafe and to invite corrosion. Such methods may also cause unnecessarily large use of concrete. For this reason, Norwegian tunnellers were only too ready to stop using mesh reinforcement and steel ribs within a few years of developing the wet process, steel-fibre reinforced shotcrete method. Commercial application of wet process fibre-reinforced shotcrete in Norway by 1978 (Opsahl, 1982) gradually caused mesh-reinforced shotcrete to fall out of use by about 1984. Use of this revolutionary initial reinforcement and final support method since 1978 has increased to a level of 60,000 to 70,000 m^3 per year in Norway, close to the highest use in the world at present, despite Norway's small population. Robotic application 10 to 20 metres above, to the side of, or in front of the operator, production rates of 10 to 25 m^3/hour, low dust levels (rebound 5 to 10%), secured rock bolting conditions in unstable ground, and no problems with uneven profiles and overbreak, have caused a revolution in driving rates and tunnelling costs.

Cast concrete lined sections for permanent support of fault zones and clay-bearing rock are disappearing from use due to their cost and time constraints when compared to fibre-reinforced shotcrete. Rib (rebar) reinforced shotcrete with shotcrete and bolting are now used as permanent support in such zones at approximately half the cost of cast concrete. Similar advantages can be expected when such types of rock support are used as permanent support in tunnels or caverns in soft jointed rocks and in over-consolidated fissured clays, such as London clay.

In the past, attempts to combine the Q-system with NATM have been reported. Certainly, a more quantitative description of the six or seven NATM rock classes, using the Q-system, or using the RMR method of Bieniawski (1989), is inherently attractive. An interesting combination of NMT and NATM principles has recently been proposed for a major tunnel in partly soft and partly hard rock. Up-front prediction of support needs using the Q-system, temporary support close to the face, monitoring of resulting performance, and adjustment of support class (if necessary) for the application of final support well behind the advancing face appears to be an ideal combination of three well tried techniques, namely Q, NMT and NATM.

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