A comparison of Tunnelling design, concepts and methods

Sid Patel

Preface

The work presented in this report has been jointly done at NCC Construction Sverige AB and the division of geotechnical and mining engineering at Luleå University of technology. Financial support has been provided by Vinnova, SBUF, NCC and Luleå University of technology.

This report comprises a literature study, where the intension of the work has been to describe and compare different tunnelling methods and methodologies. What is the major difference between methodologies and its consequences when used in hard rock conditions?

This project was originally started at a time when NCC still was working internationally in civil engineering and infra-structure projects. Several of these projects included underground works. Even though these projects was to be carried out in hard crystalline rocks, it was most often referred, in the bidding document, to that the work should be designed and performed according to the NATM method. The NATM (New Austrian Tunnelling Method) is originally developed for tunnelling in soft ground, and does not necessarily be the perfect option to be used to its full extent when tunnelling in hard rock condition. Instead, methods or methodology often used in Scandinavia may be a better and more efficient option.

Summary

This project was originally started at a time when NCC still was working internationally in civil engineering and infra-structure projects. Several of these projects included underground works. Even though these projects was to be carried out in hard crystalline rocks, it was most often referred, in the bidding document, to that the work should be designed and performed according to the NATM method. The NATM (New Austrian Tunnelling Method) is originally developed for tunnelling in soft ground, and does not necessarily be the perfect option to be used to its full extent when tunnelling in hard rock condition. Instead, other methods or methodologies may be a better and more efficient option.

For tunnelling works today three main design concepts, or methods, are commonly in use. These are the observational method, the New Austrian method of tunnelling (NATM) and the Norwegian tunnelling method (NTM). Originally these methods have been designed to formalise quite specific tasks. In the case of the observational method, a framework for handling unforeseen events is provided with a flexible approach. The NATM is originally designed for handling severe rock problems, such as squeezing or general tunnelling in soft ground conditions. NTM is to a large degree based on the NGI Q-index for rock mass classification and support recommendations and have originally been developed using hard rock cases.

As the design methods have gained acceptance, they have also been developed, extended and also incorporated in standards. The Austrian ÖNORM B2203 has incorporated NATM methods and in the Eurocode 7, the observational method has been included as an option to be used for geotechnical works. The Q-index used in NMT has been extensively developed with many more cases and a separate classification for TBM tunnels, called Q_{TBM}, for predicting performance. These methods are presented and discussed in this report.

For cases where any of the above design methods are not well suited, for example in extremely poor rock or highly stressed rock, other methods must be used, usually meaning that good engineering judgement, investigations and calculations are deployed. For extremely poor rock the ground reaction curve is a useful concept for determining deformations and support pressures of the tunnel. Numerical modelling, properly used, is another tool that can be used in the design process. For this purpose the geological strength index (GSI) is especially useful in providing input data for the model. In special cases like those mentioned above, the observational method is a good choice since it offers flexibility and risk management.

For the last decades in Scandinavia, NMT has been the method of choice for most tunnelling projects, using the Q-index for rock mass classification and the support methods of the NMT. In recent years however, there has been criticism regarding the use of the NMT/Q-index classification and the Swedish Transport Administration requires that the estimation of reinforcement based on classification methods in hard rock conditions has to be verified by block analyses. At the same time there have been several major tunnelling projects that have been considerably costlier compared to the original budget, which has put focus on tunnelling methods and practices. For these reasons this report has been put forward with the aim of evaluating the usability of some common tunnelling methods/concept for the Scandinavian environment.

List of symbols and abbreviations

c	cohesion
H	overburden height
ITA	International Tunnelling Association
NATM	New Austrian Method of Tunnelling
NMT	Norwegian Method of Tunnelling
p_0	lithostatic pressure = γH
p_i	skin resistance, bearing capacity
r	radius of cavity
R	radius of protective zone
γ	density of the rockmass
Φ	internal friction

List of figures

List of tables

Table of contents

1 Historical background

1.1 Industrial revolution

Tunnelling and mining has been carried out by mankind since its earliest day. In Europe there is evidence of flint mining that dates back something like 15 000 years [45].

With the industrial revolution, starting in the late 18th century, a major shift of technological, socioeconomic and cultural conditions took place. The demand for metals, energy and the expansion of trade required improved transportation systems like canals, roads and railways, which in turn had a great effect on mining and tunnelling activity.

Already in 1818 Brunel patented a circular tunnelling shield for use in soft ground under the Thames in London. Inside the shield there were a number of iron frames across the tunnel face in which there was room to work from. In front of the miners were wooden planks secured by screw jacks, holding back the soil in front. To move forward, each plank was removed, the soil beneath it was excavated and the plank was jacked against the new surface. When the whole face had been excavated in this way, the tunnel shield was pushed forward using jacks supported against the brick lining behind. Iron plates were then placed in the new space and bolted together.

Figure 1-1 Brunel's iron frames used inside the tunnelling shield [51].

Until the 1830's a great number of canals were built in Europe. Occasionally these would be driven in tunnels. The longest such tunnel in Britain is the Stanedge tunnel, outside the city of Manchester. It is 4.8 km long and was built 1794-1811.

In Belgium in 1828 the Charleroi canal was built in soft ground. A part of the canal was decided to driven in a tunnel through a hill with quicksand. The methods developed and used became known as the "Belgian method" and was frequently used the following century in tunnelling. In the Belgian method the cross section is divided into several drifts at different levels. After having excavated the top heading, an arch is constructed. Under the protection from this arch, the lower drifts are advanced and the sidewalls bricked, thus supporting the top heading arch [45] [47]. The obvious advantage of Belgian method is that the roof is secured first which is desirable when working in soft ground. On the other hand, the underpinning of the arch may buckle under heavy rock loads, causing a collapse of the whole masonry arch.

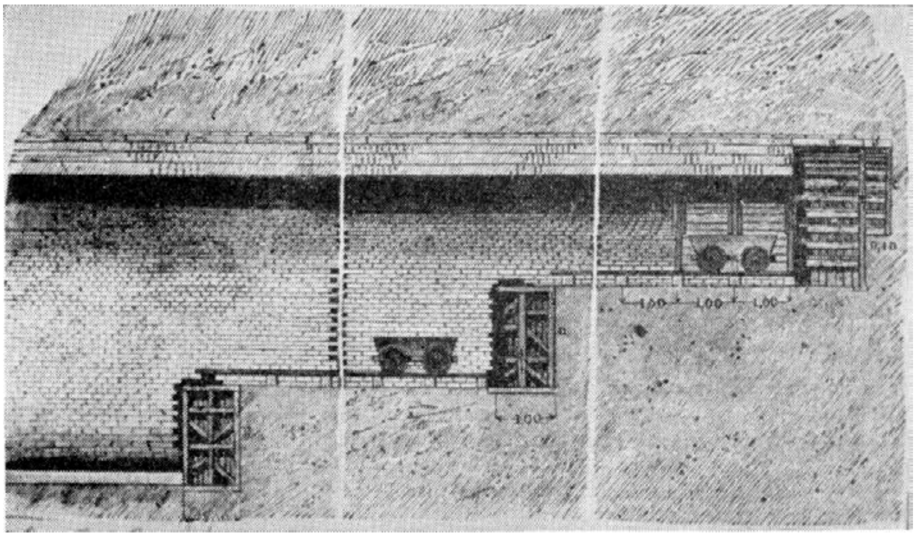


Figure 1-2 The Belgian tunnelling method in soft ground [45].

Other tunnelling methods that developed at this time were the English, German, Austrian and the Italian Cristina system. It is maybe appropriate to look a little closer at the “old” Austrian method since one of the more widely used tunnelling methods today is the “New Austrian tunnelling method”.

In the Austrian method the bottom heading is excavated first, either in shorter lengths or along the entire tunnel length and timbered. This is followed by the top heading, which extends from the bottom heading. From the top heading the roof arch is broken out to its full width top-down, followed by the slashing of the walls, all of which is timbered. The advantage of the bottom heading is that it will drain the surrounding area, thus facilitating subsequent excavation. It also serves as ventilation and aid alignment of the tunnel [45], [47].

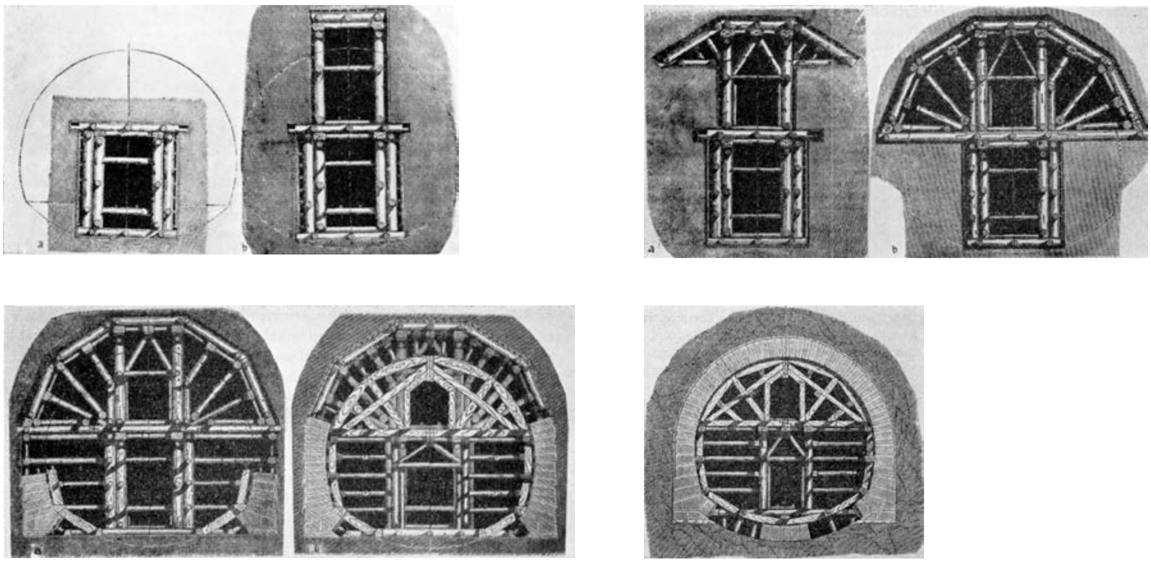


Figure 1-3 The “old” Austrian method of tunnelling [45].

1.2 Railway epoch

By the 1830’s the railway epoch had begun and canal building declined at its expense. The trains’ limited ability to climb steep slopes made tunnelling necessary when passing mountainous areas. This was especially evident in the Alps where some impressive railway tunnels were driven. The most well-known of these are listed in the table below. A map is provided in appendix Appendix 1.

Table 1.1 Great railway tunnels of the Alps

Name	Constructed	Length (km)
Fréjus	1857-1871	12.2
St. Gotthard	1872-1882	14.9
Arlberg	1880-1884	10.2
Simplon	1898-1906	19.8
Tauern	1901-1906	8.6
Lötschberg	1906-1912	14.6

All of the great Alp tunnels listed above, as most other contemporary tunnels, were built with timbering techniques which was the most widespread method until the 1950's [23]. In the middle of the 19th century with the arrival of rolling mills, steel support became more available and by the end of the century they began replacing timbering in difficult ground. A famous example of early steel support is the advancement through the infamous "pressure zone" in the Simplon tunnel. Seventy four 2.5 × 2.8 meter steel frames made of 14 inch girders and braced with heavy timbers were used to support a pilot tunnel for a length of 42 meters. From this pilot tunnel the rest of the tunnel profile was excavated.

In 1911 the American C E Akeley got a patent for a "cement gun" which was originally intended for use in taxidermy. The method of applying mortar by spraying is now known as "shotcreting". In 1914 the US Bureau of Mines began tests to replace timbering with shotcrete at the Bruceton experimental mine and by the 1920's shotcreting started being used in Europe as well. In 1950 the Swiss engineer Senn introduced a shotcreting machine with improved capacity and operational advantages over older equipment that marked a new era in shotcreting technology [23].

A patent for rock bolts was issued in 1918 by Stephan, Fröhlich and Klüpfel. From the patent specification: "*…Method for the support of roof and walls in mining without support from below. ...In order to achieve this goal, boreholes of sufficient depth will be drilled into the rock in wich rods, tubes or cables made of load bearing material, for example steel, will be inserted and fixed at the end in a proper manner or cemented along the whole length.*". In 1919 rock bolts are documented as being used in the Königshütte coalmines in nowadays southern Poland. They were used successfully to replace timbering. It would however not be until the 1940's in America and the 1950's in Europe that rock bolts were being more commonly used. The first large scale use of fully grouted rock bolts was at the Harsprånget power house in Sweden, 1952-1953, which was recognised as a major advance by Rabcewicz [24], one of the founders of the well known "New Austrian Tunnelling Method" (NATM).

2 Design methods for tunnelling

2.1 The observational method

The observational method was pioneered and formulated by Karl Terzaghi beginning in the late 1920's [16]. In geotechnical engineering, observations of the ground and its response during the construction stages are carried out and by evaluating the feedback, the designs can be updated accordingly.

Before the observational method, two methods could be used for handling uncertainties in a project according to Terzaghi [35]. The first method is to use high factors of safety and the other method is to design according to general experience. Terzaghi concludes that: *"the first method is wasteful; the second is dangerous"*.

By using an observational method, Terzaghi suggested a procedure that uses a base design from all available information, a detailed list of possible discrepancies between assumed and real conditions and calculations based on the original assumptions of various quantities that can be measured or observed in the field.

Terzaghi's co-worker Ralph Peck later formalised the principles of the observational method in an article at the Ninth Rankine Lecture [35]. The items listed below were presented to describe in short the parts that make a full observational application.

Elements of the observational method [35]:

a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.

b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions in this assessment geology often plays a major role.

c) Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.

d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.

e) Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.

f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.

g) Measurement of quantities to be observed and evaluation of actual conditions.

h) Modification of design to suit actual conditions.

Powderham [36] has suggested as a minimum level, that the observational method requires a base case design along with contingent designs. Thus follows that the observational method will require more design input compared to other methods.

2.1.1 Applications

Two general applications of the observational method are described by Peck [35]: the "best way out" and *ab initio* applications. The "best way out" application is used when construction has already started and some unexpected or unacceptable development occurs during construction. The *ab initio* applications are those where the observational method is used from the start and throughout the project. Peck remarked that "best way out" applications are much more common. The observational method is put to best use when executed as an *ab initio* application as it will naturally offer a great flexibility in planning and execution of the project.

Regardless of which application above is used, the aim of the observational method is to save costs, maintain safety and minimise the risks. This is done by evaluating feedback from actual conditions and applying necessary modifications to the design. In an article from 1998, Powderham [36] has suggested a basic approach of an observational application through progressive modification as follows:

1) Commence construction with a design providing an acceptable level of risk to all stakeholders.

2) Maintain or improve the acceptable level of safety.

3) Implement each change from a position of established safety through incremental steps of monitored and demonstrable acceptable performance.

This approach has been illustrated by Powderham in Figure 2-1. The vertical planes along the O-axis represent the current state of knowledge and on the other two axes cost vs. risk is shown. A project should ideally be located between point A and C at the start, giving it a potential for cost savings and managed risk handling by progressive modifications. This is illustrated by the paths along the lines towards point D_1. A “best way out” application is illustrated along line BD_2 with point D_2 representing the extra costs caused by corrective measures.

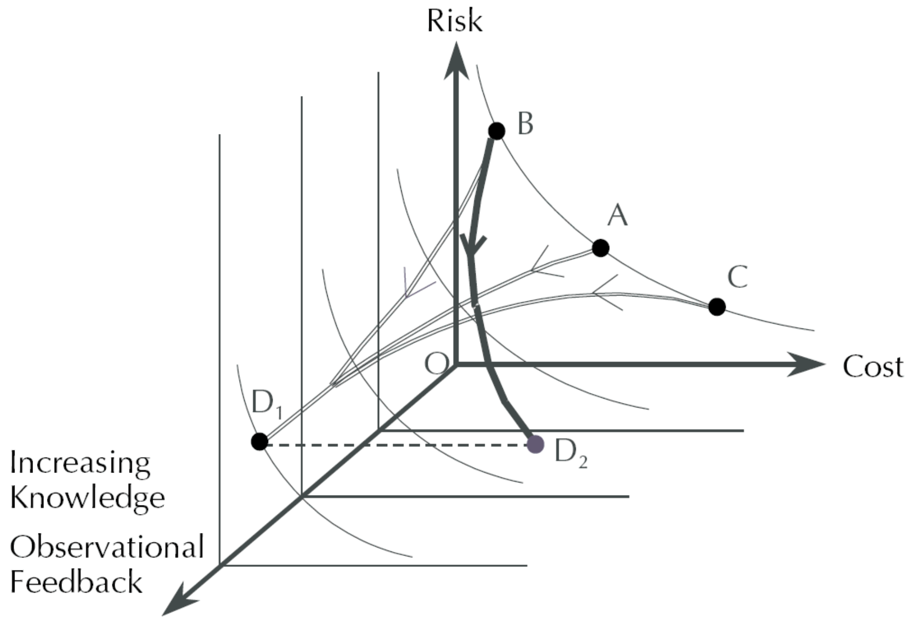


Figure 2-1 The observational method – knowledge, risk and cost [36].

2.1.2 Eurocode 7

For using the observational method, Eurocode 7 gives the following recommendations:

1) Because prediction of geotechnical behaviour is often difficult, it is sometimes appropriate to adopt the approach known as the observational method, in which the design is reviewed during construction. When this approach is used the following four requirements shall all be made before construction is started:

   a) The limits of behaviour which are acceptable shall be established.

   b) The range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits.

   c) A plan of monitoring shall be devised which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage; and with sufficiently short intervals to allow contingency actions to be undertaken successfully. The response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system.

d) A plan of contingency actions shall be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.

2) During construction the monitoring shall be carried out as planned and additional or replacement monitoring shall be undertaken if this becomes necessary. The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put in operation if this becomes necessary.

The term “acceptable probability” above can be interpreted that a more cautious and conservative approach is recommended compared to Peck’s “most probable conditions” (see chapter 2.1). In choosing a higher degree of probability, the base design will be more conservative. The need for calculations of values for the anticipated and most unfavourable conditions has not been included in the Eurocode 7 compared to Peck.

2.1.3 Adaption and Implementation of the observational method in Sweden

Holmberg & Stille have studied implementation of the Eurocode EN1997-1:2004 in underground projects from a Swedish perspective [20]. In Sweden the use of the observational method is also sometimes referred to (right or wrong) as “active design”.

Holmberg & Stille argues that the observational method fits well within current design practices and procedures and stresses the importance of robust and practical methods for analysis, supervision and visualisation of monitoring results. Analysis of data is done with the Bayesian statistical approach. Predictions take current knowledge into account and new data are combined with prior data. All of this which is consistent with the observational method according to the authors who further conclude that the observational method is close to the Bayesian view and that the method in principle is a formal way to use the experienced engineer’s way to work with complex designs.

The importance of classification as a tool is stressed for underground works. The classification is the basis for choosing measures such as rock reinforcement.

The main goal of the observational method is stated as a design that is economical with regards to the site specific conditions. There are similarities in the design process and work methods that are used today with the observational method. However the aim of the observational method should be to establish rock mass classes and measures and actions belonging to these. More stringent requirements are needed regarding transparency and traceability of data. These new demands require that contract documents are adapted for using the observational method. Holmberg, M., Stille, H. (2007). Observationsmetodens grunder och dess tillämpningar på design av konstruktioner i berg. SveBeFo rapport 80. ISSN 1104-1773.

2.1.4 ITA

ITA’s “workgroup on general approaches to the design” has recommended in their guidelines for tunnel design [20] that the following conditions are met when using an observational method:

1) The chosen tunnelling process must be adjustable along the tunnel line.

2) Owner and contractor must agree in advance on contractual arrangements that allow for modifications of the design on an ongoing basis during the project.

3) The field measurements should be interpreted on the basis of a suitable analytical concept relating measurement data to design criteria.

4) The interpretation of a particular instrumented section must be used to draw conclusions about the other sections of the tunnel. Hence, the experiences are restricted to those tunnel sections that are comparable with respect to ground conditions, ground cover, etc.

5) Field measurement should be provided throughout the entire length of the tunnel in order to check its assumed behaviour.

It is maybe not surprising that ITA has listed contractual arrangements as the second item. Tunnelling is by nature a risky business and there are many examples of projects where unforeseen problems have caused big delays and financial losses.

2.1.5 Summary

Clearly an observational method cannot be used if the design is not allowed to change during construction. The observational method requires great flexibility. This flexibility can however be a bit of two-pronged. For example, action needed that will slow down construction might be justified from an engineering viewpoint, but is undesirable from a contractual viewpoint. As Peck [35] wrote: *"The possibility of having to slow down construction is a drawback inherent in the method. It may cause financial loss and may even make the financing for the project difficult to arrange"*.

For an *ab initio* application, large discrepancies between the base design and actual conditions can be more of a problem than for a "best way out" application, since in a "best way out" scenario, cost and time penalties would have already been expected. For these reasons a base design on the conservative side would be more appropriate, allowing for greater flexibility in dealing with unfavourable development. As shown in Figure 2-1, the desired path of an observational *ab initio* application is that of base design which through modifications will reduce costs while maintaining an acceptable risk level.

For an observational method application for underground works, the most important elements can be summarised as:

- Assess the possible behaviours within the limits of acceptable probability and provide a reliable base design.
- Prepare contingency plans for all foreseeable deviations of actual conditions from anticipated.
- Continually evaluate feedback from actual conditions and make modifications accordingly.
- Make contractual arrangements so that an observational approach can be used.

2.2 NATM

2.2.1 Philosophy

In 1948 Rabcewicz patented (Österreichische patent 165573) the dual lining support, consisting of initial and final support. In the patent specification he also described tunnelling principles which was later included in the NATM [15], among them the concept of allowing the rock to deform before applying the final lining to reduce the loads on it [21].

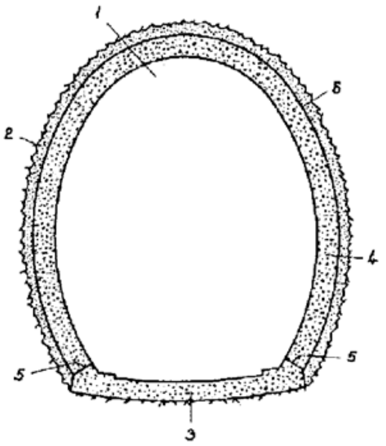


Figure 2-2 Dual lining in Rabcewicz's patent from 1948.

The “New Austrian Tunnelling Method” was first presented at a lecture in 1962 by Rabcewicz at the 13th Geomechanics Colloquium in Salzburg. Two years later the first article in English was published [37]. In this article the NATM was presented in the abstract as: *“...a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring, called an ‘auxiliary arch‘, the deformation of which is measured as a function of time until equilibrium is obtained”.*

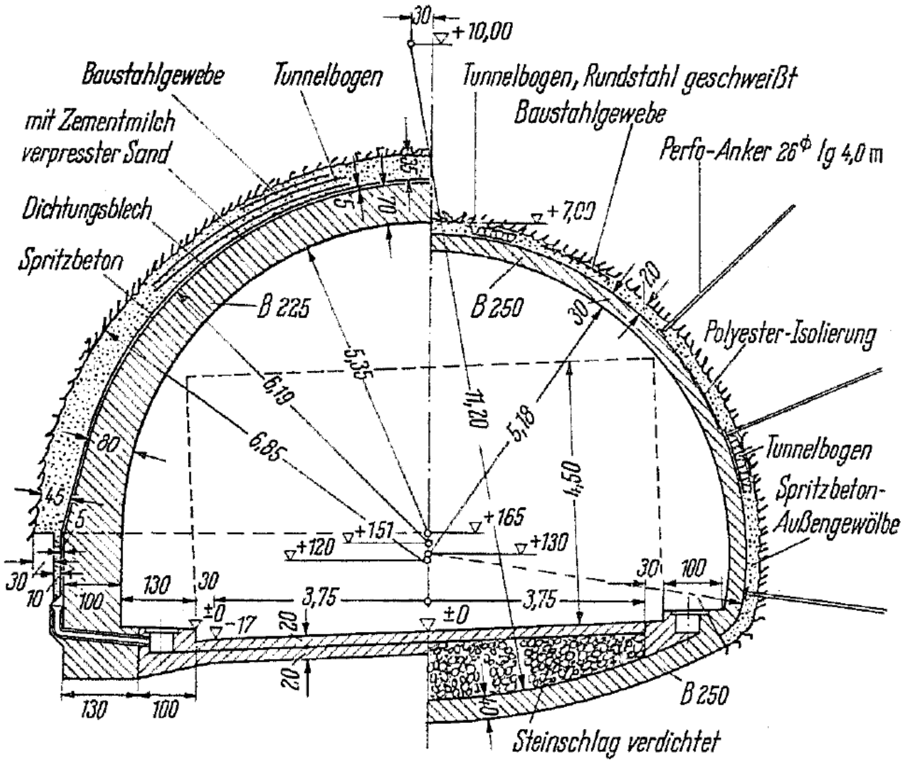


Figure 2-3 Comparison of a tunnel section at the Massenbergtunnel built using the old (left) and New Austrian Tunnelling Method (right) [39].

From this introduction, three main elements of the NATM are clear; the application of thin sprayed concrete lining, closure of the supporting ring and systematic measurements of deformations. Rabcewicz explains that the primary lining shall be installed immediately after excavation, and that the excavation shall be done in full face whenever possible so that a complete ring can be closed at the shortest possible distance from the excavation face. The effect of the primary shotcrete is to prevent disintegration of the rock and allowing it to mobilise its strength and to participate in the arch action. To minimise the lateral deformations, it is important to close the lining by an invert. The zone of arch action can be increased by adding rock bolts.

Rabcewicz also stresses the need for practical knowledge and close collaboration with the engineering geologist. The necessity of measurements is expressed in the final remarks of the article series: *"Evaluating forces by the measurements with respect to time is the very basis of the method and the sole means of economical design in accordance with the actual properties of rock"* [40].

2.2.2 Theory

The theoretical principles behind the NATM have been described by Rabcewicz [37] in terms of "stress rearrangements pressures". When excavating a tunnel, the stress redistribution will create a new equilibrium around the cavity. Depending on if the shear strength of the rock is exceeded or not, this will be attained with or without a lining. This is described as a process that is mechanical, progressive and generally occurs in three stages (see Figure 2-4).

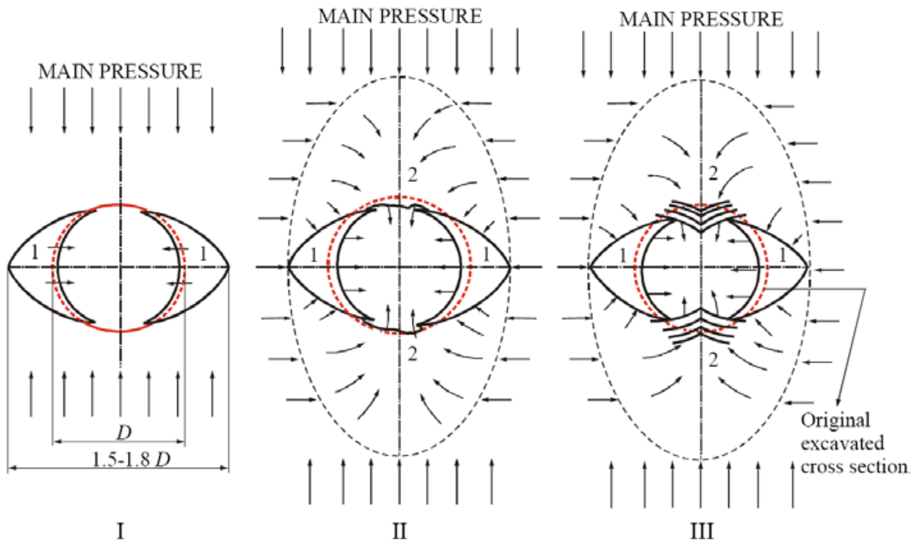


Figure 2-4 Mechanical process and sequence of failure around a cavity by stress arrangement pressure (after Rabcewicz [38], modified by Karakus & Fowell [21]).

At the first stage, wedge shaped blocks are formed by shearing along the Mohr surfaces and move in a direction vertical to the main pressure into the cavity (I). This inwards movement and subsequent increase in span will cause convergence of the roof and floor (II). At the last stage, as the movements increase and under continued pressure, the rock may buckle and protrude into the cavity, causing "squeezing" conditions (III).

During the process of stress rearrangement, a "protective zone" is formed around the cavity, also known as the "Trompeter zone", as the new equilibrium is being established. With the formation of the protective zone, surface stresses decreases as the surface deforms. The radial stress which has to be counteracted by the bearing capacity [of the lining], also called "skin resistance", at the periphery is reduced when the peak point of tangential stresses is moved away from the cavity. Simultaneously the radius of the cavity is being decreased due to the movements.

The relations between these stresses have been described mathematically by Fenner-Talobre and Kastner [40] as:

$$p_i = -c\cot\Phi + p_0\left[c\cot\Phi + (1-\sin\Phi)\right]\frac{r}{R}^{\frac{2\sin\Phi}{1-\sin\Phi}} \qquad \textit{Equation 2.1}$$

In a simplified form, where cohesion is omitted, the skin resistance or bearing capacity is expressed as [37]:

$$p_i = p_0(1-\sin\Phi)\frac{r}{R}^{\frac{2\sin\Phi}{1-\sin\Phi}} \qquad \textit{Equation 2.2}$$

Symbols: c = cohesion, p_i = skin resistance, p_0 = lithostatic pressure (γH), where H = overburden height and γ = density of the rockmass, r = radius of cavity, R = radius of protective zone, Φ = internal friction of the rock mass.

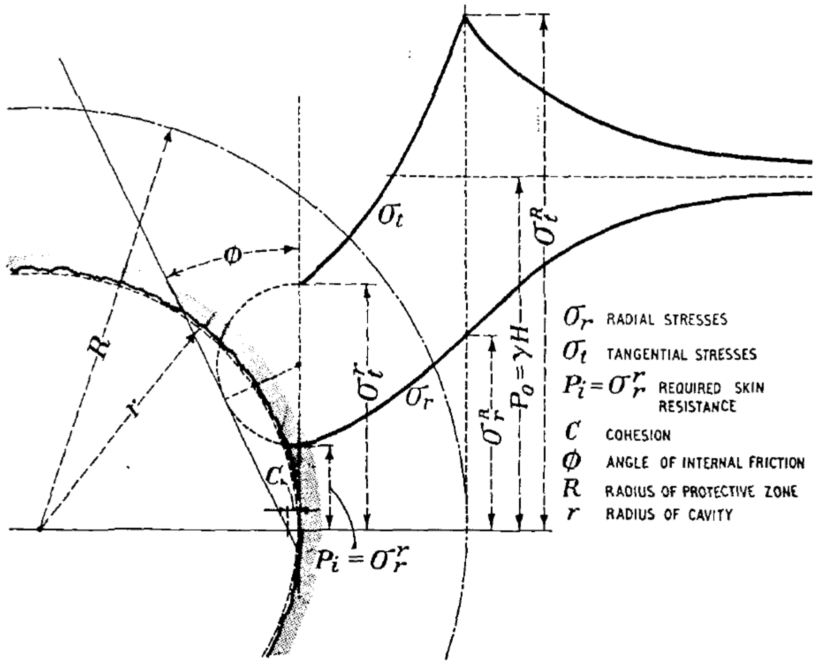


Figure 2-5 Schematic representation of stresses around a circular cavity (after Kastner) [37].

Rabcewicz's interpretation of this theoretical reasoning was that with a lining of low skin resistance, the protective zone increases, the skin zone loosens up and subsequently the internal friction Φ decreases. If the loosening becomes so great that open cracks and seams are formed, the skin zone will loose its bearing capacity almost entirely, the practical effect being that of a latent increase in span.

According to Rabcewicz [37] the decrease of internal friction of the rock mass (Φ) takes place almost directly after excavation, which causes loosening, whereas the development of the protective zone due to stress rearrangement is a slower process. As a consequence, lining of the newly excavated rock face needs to be done as soon as possible to avoid loosening. The lining has to be strong enough (skin resistance) to prevent further loosening and flexible enough to allow the formation of the protective zone. Rabcewicz's answer to these problems was to use shotcrete with additional rock bolts if required as the primary lining.

2.2.3 Summary

The main original elements of the NATM can from what has been described above be summarised as:

- Mobilisation of the rock mass strength

- Primary lining consisting of a thin shotcrete layer placed immediately after excavation.
- Closure of the lining ring at the shortest possible distance from the excavation front. For this reason, full-face excavation is preferred when possible.
- Measurements of deformations until equilibrium, evaluate the loads acting upon the lining.

2.3 ÖNORM B 2203

The Austrian code ÖNORM B 2203 (1983 edition) has been modified to comply with NATM philosophy where the adjustment to changing geomechanical situations is a fundamental principle. According to NATM philosophy the detailed tunnel design should take place during construction and therefore, the main steps of a design procedure are repeated in principle for each excavation round [27]. The design and classification principles are:

- Collection of geological, hydrological and geotechnical information. Classification of "rock mass character".
- Forecast of rock mass behaviour according to "rock mass type" descriptions (see Table 2.1).
- Excavation design according to "excavation classes".

A first classification step is to divide the tunnel into areas of similar geology and geotechnical character or "rock mass character", which is presented for each tunnel section. The predicted "rock mass type" distribution according to Table 2.1 is prepared for each section, preferably in a tabular format as percentages of section length.

Table 2.1 Rock mass types in ÖNORM B 2203

Main rock mass type			
A	**Stable to overbreaking**	Stresses acting on rock mass do not cause major failures.	
B	**Friable**	Disintegration due to structural weakness and/or lack of interlocking.	
C	**Squeezing**	Strength of rock mass is exceeded to great depth; this type also includes rock bursts and swelling rock.	
Rock mass types in detail			
Type		**Rock mass behaviour**	**Demands on excavation and support for conventional tunnel driving**
A1	**Stable**	Minor deformations that decline rapidly, no spalling.	No support required, unlimited round length.
A2	**Overbreaking**	Minor deformations that decline rapidly; some spalling at the crown due to discontinuities.	Support required in places; round length governed by overbreak.
B1	**Friable**	Minor deformations that decline rapidly; structural weakness and blasting operations lead to loosening and the separation of blocks in the crown and upper wall.	Small quantities of systematic support; reduced round length governed by stand-up length; possible support ahead of face.

Main rock mass type			
B2	**Very friable**	Deformations decline rapidly; poor structural strength, little interlocking, high mobility of rock mass and blasting operations lead to rapid and deep loosening where unsupported.	Systematic support except in invert; support of face; subdivision of cross section; systematic support ahead of face (forepoling); round length is dependent on reduced stand-up time and stand-up length.
B3	**Rolling**	Excavation even in small cross sections leads to inflow of rock material; lack of cohesion and interlocking are responsible for insufficient stability.	Support ahead of face (forepoling) and improvement of rock mass quality are required to allow advance in small cross sections; systematic support of all excavation surfaces.
C1	**Rock bursting**	Sudden release of energy leads to explosive rock failure.	Closely spaced short rock bolts; stress relief by drilling and relief blasting.
C2	**Squeezing**	Pronounced deformations that take long to decline; development of failure zones and plastic zones in plastic, cohesive rock mass.	Systematic support around the cross section; tunnel face is generally stable.
C3	**Heavily squeezing**	Large deformations, rapid at the beginning, taking long to decline; development of deep reaching failure zones and plastic zones.	Extensive support of all excavated surfaces; deformable support is generally necessary, round length is governed by the degree of stability of the face and deformation speed.
C4	**Flowing**	Very low cohesion, low friction, soft and plastic consistency of rock mass; material will flow into the tunnel even through very small unsupported areas.	Improvement of rock mass by advance support or special methods is necessary to allow excavation in small sections.
C5	**Swelling**	Rock mass with mineral content that increases in volume by absorbing water, *e.g.* swelling clay- minerals, salts, anhydrite.	Provision of supports capable of resisting the swelling pressure or of reserve space to allow volume increase due to swelling.

Classification for payment purposes is done by numerical evaluation of support costs and round lengths for the top heading and bench. For the invert a non-numerical excavation classification according to open length and construction type is used [1].

The key element is the definition of an excavation class matrix that is characterised by round length and support factor (*sf*) which are the two most important factors for the costs in cyclical conventional tunnelling (see Table 2.2 below).

Table 2.2 The excavation class matrix. Modified after Ayaydin [1].

Maximum round length (m)		**Support factor (*sf*)**								
		0.7	**1.2**	**2.0**	**3.0**	**4.5**	**6.8**	**10.0**	**15.0**	**23.0**
1	no limit									
2	4.00									
3	3.00									
4	2.220									
5	1.70				- 20%	**5/4.5**	+ 20%			
6	1.30									
7	1.00									
8	0.80									
9	0.60									
10	0.45									

The support factor reflects the cost of support measures and is calculated according to:

$$sf = \frac{\sum sq \times rf}{ar} \qquad \textit{Equation 2.3}$$

Where *sq* is the quantity of a supporting element per linear meter of tunnel, *ar* is the rating area and *rf* is the rating factor. The rating area (*ar*) is defined according to the subdivision of the cross section into top heading, bench, full face *etc*. as:

$$ar = \frac{C \times W}{4} \qquad \textit{Equation 2.4}$$

Where *C* is the circumference without the invert and *W* is the width of the cross section.

The rating factor (*rf*) is a dimensionless number that reflects the relative influence of different support types and quantities on tunnelling works. Rating factors have been calculated on data from completed tunnel projects (see Table 2.3 below). In order to avoid too many excavation classes, round length ranges are defined and the support factors are allowed to vary by ±20%. Other factors than round length and support works that will affect excavation progress are not accounted for in the excavation classes and have to be dealt with separately, for example water problems.

Table 2.3 Rating factors for support elements [1].

Support element		**Rating factor (*rf*)**	**Per unit**
Rock bolts	Swellex & expansion	1.0	m
	SN, mortar	1.5	m
	Self-drilling	2.0	m
	Grouted	2.5	m
	Pre-stressed, mortar	3.0	m

Support element		Rating factor (*rf*)	Per unit
Wire mesh	First layer	1.0	m^2
	Second layer	1.5	m^2
	Invert	0.5	m^2
Steel arch and load distribution beam		2.0	m
Shotcrete (theoretical quantity)		15.0	m^3
Deformation slots		4.0	m
Forepoling spiles	Non-mortar embedded	0.7	m
	Mortar embedded	1.0	m
	Self-drilling	1.5	m
	Grouted	2.0	m
	Spiles for grouting	3.0	m
Linear plates	Lagging	2.5	m^2
	Forepoling	4.0	m^2

During the excavation works the contracting partners shall agree on necessary rock works needed. These works are correlated to a corresponding excavation class which is directly related to the cost.

2.4 Norwegian Method of Tunnelling (NMT)

The Norwegian method of tunnelling, abbreviated NMT, has been described in article from 1992 by Barton *et al* [6]. Some of the major features of NMT are shown below. A more detailed description is presented in Appendix 4.

Essential features of NMT are (compiled after Barton *et al* [6]):

1) Areas of usual application:

   Jointed rock at the harder end of the scale (σ_c = 3-300 MPa), clay bearing zones, stress slabbing (Q = 0.001-10)

2) Usual methods of excavation:

   Drill and blast, hard rock TBM, hand excavation in clay zones

3) Temporary support and permanent support may be any of the following:

   Cast concrete arches; steel fibre reinforced concrete , reinforced ribs of shotcrete and systematic bolting; systematic bolting and shotcrete; systematic bolting; steel fibre reinforced concrete; shotcrete; spot bolting; no support

4) Rock mass characterisation is used for:

   Predicting rock mass quality; predicting support needs; updating of rock mass quality and support needs during tunnelling (monitoring only in critical cases)

The NMT is a Norwegian response to the NATM [32]. Although not explicitly expressed, the Q-system (see chapter 3.3) is an important part of the NMT as it has been adapted to NMT ideas and methods of rock supporting.

NMT puts great emphasis on thorough descriptions of geological and geotechnical aspects both before and during the tunnelling works. For these purposes a quantitative description of the rock mass, implying the Q-index, is considered a key requirement. An agreed rock mass documentation plays an important role in the tendering, tender evaluation and during the production phases of a tunnelling project.

Prediction of the rock mass quality along the tunnel alignment is used for listing the amount of rock support, tunnelling methods and pre-grouting works that is foreseen for the various parts. Support work is usually divided into primary support that is installed at the face and final support that is installed behind the face. Correlations between seismic velocities, deformation modulus, Lugeon values and the rock mass quality (Q) are suggested in the Q-system as an aid for predicting the rock mass quality (see chapter 3.3).

When rock problems are encountered, the owner and contractor shall agree on the most suitable and practical solution and compensation to the contractor is done according to the approved unit price list.

2.5 NATM vs. NMT

Barton and Grimstad [5] have explained the different applications of NMT and NATM with that NATM is most appropriate in soft ground that is machine excavated, while NMT is most appropriate for tunnels in jointed rock which tends to overbreak and that is mainly excavated with drill and blast techniques. Only in extremely poor rock masses with a recommended rock reinforcement category of 8 or 9 in the Q-system (see Figure 3-8), a possible overlap of the two systems might be present [5]. In the opinion of some authors however (*e.g.* Palmström & Broch [33]) the Q-system should not be used for such poor rock masses since it is not well suited to handle ground behaviour associated with these types of rock masses such as squeezing ground. NATM on the other hand is originally designed to handle extremely poor rock masses such as squeezing ground.

2.5.1 Survey

In a survey addressed to members of the Swedish society of engineering geology (Byggnadsgeologiska sällskapet), 34% of the respondents said they know the NMT “well” or “very well”, 37% say they know the method “somewhat” and 29% do not know the method at all. However all respondents know of the Q-method for rock mass classification at least “somewhat” and as much as 80% say they know of the Q-method “well or “very well”.

For NATM the corresponding figures are: 42% of the respondents say they know the NATM “well” or “very well” and 45% say they know the method “somewhat”. Only 12% do not know the method at all.

It seems that the NATM is better known among the respondents than the NMT. The relatively low recognition of the NMT is maybe a bit surprising considering that Norway is a neighbouring country to Sweden and the fact that all of the respondents at least know “somewhat” of the Q-system for rock mass classification, which is an important part of the NMT. This is probably due to the fact that the Q-system was developed before the introduction of the NMT and it has been widely used for rock mass classification in Sweden.

2.6 Active tunnel design

The “Active Tunnel Design” (ATD) concept is presented as a “new concept for pre-design, contractual lay-out and construction management” [49] by that have originated from Sweden. It is claimed to be based on the experience from more than 200 tunnels.

The concept of active tunnel design is based on defining rock classes for each tunnel type and geological setting during the design stage. Each class is related to a bid price, advance rate, rock support measures and other items needed for that particular class. During construction, mapping and tests of the excavated areas are carried out and the rock class is decided. Rock supporting and payment to the contractor is done in accordance to the actual rock class encountered. Hence, this is the most important part of the Active Tunnel Design concept – to objectively establish the actual rock class. The benefit claimed being that a complete geological and rock mechanical documentation and quantities for payment can be done in one operation. Depending on the actual mapped rock conditions, adjustments to the time plan might be decided upon.

Basically the Active Tunnel Design concept is built on systematic geological and rock mechanical mapping and subsequent rock mass classifications that form the basis for payment to the contractor and adjustments to the time plan.

3 Rock mass classification systems

Rock mass classification systems serve the purpose of characterising and classifying rock masses, usually for construction purposes such as tunnelling, mining, foundations etc. Any classification system will be more or less suited to a specific task depending on its intended use. It is therefore very important to know the limitations of the classification system being used for a certain task, which is the case with any other strategy or method for that matter.

The primary objective of any classification system is to quantify the properties of the rock mass based on past experience. Any classification system serves the general purposes of [29]:

a) grouping areas of similar geo-mechanical characteristics

b) providing guidelines for stability performance

c) selecting appropriate support

Although more than a hundred classification systems have been developed in the last decades [26], only a few are commonly used for rock engineering. The most common used are probably the RQD, RMR and Q-system, which are discussed in following chapters.

A summary of some main rock mass classification systems has been compiled by Palmström, see Table 3.1 below.

Table 3.1 Some main rock mass classification systems (modified after Palmström [31] and Palmström & Stille [34]).

Name of classification	**Form and type***					**Main applications**	**Reference**
	Descriptive	**Numerical**	**Behaviouristic**	**General**	**Functional**		
Lauffer's stand-up time classification	×			×		For input in tunnelling design	Lauffer, 1958
Rock classification for rock mechanical purposes	×			×		For input in rock mechanics	Patching & Coates, 1968
The rock quality designation (RQD)		×		×		Based on core logging; used in other classification systems	Deere et al., 1967
The Geological Strength Index (GSI)		×			×	For design of support in underground excavations	Hoek, 1994

Name of classification	Form and type*					Main applications	Reference
The Rock Mass index (RMi) system		×			×	For general characterisation, design of support, TBM progress	Palmström, 1995
The Terzaghi rock load classification system	×		×		×	For design of steel support in tunnels	Terzaghi, 1946
The new Austrian tunnelling method (NATM)	×		×			Tunnelling concept for excavation and design in incompetent (overstressed) ground	Rabcewicz, Müller & Pacher, 1958 - 64
The size-strength classification		×			×	Based on rock strength and block diameter; used mainly in mining	Franklin, 1975
The rock structure rating (RSR) classification		×			×	For design of (steel) support in tunnels	Wickham et al., 1972
The rock mass rating (RMR) classification		×			×	For use in tunnel, mine and foundation design	Bieniawski, 1973
The Q classification system		×			×	For design of support in underground excavations	Barton et al., 1974
The unified classification of soils and rocks	×			×		Based on particles and blocks for communication	Deere et al., 1969
The typological classification	×			×		For use in communication	Matula & Holzer, 1978
The unified rock classification system	×			×		For use in communication	Williamson, 1980
Basic geotechnical classification (BGD)	×			×		For general use	ISRM, 1981

*) Definition of the descriptions:

Descriptive form: the input to the system is mainly based on descriptions

Numerical form: the input parameters are given numerical ratings according to their character

Behaviouristic form: the input is based on the behaviour of the rock mass in a tunnel

General type: the system is worked out to serve as a general characterisation

Functional type: the system is structured for a special application (for example for rock support)

3.1 Rock Quality Designation

Rock quality designation index (RQD) was developed by Deere in 1964 [13] to provide quantitative estimates of rock mass quality from drill cores. RQD is defined as the percentage of intact core pieces longer than 10 cm of the total core length (see Equation 3.1 and Figure 3-1).

The core pieces are measured along the centreline or along a full circular barrel section. The recommendation is to use the centreline for measurements. Core breaks that are caused by handling or the drilling process are disregarded.

$$RQD = \frac{\sum Length\,of\;core\;pieces > 10\,cm\,length}{Total\;length\,of\;core\,run} \times 100\,[\%] \qquad \text{Equation 3.1}$$

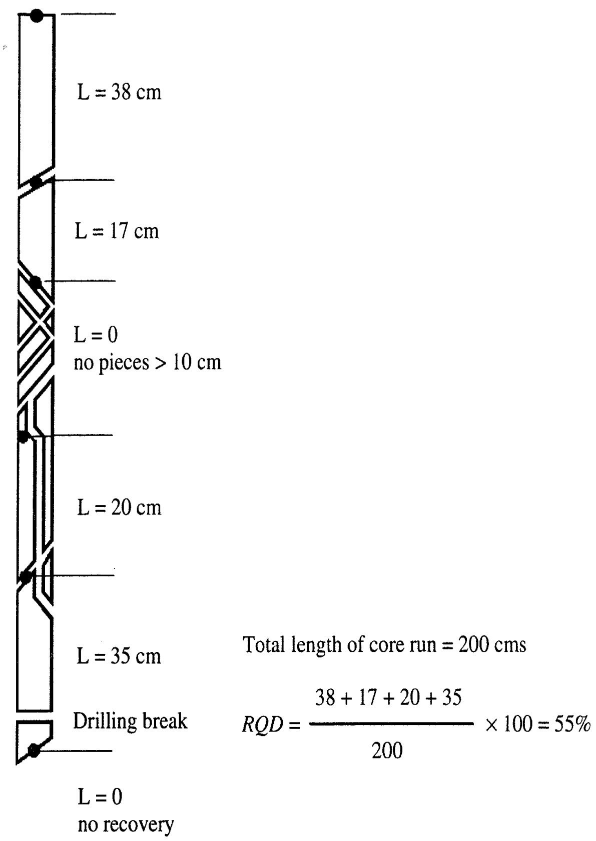


Figure 3-1 Procedure for measurement and calculation of RQD [19].

Originally RQD was based on cores of NX size (54.7 mm) obtained with double-tube core barrels, but according to Deere's experience [22] other core sizes and drilling techniques are also applicable, provided proper drilling and core handling is utilised. According to Deere, core sizes between BQ (36 mm) and PQ (85 mm) are applicable for RQD measurements, but NX and NQ (47.5 mm) core sizes are most optimal.

Various core lengths have been proposed for RQD measurements, but Deere suggests that a 10 cm length should be used at all times [22]. Since RQD is sensitive to the length of the core run, it is also recommended that the calculation of RQD is based on the actual drill-run lengths used in the field and preferably no longer than 1.5 m [22].

Palmström has suggested that RQD can be correlated to the volumetric joint count, which is the number of joints of all discontinuity sets along a unit length, for clay-free rock as [19]:

$$RQD = 115 - 3.3 \times J_v \qquad \textit{Equation 3.2}$$

Since RQD usually is logged directly after the core recovery, the great benefit is that it provides an early indication of the rock mass quality and as such allows engineers to compare the rock mass quality within a specific site for further design considerations. For large excavations the RQD is of questionable value as it is unlikely that all discontinuities logged in the core are of importance to the tunnel stability [13]. Deere has also pointed out this by stating that RQD should be used for finding areas of poor rock quality and as an aid in siting excavations in the best ground possible. Secondly it can be used for assessing tunnelling conditions and selecting initial support [22].

RQD is a standard procedure that is used in core logging, outcrop mapping etc. and useful as an early indication of the rock quality. However, its most important role is as a component of rock mass classification systems such as RMR and NGI's Q-system which are described in the following chapters.

3.1.1 Correlations of deformation modulus and RQD

Correlation between RQD and the rock mass deformation modulus based on field studies have been suggested by several authors. One of the most recent correlations has been made by Zhang & Einstein [52]. Data in the range of $0 \leq RQD \leq 100$ have been compiled from published literature. The rock types include mudstone, siltstone, sandstone, shale, dolerite, granite, limestone, greywacke, gneiss and granite gneiss. See Figure 3-2 below.

Zhang & Einstein recommend three relationships for the ratio of E_m/E_r and RQD, where E_m and E_r are the deformation moduli of the rock mass and the intact rock respectively:

Mean:

$$E_m / E_r = 10^{0.0186 \times RQD - 1.91} \qquad \textit{Equation 3.3}$$

Lower bound:

$$E_m / E_r = 0.2 \times 10^{0.0186 \times RQD - 1.91} \qquad \textit{Equation 3.4}$$

Upper bound:

$$E_m / E_r = 1.8 \times 10^{0.0186 \times RQD - 1.91} \qquad \textit{Equation 3.5}$$

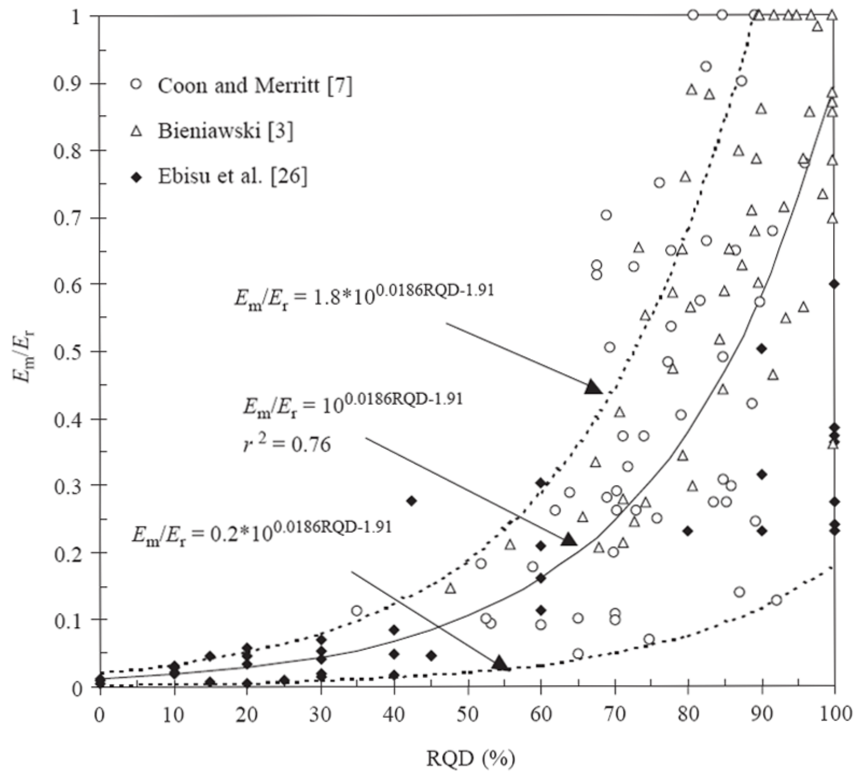


Figure 3-2 Recommended relationships between RQD and E_m/E_r by Zhang and Einstein [52].

According to Zhang and Einstein the large scatter of data may be caused by different test methods being used, directional effects due to anisotropy of the rock masses, discontinuity conditions such as aperture and filling material and the insensitivity of RQD to jointing frequency. Accordingly the recommended relationships above give indications of the mean and bounding values for a certain rock mass. It is mentioned by Zhang and Einstein that it is more reasonable to estimate the deformation modulus from RMR or Q-values (see chapter 3.1.1.1 below) since these in addition to RQD also consider other parameters that are of importance to the deformation modulus of the rock mass. However, in cases where only RQD-values are available, the estimation of Zhang and Einstein can be useful.

3.1.1.1 Correlation of deformation modulus to other rock mass classifications

The deformation modulus is an important parameter for the analysis of rock masses and since field tests are costly and time consuming, many authors have proposed empirical relationships between the deformation modulus of an isotropic rock mass and different classification systems values, such as RMR, Q-index and GSI [18]. Some of these are presented in the table below.

Table 3.2 Empirical estimations of rock mass modulus (modified after Hoek & Diederichs [18]).

Relationship with E_r	**Author(s) & year**	**Curve No.**
$E_r = 2 \times RMR - 100$	Bieniawski 1978	1
$E_r = 10^{((RMR-10)/40)}$	Serafim & Pereira 1983	2
$E_r = \dfrac{E_i}{100(0.0028 \times RMR^2 + 0.9 \times e^{(RMR/22.82)})}$; E_i = 50 GPa	Nicholson & Bieniawski 1990	3
$E_r = E_i \left(\dfrac{1}{2} \left(1 - \cos \left(\dfrac{\pi \times RMR}{100} \right) \right) \right)$; E_i = 50 GPa	Mitri *et al* 1994	4
$E_r = \dfrac{1}{10} \times \left(\dfrac{RMR}{10} \right)^3$	Read et al 1999	5
$E_r = 10 \times Q_c^{1/3}$ where $Q_c = Q \times \sigma_{ci} / 100$ and $\sigma_{ci} = 100\,\text{MPa}$	Barton 2002	6
$E_r = \left(1 - \dfrac{D}{2}\right) \sqrt{\dfrac{\sigma_{ci}}{100}} \times 10^{((RMR-10)/40)}$; $D = 0$; $\sigma_{ci} = 100\,\text{MPa}$	Hoek *et al* 2002	7
$E_r = E_i (s^a)^{0.4}$; $E_i = 50\,\text{GPa}$; $s = e^{((GSI-100)/9)}$; $a = \dfrac{1}{2} + \dfrac{1}{6}\left(e^{(-GSI/15)} - e^{(-20/3)}\right)$; $GSI = RMR$	Sonmez *et al* 2004	8
$E_r = E_i \times s^{1/4}$; $E_i = 50\,\text{GPa}$; $s = e^{((GSI-100)/9)}$	Carvalho 2004	9
$E_r = 7(\pm 3)\sqrt{10((RMR-44)/21)}$	Diederichs & Kaiser 1999	10

Hoek and Diederichs [18] do not recommend any of the estimations above in particular, but note that the estimations made by Mitri (4), Sonomez *et al* (8) and Carvalho (9) show poorer correlation to the full range of measurement data.

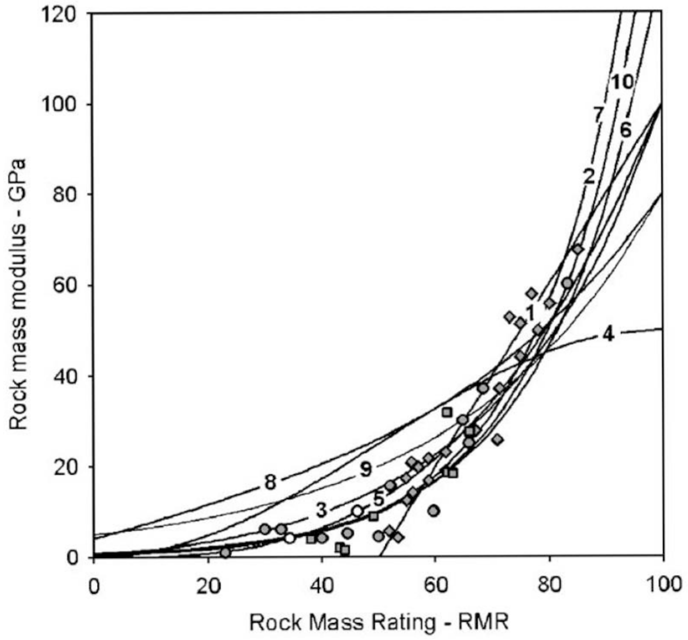


Figure 3-3 Empirical equations for predicting rock mass deformation modulus with data from in situ measurements [18]. Curve numbers correspond to those in Table 3.2.

3.2 RMR

The Rock Mass Rating (RMR) method, or Geomechanics Classification, was developed by Bieniawski in 1973. It is originally based on 49 case histories from relatively large underground openings. Another 62 case histories from coal mining were added by 1987 and a total of 351 case histories were included by 1989 [42].

Since its introduction in 1973 the RMR system has been updated in 1974, 1975, 1976, 1979 and 1989. When using the RMR system it is therefore important to state which of these versions that is used.

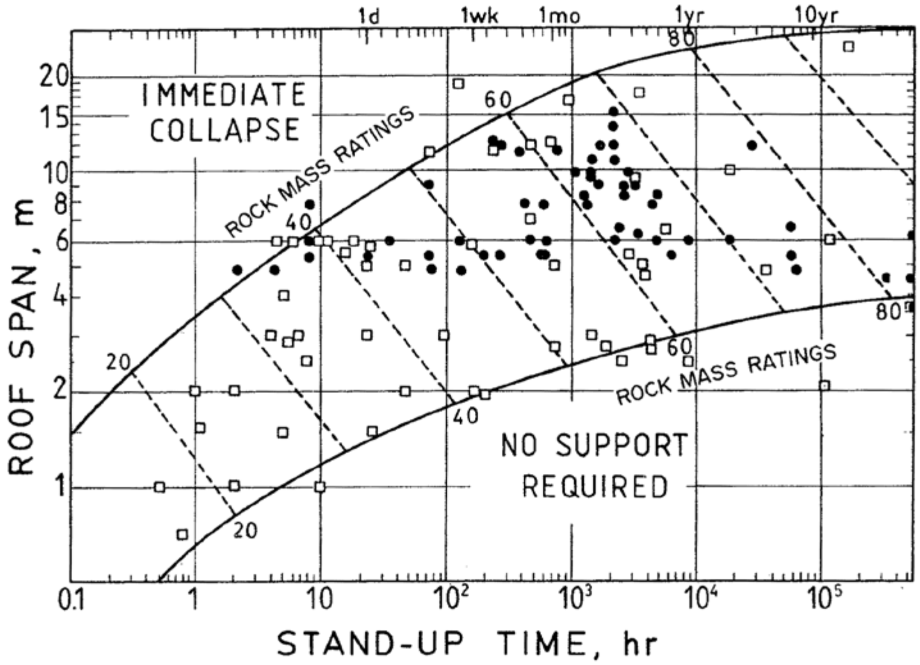


Figure 3-4 RMR rock mass classes with case histories. ● = mining roof falls; □ = tunnelling roof falls; contour lines = limits of applicability [22].

Bieniawski has presented the following aims of the RMR-system [22]:

1) To identify the most significant parameters influencing the behaviour of a rock mass.
2) To divide a particular rock mass formation into a number of rock mass classes of varying quality.
3) To provide a basis for understanding the characteristics of each rock mass class.
4) To derive quantitative data for engineering design.
5) To provide a common basis for communication between engineers and geologists.

Six parameters are used in the RMR system to classify rock masses. These are:

1) Uniaxial compressive strength of intact rock material
2) Rock quality designation (RQD)
3) Spacing of discontinuities
4) Condition of discontinuities
5) Groundwater conditions
6) Orientation of discontinuities

The sum of ratings for each of these parameters is the RMR-value. Parameters and ratings are presented in Appendix 5. Sometimes an RMR_{basic}-value is referred to which is the sum of the first five rating parameters only. The reason for this is that the rating of the sixth parameter is evaluated from the type of engineering application (tunnel, foundations & dams) and as such is not directly related to the rock mass. Therefore the RMR-value is sometimes expressed as:

$$RMR = RMR_{basic} + \textit{adjustment for orientation of discontinuities}$$

The resulting RMR-value is grouped into one of five rock mass classes ranging from “Very poor” to “Very good”, see table Table 3.3 below.

Table 3.3 Rock mass classes in the RMR system and their meanings (modified after Edelbro [13])

	Rock Mass Rating (rock class)				
RMR-value	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Classification of rock mass	Very good	Good	Fair	Poor	Very poor
Average stand-up time	10 years for a 15 m span	6 months for a 8 m span	1 week for a 5 m span	10 hours for a 2.5 m span	30 minutes for a 1 m span
Cohesion of the rock mass	> 400 kPa	300 – 400 kPa	200 – 300 kPa	100 – 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

The RMR-system also provides guidelines for selection of permanent rock reinforcement (see Appendix 5). The support load can be calculated from the RMR-value as suggested:

$$P = \left[\frac{100 - RMR}{100}\right]\gamma B \qquad \textit{Equation 3.6}$$

Where P is the support load (kN), B the tunnel width (m) and γ the rock density (kg/m^3).

Correlations have also been suggested between the modulus of deformability of the rock mass *in situ* and the RMR-value. For RMR-values greater than 50 the following correlation has been made:

$$E_M = 2 \times RMR - 100 \qquad \textit{Equation 3.7}$$

For RMR-values less than 50 the following correlation is proposed:

$$E_M = 10^{(RMR-10)/40} \qquad \textit{Equation 3.8}$$

According to Deere [22] the strengths of the RMR system are:

- It is simple to use and the classification parameters are easily obtained from borehole data or underground mappings.
- It is applicable and adaptable to many different situations, such as coal mining, hard rock mining, slope stability, foundation stability and tunnelling.
- It is capable of being incorporated to theoretical concepts.
- It is adaptable for use in knowledge-based expert systems. With fuzzy set methods, the subjectiveness or fuzziness inherent in a classification can be considered in the expert system.

The limitations, also according to Deere, are:

- The output from RMR classifications tends to be conservative which can lead to overdesigned rock support systems. It is therefore recommended that rock behaviour is monitored and that rock classification predictions are adjusted to local conditions.
- The range of applicability may be indicated by the data base used for its development (see Figure 3-4). Caution has to be used when applying the classification system outside these boundaries.

- Rock mass classifications are not to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and the site geology.

3.3 Q-system

The "Q-system" was published in 1974 by Barton, Lien and Lunde [7] from the Norwegian geotechnical institute (NGI). The original Q-system includes correlations between rock quality and support from more than 200 case records and as such is an empirical method. Many of the cases are from Cecil's works published in 1970 [11]. The examined case records include 9 sedimentary rock types, 13 igneous rock types, 24 metamorphic rock types and 9 sedimentary rock types. In more than 80 cases clay filled joints were involved, but in most cases the joints are unfilled with unweathered or only slightly weathered joint surfaces.

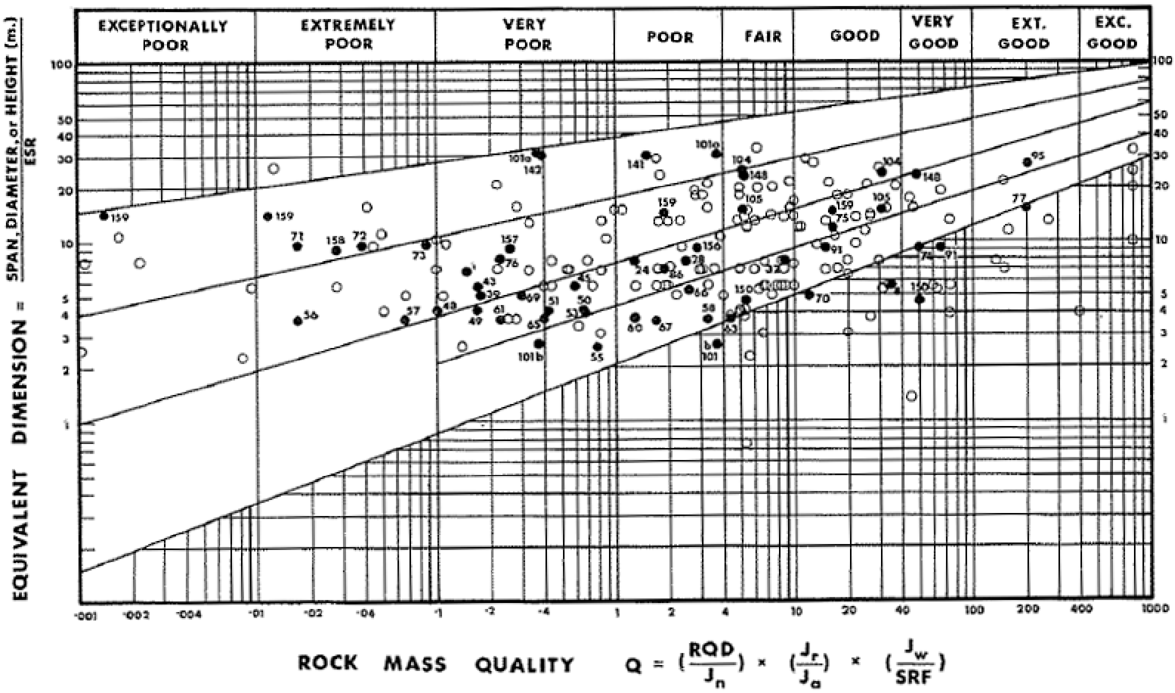


Figure 3-5 The original Q-system support chart with case records plotted.

To classify a rock mass, six basic parameters are determined, these are:

1) RQD (rock quality designation)
2) J_n (joint set number)
3) J_r (joint roughness number)
4) J_a (joint alteration number)
5) J_w (joint water reduction factor)
6) SRF (stress reduction factor)

From these parameters the Q-index (rock mass quality Q) is defined as:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \qquad \textit{Equation 3.9}$$

The Q-system claims to cover rock conditions from exceptionally poor, squeezing ground to exceptionally good unjointed rock masses, the corresponding Q-range being 0.001 to 1000. The resulting Q-value is used to classify the rock mass according to Table 3.4 below:

Table 3.4 Classification of rock mass based on Q-value [7].

Q-value	Classification
0.001- 0.01	Exceptionally poor
0.01- 0.1	Extremely poor
0.1- 1	Very poor
1- 4	Poor
4- 10	Fair
10- 40	Good
40- 100	Very good
100- 400	Extremely good
400- 1000	Exceptionally good

Barton *et al* present an empirical relationship between permanent rock support pressure and the Q-index which is based on 25 case records. For the roof pressure this is defined as:

$$P_{roof} = \left(\frac{2}{J_r}\right) \cdot Q^{-\frac{1}{3}} \qquad \left[kg/cm^2\right] \qquad \textit{Equation 3.10}$$

An improved relationship based on Equation 3.10 that also takes into account the joint set number (J_n) is also presented. This is defined as:

$$P_{roof} = \frac{2 \cdot J_n^{1/2} \cdot Q^{-1/3}}{3 \cdot J_r} \qquad \left[kg/cm^2\right] \qquad \textit{Equation 3.11}$$

The significance of J_n in Equation 3.11 is explained as being dependent of the presence of three or less joint sets, since this will limit the degree of freedom of block movement greatly. Both equations above will give same estimate of support pressure for three joint sets ($J_n = 9$) and Equation 3.11 will give lower support pressures compared to Equation 3.10 for less than three joint sets ($J_n < 9$) and vice versa.

3.4 Q-system development

Since its release in 1974 the Q-system has been updated and developed on several occasions. The main developments are shown in Table 3.5 below.

The latest update of the Q-system from 2002 [2] is a summary of developments of the Q-system since its first release. The application of the Q-system for prediction, correlation and extrapolation of site investigation data is also described.

Table 3.5 Main developments of the Q-system (modified after Palmström et al [32]).

Year	Development
1974	The Q-system is released.

Year	Development
1980	Hoek & Brown describes the use of the Q-system for input parameters of the Hoek & Brown failure criterion of rock masses.
1988	A new chart for rock support recommendations is presented (see Figure 3-8).
1992	The NMT concept is presented.
1993	Updating of the Q-system. Adjustment of SRF-values, incorporation of new support methods and addition of rock mass deformation modulus calculations.
1995	Uniaxial compressive strength is incorporated into the evaluation of the Q-value.
1999	The Q_{TBM}-system is released. Use for calculating penetration and advance rates in TBM-tunnelling from Q-values.
2002	Further development and summing up. Additional case records evaluated. [2]

With the introduction of the Q_c-value in 1995 into the Q-system, the uniaxial compressive strength (σ_c) is included in the description of rock quality. Normalisation of σ_c is done to the value of 100 MPa, which is considered the hard rock norm. The Q_c-value is used for correlating rock mass quality to seismic P-wave velocity, static deformation modulus of the rock mass and Lugeon values. The Q_c-value is defined as:

$$Q_c = Q \times \frac{\sigma_c}{100} \qquad \textit{Equation 3.12}$$

Based on data from several tunnelling projects a correlation between the rock mass quality and seismic P-wave velocity (V_p) has been proposed. Rock mass quality is input as the Q_c-value, thus taking into account the strength of the rock. Corrections for depth and porosity have been established from field data and are presented in a chart (see Figure 3-6). The correlation between Q_c and V_p is expressed as:

$$V_p \approx 3.5 + \log Q_c \qquad [km/s] \qquad \textit{Equation 3.13}$$

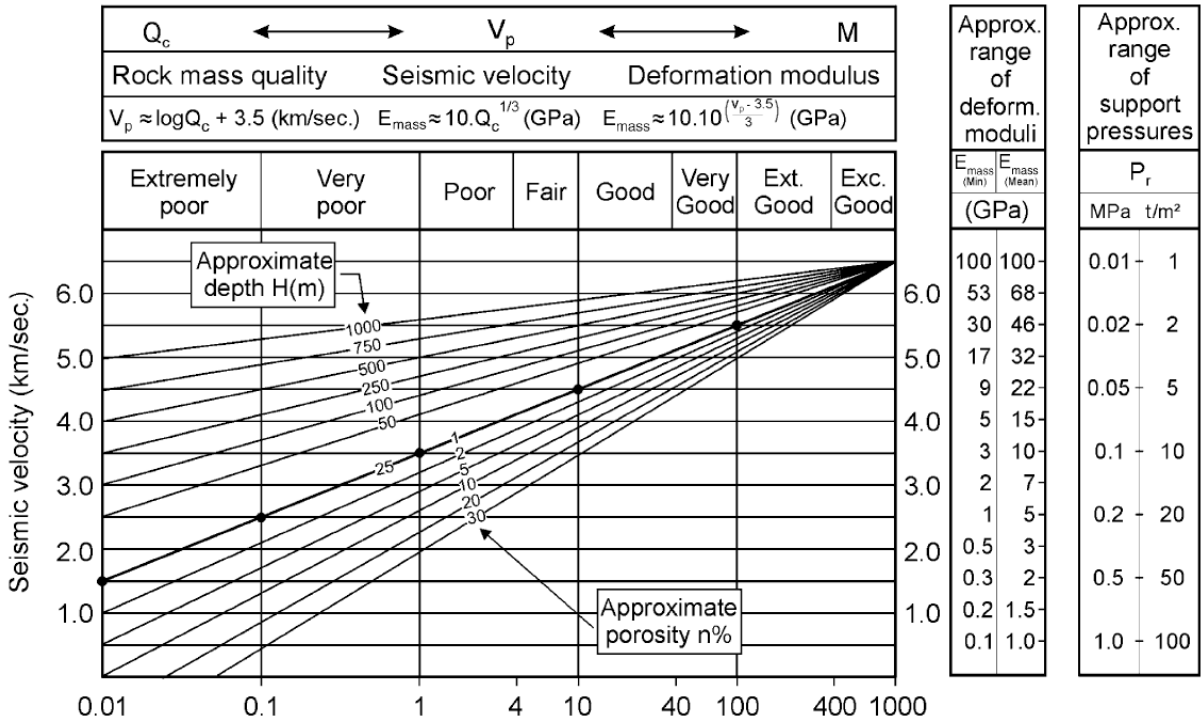
The static deformation modulus of a rock mass (E_{mass}) is closely related to the uniaxial compressive strength (σ_c) and subsequently the seismic P-wave velocity (V_p). Based on the Q_c-value, the following relationship to the static deformation modulus is presented:

$$E_{mass} \approx 10 \times Q_c^{1/3} \qquad [GPa] \qquad \textit{Equation 3.14}$$

Based on V_p, the relationship is:

$$E_{mass} \approx 10 \times 10^{(V_p - 3.5)/3} \qquad [GPa] \qquad \textit{Equation 3.15}$$

The static deformation modulus is integrated into the correction charts for seismic P-wave velocity in relation to depth and porosity (Figure 3-6). Note that the units used for V_p and E_{mass} are km/sec and GPa respectively.



$$Q_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right] \frac{\sigma_c}{100}$$

Figure 3-6 Correction chart for V_P in relation to depth and porosity. The bold line in the middle represents Equation 3.13.

The original correlation between rock mass quality (based on *Q*) and support pressure has been modified to output the result in units of MPa and is expressed as:

$$P_r = \frac{J_r}{20 \times Q^{1/3}} \qquad [MPa] \qquad \textit{Equation 3.16}$$

Approximately linear trends between tunnel deformation and Q-values normalised to the span have been noticed in log-log plots of collected data. It is pointed out that the spread of data is rather large which can be seen in Figure 3-7. For example, with a SPAN/Q ratio of 0.1 the corresponding range of deformation is between 1 and 100 mm with an average of 10 mm. The average trend line equation is presented below where *Δ* represents the deformation. Note that the units of *SPAN* and *Δ* are in meters and millimetres respectively.

$$\Delta \approx \frac{SPAN}{Q} \qquad [mm] \qquad \textit{Equation 3.17}$$

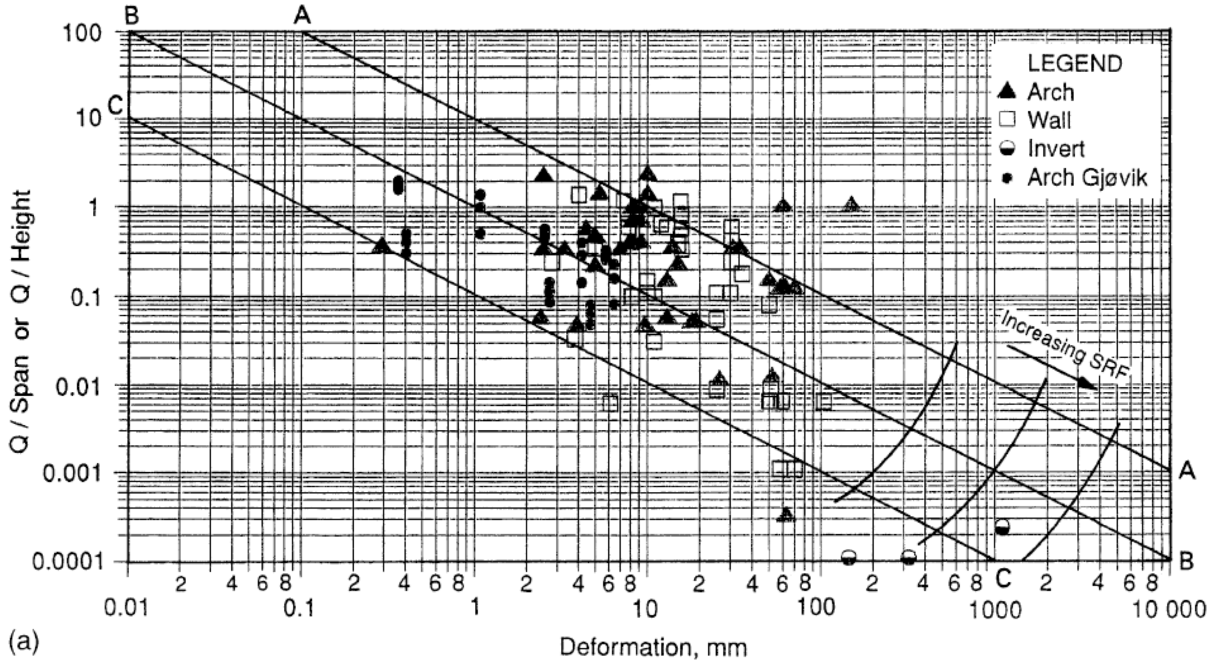


Figure 3-7 *Q-value/SPAN vs. radial deformation and convergence data for tunnels and caverns [2].*

Possible correlations between the Q-value and Lugeon-values are also discussed. For near-surface conditions (< 25 m) the simple relationship below is suggested:

$$L \approx \frac{1}{Q_c} \qquad \textit{Equation 3.18}$$

Barton also discusses the possible depth dependencies of this relationship in the latest update [2] and presents a chart showing tentative trends.

Other applications of the Q-system discussed by Barton are the evaluation of frictional and cohesive components of a rock mass and improvement of rock quality through pre-grouting.

The latest version of the Q-system is presented in Appendix 6. The most recent version of the Q-support chart is presented below in Figure 3-8. As can be seen, it has been quite reworked compared to the original Q-support chart from 1974 shown in Figure 3-5.

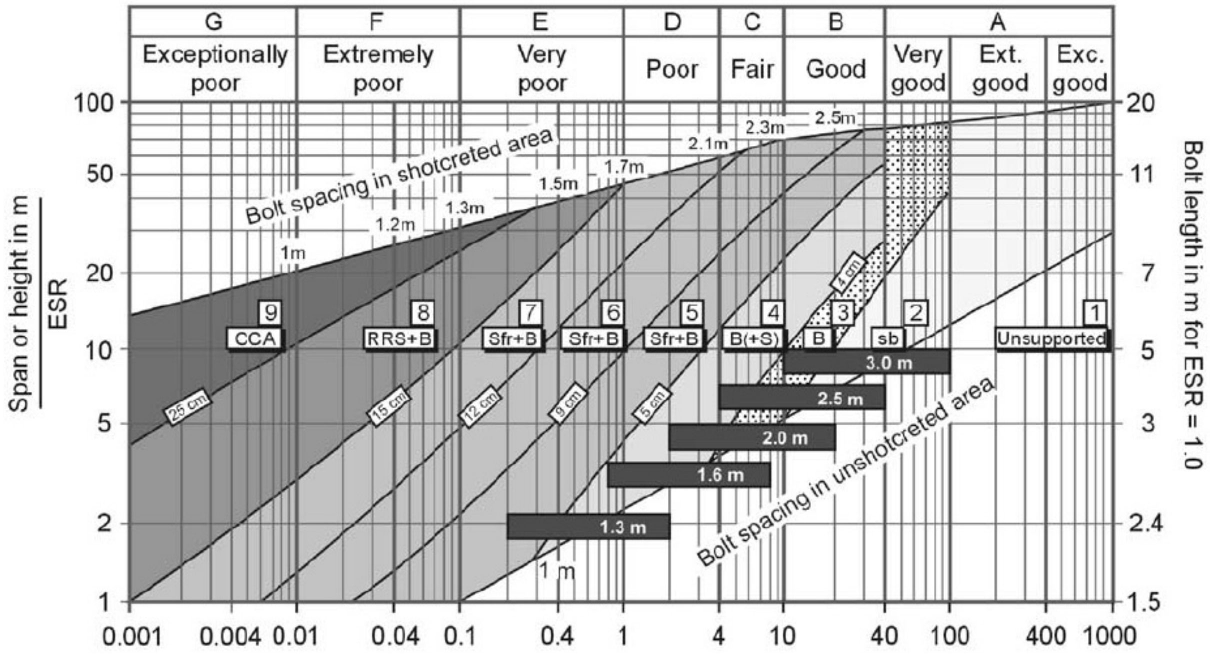


Figure 3-8 The 1993 updated Q-support chart.

3.5 Q_{TBM}

A development of the Q-system is the Q_{TBM} tunnelling prognosis model proposed by Barton & Abrahão [4] in which penetration and advance rates for TBMs can be calculated by using the six Q-parameters and five other parameters. Underlying the prognosis model are 140 case records. The parameters used for the model are:

RQD_0 = RQD oriented along the tunnel axis

J_n, J_r, J_a, J_w, SRF are the Q-parameters (see chapter 3.3)

SIGMA = rock mass strength [MPa]

F = thrust per cutter [tnf]

CLI = Cutter Life Index

q = quartz content [%]

σ_θ = average bi-axial stresses along the tunnel face [MPa]

From these parameters the value of Q_{TBM} is calculated as:

$$Q_{TBM} = \frac{RQD_0}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \cdot \frac{SIGMA}{F^{10}/20^9} \cdot \frac{20}{CLI} \cdot \frac{q}{20} \cdot \frac{\sigma_\theta}{5} \qquad \textit{Equation 3.19}$$

The penetration rate (*PR*) is then calculated using the following formula:

$$PR = 5(Q_{TBM})^{-0.2} \qquad \textit{Equation 3.20}$$

The actual advance rate (*AR*) can be calculated using:

$$AR = PR \cdot T^m \qquad \textit{Equation 3.21}$$

Where T is the time to advance a certain length and m (deceleration) is a performance indicator that is the negative gradient for reduced performance with increasing time. Deceleration m can be expressed as:

$$m = \log U / \log T \qquad \textit{Equation 3.22}$$

Where U is the utilisation.

In the article by Barton & Abrahão [4] an example is provided on how to perform the calculations needed to establish *PR* and *AR* from the input data along with explanations.

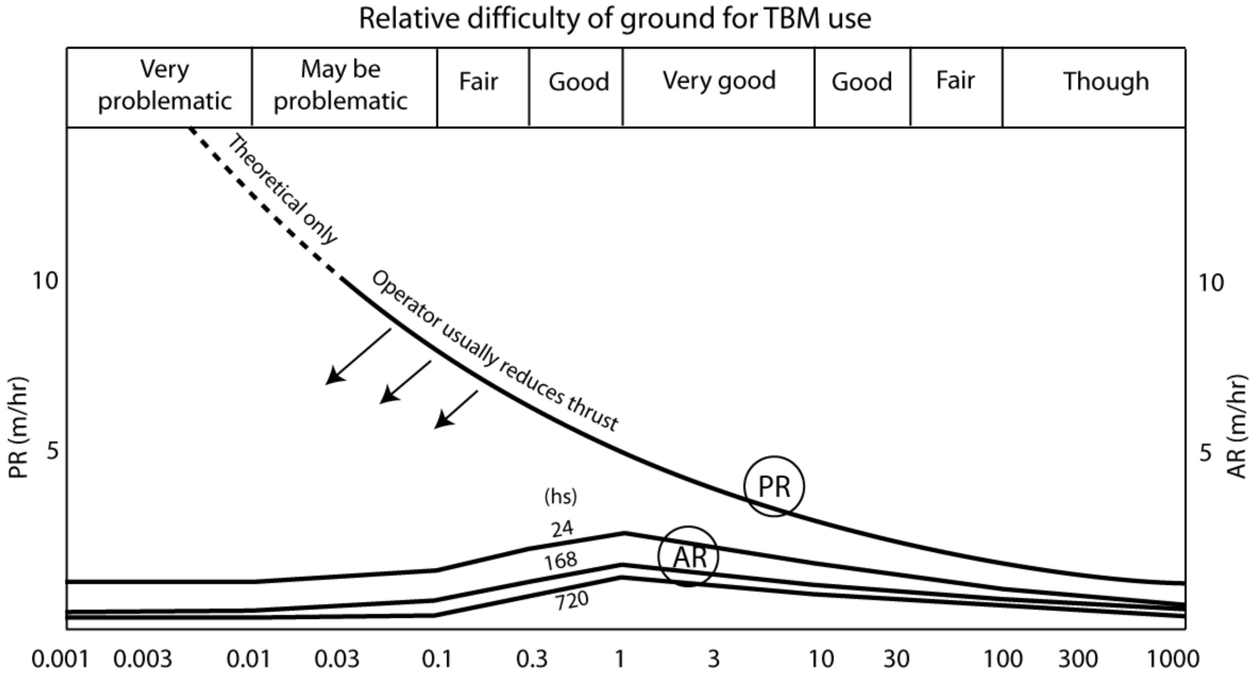


Figure 3-9 Correlation of advance- and penetration rates to Q_{TBM}-classification values (modified after Barton & Abrahão [4]).

3.5.1 Comments

Barton [3], in a reply to criticism of the Q_{TBM} prognosis model made by Blindheim [9], has acknowledged some of the limitations of the model due to the complexity of dealing with so many parameters and the need to include data from more TBMs. The main criticism by Blindheim is that the complex interaction of all the parameters involved in TBM tunnelling is difficult to quantify into a single Q_{TBM}-value. Blindheim is also of the opinion that some of the input parameters in the Q_{TBM} model are irrelevant or even misguiding for TBM performance. The last word in this debate is for sure not said.

Two models that are quite widely used are the NTH (Norwegian Institute of Technology) and CSM (Colorado School of Mines) methods [44]. The NTH method is an empirical method that uses a group of rock parameters and indices which were originally developed for drillability of hard rock. The CSM method is based on the individual cutter forces to determine overall performance. The formulas developed are based on full size cutting tests of cutters in various rock types. Both models have been compared several times with results that are close to each other [44].

A rock mass excavability (RME) indicator has recently (2006) been proposed by Bieniawski von Preinl *et al* [8] to predict TBM performance that is based on five input parameters: (1) uniaxial compressive strength, (2) abrasivity, (3) rock mass jointing at the tunnel front, (4) stand-up time depending on excavation method and (5) ground water inflow. RME is correlated to output values such as: rate of advance (ATA), penetration rate and specific energy of excavation. The RME indicator is intended as an aid to designers in choosing the most effective tunnel construction method in the early stages of a tunnelling project. Currently the RME indicator is mainly based on cases from double-shielded TBMs, but future work is said to include data from other types of machines as well.

The Q_{TBM} model can be considered being mainly an empirical method based on the Q-system, rock strength and stress properties with some additional machine specific data (cutter thrust; F & cutter life index; CLI).

3.6 GSI

The Geological Strength Index (GSI) has been developed by Hoek *et al* in several stages since 1992 [28] with the main purpose of serving as input to the well known Hoek-Brown failure criterion in poorer rock masses where the RMR system is not sufficient (RMR < 25) [19].

In the version from 2002 [17], the generalised Hoek-Brown failure criterion for jointed rock masses is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \qquad \textit{Equation 3.23}$$

Where σ_1' is the major principal stress, σ_3' is the minor principal stress, σ_{ci} is the uniaxial compressive strength of the intact rock material and m_b is a reduced value of the material constant m_i that is given by:

$$m_b = m_i \times e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad \textit{Equation 3.24}$$

Estimates of m_i for intact rock mass, *D*, which is a factor depending on the disturbance of the rock mass due to blast damage and stress relaxation and *GSI* are presented in Appendix 7. Note there are two input tables for the *GSI* estimate, one for jointed rock and one for heterogeneous rock masses.

The material constants for the rock mass, *s* and *a*, are given by following relationships:

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad \textit{Equation 3.25}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \qquad \textit{Equation 3.26}$$

Uniaxial compressive strength is obtained by setting $\sigma_3' = 0$ in Equation 3.23, giving:

$$\sigma_c = \sigma_{ci} \times s^a \qquad \textit{Equation 3.27}$$

Tensile strength is obtained by setting $\sigma_1' = \sigma_3' = \sigma_t$ in Equation 3.23, giving:

$$\sigma_t = -\frac{s \times \sigma_{ci}}{m_b} \qquad \textit{Equation 3.28}$$

For rock masses with $\sigma_{ci} \leq 100\ MPa$ the rock mass modulus is given by:

$$E_m = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{((GSI-10)/40)} \qquad \textit{Equation 3.29}$$

For $\sigma_{ci} < 100\ MPa$ the rock mass modulus is given by:

$$E_m = \left(1 - \frac{D}{2}\right) \times 10^{((GSI-10)/40)} \qquad \textit{Equation 3.30}$$

The GSI system is based on assessment of lithology, structure and condition of discontinuity surfaces from visual inspections of outcrops, excavations or boreholes. The descriptions of rock masses and other input parameters (see Appendix 7) are essentially qualitative.

The free computer software RocLab by RocScience [43] is very helpful in determining the rock mass properties. The input tables are built into the software and calculations are presented as failure envelope plots and in plain text (see Figure 3-10 below).

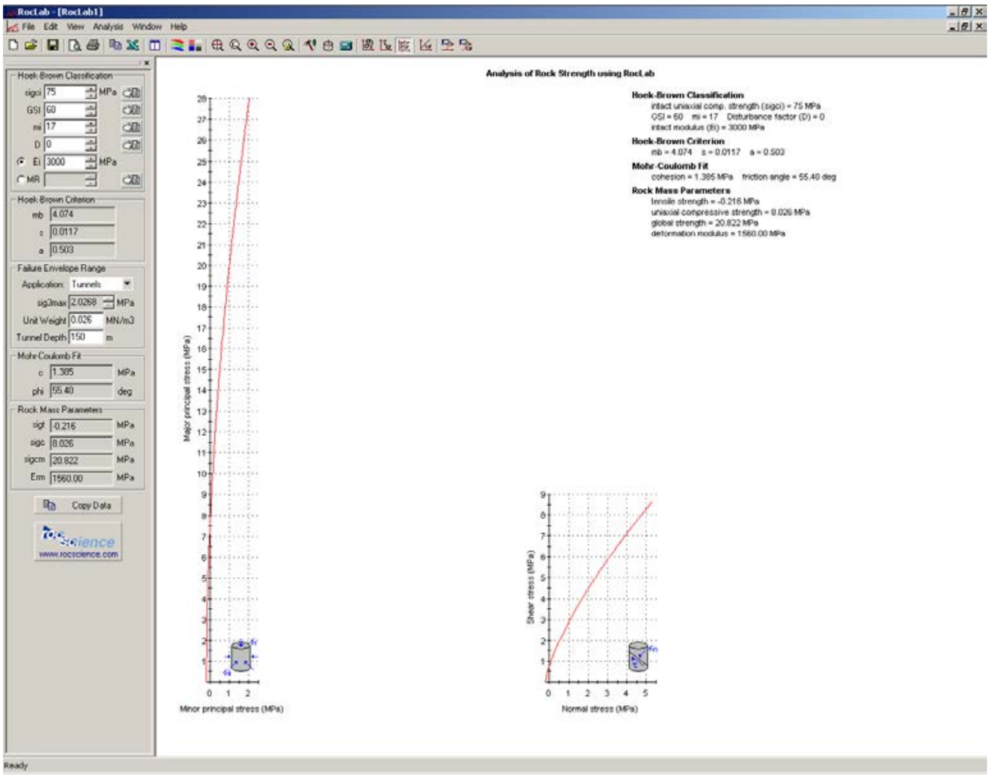


Figure 3-10 Screenshot from the RocLab [43] analysis result window.

Hoek *et al* [19] give some instructions on how GSI can be estimated based on RMR or Q-ratings. For $RMR_{89} > 23$ GSI is estimated from:

$$GSI = RMR_{89}' - 5 \qquad \textit{Equation 3.31}$$

When using this relationship the RMR parameters for groundwater is set to 15 and the parameter for joint orientation is set to zero. The resulting RMR is the RMR_{89}' value that is used in Equation 3.31 above.

For RMR below 23 it is recommended that the Q-system be used instead for estimating GSI from:

$$GSI = 9 \times \log_e Q' + 44$$ *Equation 3.32*

The value of Q' in this equation is the Q-index with the joint water reduction factor (Jw) and stress reduction factor set to 1 respectively. The modified Q-index (Q') is calculated from:

$$Q' = \frac{RQD}{J_n} \times \frac{J_r}{J_a}$$ *Equation 3.33*

3.7 Ground reaction curve

The interaction between stresses and deformations around a tunnel opening and the support elements can be represented by a “ground reaction curve”, commonly abbreviated GRC. As the tunnel is excavated a redistribution of stresses will occur in the rock mass resulting in deformations of the tunnel walls. If the deformations are large enough, the rock mass closest to the tunnel opening will plasticise. The fictitious, stabilising inner pressure of the tunnel is plotted as a function of the deformations. The inner pressure is used to determine the load on the support.

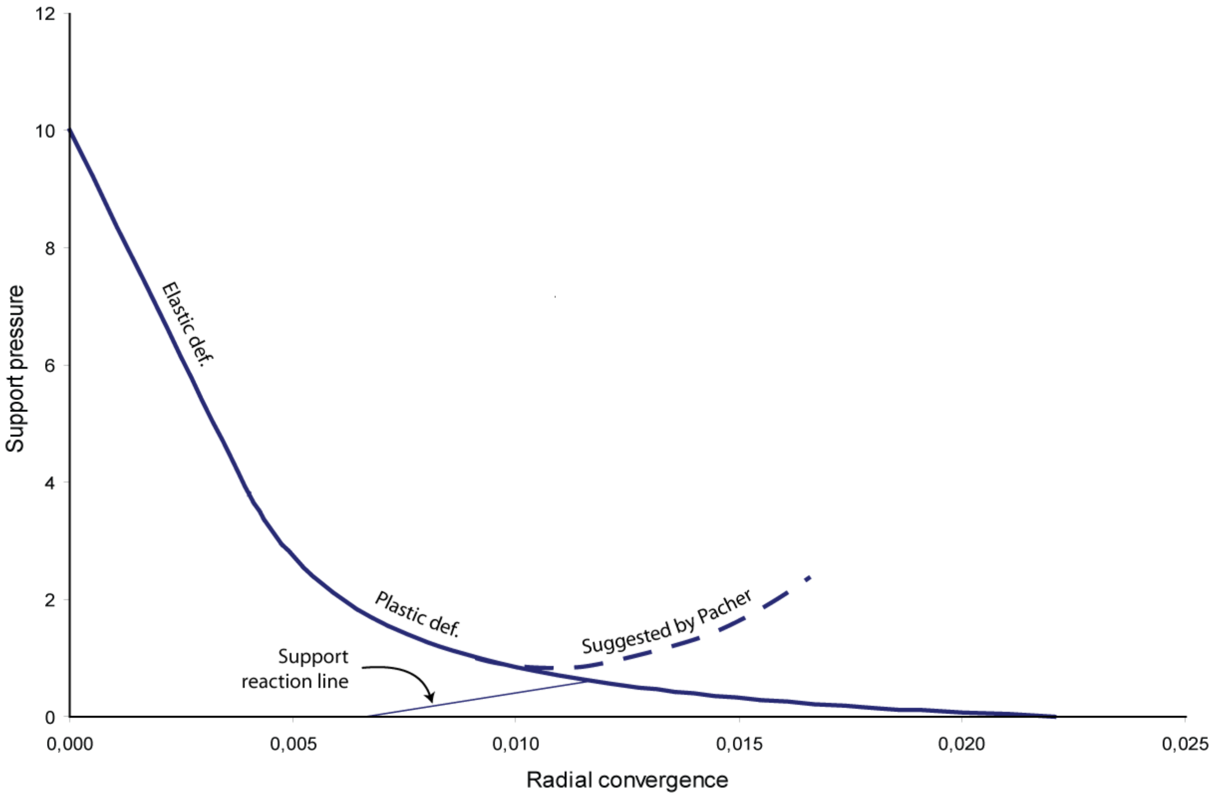


Figure 3-11 Rock–support interaction diagram with ground reaction curve and Pacher’s suggested curve.

One of the first published solutions to the ground reaction curve was done by Fenner in 1938 [14] (see also ch. 2.2.2). Fenner used the Mohr-Coulomb strength criterion and an elastic-plastic stress-strain model, not considering plastic volumetric strains [10].

In 1964 Pacher supplemented Fenner’s ground characteristics curve to describe the processes that take place when excavating a tunnel. Pacher proposed a through shaped curve that have a minimum internal pressure at a certain deformation and after this point the internal pressure will increase again [25] (see Figure 5-1). The curve is therefore sometimes referred to as the “Fenner-Pacher curve”, especially in NATM literature.

The GRC is often associated with the NATM, but in fact the original definition of NATM from 1964 makes no reference to the Fenner-Pacher curve [15]. It has however been used in later definitions of NATM to explain the deformations around the excavation and the support pressures.

3.7.1 Analytical solution

Deformations around an excavation front are a complex three dimensional problem. To be able to handle these with analytical solutions, the problem can be considered in two dimensions and the GRC concept can be used. The GRC concept makes the following assumptions:

i) The excavation is circular.

ii) The rock mass is isotropic and homogenous.

iii) The rock mass is an initially elastic material.

iv) The stresses of the rock mass are hydrostatic, i.e. equal in all directions.

By simplifying the problem according to above, the ground reaction curve can be calculated for a rock mass. A summary of solutions is presented in Appendix 3 that has been compiled by Brown *et al* [10].

If the rock mass is grouted or reinforced with grouted bolts, other calculations are needed since the reinforcements will interact with the rock mass. Stille *et al* [50] have presented GRC solutions for these cases as well as for unsupported rock masses. The solution for an unsupported elasto-plastic rock mass is described briefly below [50].

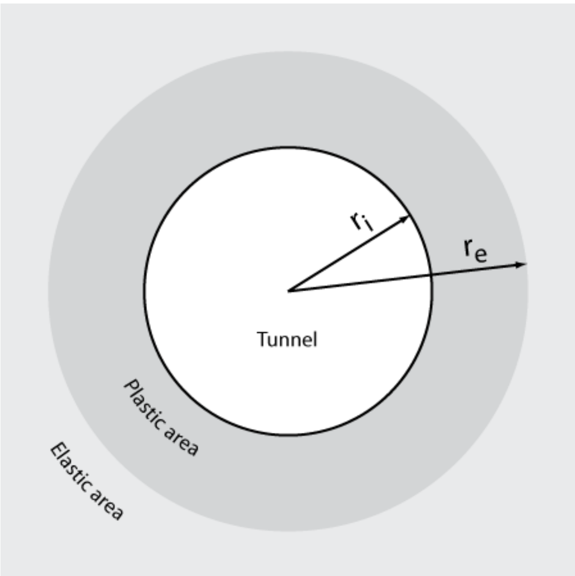


Figure 3-12 The rock mass may deform both plastically and elastically around a tunnel excavation if the stresses are large enough.

The boundary between the elastic and plastic zone (see Figure 5-2) is calculated using:

$$\frac{r_e}{r_i} = \left[\frac{\left(\frac{2}{1+k} \cdot (p_0 + a) - a \right) + a_r}{p_i + a_r} \right]^{\frac{1}{k_r - 1}} \qquad \textit{Equation 5.34}$$

Where:

The subscript “r” denotes residual values.

r_i = the tunnel radius

r_e = radius of the plastic/elastic boundary

p_0 = the initial stress in the rock mass before excavation

p_i = the fictitious inner radial pressure

c = cohesion of the rock mass

Φ = inner friction angle of the rock mass

$$a = \frac{c}{\tan \Phi} \qquad \textit{Equation 5.35}$$

$$k = \tan^2 (45° + \Phi / 2) \qquad \textit{Equation 5.36}$$

If r_e/r_i is smaller than 1, only elastic deformations are present around the tunnel opening which is represented by the straight line on the GRC (see Figure 5-1). After this point and as the deformations increase, plasticising will take place around the tunnel opening until the inner pressure reaches zero.

The radial stress at the boundary between the elastic and plastic zone is calculated as:

$$\sigma_{re} = \frac{2}{1+k} \cdot (p_0 + a) - a \qquad \textit{Equation 5.37}$$

The radial deformation if only elastic deformations are present ($r_e/r_i > 1$) is calculated as:

$$u_i = r_i \frac{1+\nu}{E} \cdot (p_0 - \sigma_{re}) \qquad \textit{Equation 5.38}$$

The radial deformation when plasticising has occurred ($r_e/r_i < 1$) is calculated as:

$$u_i = r_i \frac{r_i A}{f+1} \cdot \left[2 \left(\frac{r_e}{r_i} \right)^{f+1} + (f-1) \right] \qquad \textit{Equation 5.39}$$

Where:

$$A = \frac{1+\nu}{E} \cdot (p_0 - \sigma_{re}) \qquad \textit{Equation 5.40}$$

$$f = \frac{\tan(45° + \Phi / 2)}{\tan(45° + \Phi / 2 + \psi)} \qquad \textit{Equation 5.41}$$

E = Young's modulus of the rock mass

ν = Poisson's ratio of the rock mass

ψ = dilatancy angle

These calculations can be done easily in a spreadsheet programme and the GRC be plotted graphically. The easiest way is to prepare a table with three columns containing the input values for p_i and the calculated values of r_e/r_i and u_i. The first two rows shall contain the points that represent the elastic part of the radial deformation, *i.e.* at initial pressure p_0 and σ_{re}, and the last row the point representing zero inner pressure (p_i=0). The points in-between are on the plastic part of the GRC and are selected so that a good curve fitting is achieved using the chart functions of the spreadsheet programme. p_i-values are plotted on the y-axis of the chart and the corresponding u_i-values on the x-axis. A GRC table example setup is shown in Table 5.1 below.

Table 3.6 Example spreadsheet setup for GRC-plotting.

Row	**p_i**	**r_e/r_i**	**u_i**
1	Enter p0 value here		Enter "0" here
2	Enter σre value here	Enter "1" here	Calculate using Equation 3.38
…	($\sigma_{re} > p_i > 0$)	Calculate using Equation 3.34	Calculate using Equation 3.39
n	Enter "0" on the last line	Calculate using Equation 3.34	Calculate using Equation 3.39

References

[1] Ayaydin, N. (2000). Classification of excavation with Austrian code B2203: main aspects and experiences. In: Tunnelling and underground space technology, Vol 15, No 4, p 491-495. ISSN 0886-7798.

[2] Barton, N. (2002). Some new Q-value correlations to assist in site characterisation and tunnel design. In: International journal of rock mechanics and mining sciences, Vol 39, No 2, p 185-216. ISSN 1365-1609.

[3] Barton, N. (2005). Comments on 'a critique of QTBM'. In: Tunnels and tunnelling international, July, 2005, p 16-19. ISSN 1369-3999.

[4] Barton, N., Abrahão, R. (2003). Employing the Q_{TBM} prognosis model. In: Tunnels and tunnelling international, December, p 20-23. ISSN 1369-3999.

[5] Barton, N., Grimstad, E. (1994). Rock mass conditions dictate choice between NMT and NATM. In: Tunnels and tunnelling international, October, 1994. ISSN 1369-3999.

[6] Barton, N., Grimstad, E., Aas, G., Opsahl, O. A., Bakken, A., Pedersen, L., Johansen, E. D. (1992). Norwegian method of tunnelling. In: World Tunnelling, June & August, 1992. ISSN 0956-8700.

[7] Barton, N., Lien, R., Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. Norwegian geotechnical institute, No 106. ISBN 82-546-0034-1.

[8] Bieniawski von Preinl, Z. T., Celada Tamames, B., Galera Fernández, J. M., Álvarez Hernández, M. (2006). Rock mass excavability indicator: new way to selecting the optimum tunnel construction method. In: Tunnelling and underground space technology, Vol 21, No 3-4, p 237. ISSN 0886-7798.

[9] Blindheim, O. T. (2005). A critique of Q_{TBM}. In: Tunnels and tunnelling international, June, 2005, p 32-35. ISSN 1369-3999.

[10] Brown, E. T., Bray, J. W., Ladanyi, B., Hoek, E. (1983). Ground response curve for rock tunnels. In: Journal of geotechnical engineering, Vol 109, No 1. ISSN 0733-9410.

[11] Cecil, O. S. (1970). Thesis: Correlations of rock bolt-shotcrete support and rock quality parameters in Scandinavian tunnels. Urbana University of Illinois, p 414.

[12] Chang, Y., Brantmark, J. (1996). Ung sprutbetong som bergförstärkning. Stiftelsen svensk bergteknisk forskning, No 24. ISSN 1104-1773, ISRN SVEBEFO-R--24--SE.

[13] Edelbro, C. (2003). Thesis: Rock mass strength - a review. Luleå University of technology, No 16. ISSN 1402-1536, ISRN LTU-TR--03/16--SE. URL: http://pure.ltu.se/ws/fbspretrieve/158179.

[14] Fenner, R. (1938). Untersuchungen zur Erkenntnis des Gebirgsdruckes. In: Glückauf, Vol 74, p 681-695, 705-715. ISSN 0340-7896.

[15] Golser, J. (1996). Controversial views on NATM. In: Felsbau: Zeitschrift für Geomechanik und Ingenieurgeologie im Bauwesen und Bergbau, Vol 14, No 2, p 69-75. ISSN 0174-6979.

[16] Goodman, R. E. (2002). Karl Terzaghi's legacy in geotechnical engineering. In: Geo-Strata, October 2002. URL: http://www.geoengineer.org/terzaghi2.html.

[17] Hoek, E., Carranza-Torres, C., Corkum, B. (2002). Hoek-Brown failure criterion - 2002 edition. In: NARMS - TAC conference, Toronto, 2002, p 267-273.

[18] Hoek, E., Diederichs, M. S. (2006). Empirical estimation of rock mass modulus. In: International journal of rock mechanics and mining sciences, No 43, p 203-215. ISSN 1365-1609.

[19] Hoek, E., Kaiser, P. K., Bawden, W. F. (1998). Support of underground excavations in hard rock. A.A. Balkema. ISBN 90-5410-187-3.

[20] Holmberg, M., Stille, H. (2007). Observationsmetodens grunder och dess tillämpningar på design av konstruktioner i berg. SveBeFo rapport 80. ISSN 1104-1773.

[21] ITA Workgroup on General Approaches to Design, Duddeck, H. (Ed.). (1988). ITA Guidelines for the design of tunnels. In: Tunnelling and underground space technology incorporating trenchless technology research, Vol 3, No 3, p 237-249. ISSN 0886-7798. URL: http://www.ats.org.au/upload/uploads/TUST_88_v3_n3_237-249.pdf.

[22] Karakus, M., Fowell, R. J. (2004). An insight into the new Austrian tunnelling method (NATM). In: ROCKMEC 2004 - 7th regional rock mechanics symposium, Sivas, Turkey.

[23] Kirkaldie, L. (Ed.). (1988). Rock classification systems for engineering purposes. In: Symposium on rock classification systems for engineering purposes, Cincinatti, Ohio, 1987. ISBN 0-8031-0988-1.

[24] Kovári, K. (2003). History of the sprayed concrete lining method - part I: milestones up to the 1960s. In: Tunnelling and underground space technology incorporating trenchless technology research, Vol 18, No 1, p 57-69. ISSN 0886-7798.

[25] Kovári, K. (2003). History of the sprayed concrete lining method - part II: milestones up to the 1960s. In: Tunnelling and underground space technology incorporating trenchless technology research, Vol 18, No 1, p 71-83. ISSN 0886-7798.

[26] Kovári, K., Lunardi, P. (2000). On the observational method in tunnelling. Swiss federal institute of technology Zürich, Institute for geotechnical engineering. URL: http://www.igt.ethz.ch/www/search/PublicationDisplay.asp?publication=539.

[27] Kuvik, M., Wagner, P. (1999). The relation among various methods of rock mass quality evaluation for the tunnelling purposes. In: Acta geologica universitatis comenianae, No 54, p 69-77. ISSN 1335-2830. URL: http://www.fns.uniba.sk/prifuk/casopisy/geol/199954/index.htm.

[28] Lauffer, H. (1997). Rock classification methods based on the excavation response. In: Felsbau: Zeitschrift für Geomechanik und Ingenieurgeologie im Bauwesen und Bergbau, Vol 15, No 3, p 179-182. ISSN 0174-6979.

[29] Marinos, V., Marinos, P., Hoek, E. (2005). The geological strength index: applications and limitations. In: Bulletin of engineering geology and the environment, Vol 64, No 1, p 55-65. ISSN 1435-9529.

[30] Milne, D., Hadjigeorgiou, J., Pakalnis, R. (1998). Rock mass characterisation for underground hard rock mines. In: Tunnelling and underground space technology. ISSN 0886-7798.

[31] Müller, L. (1978). The basic ideas and principles of the so-called "New Austrian tunnelling method". In: Shotcrete for underground support III, p 53-63.

[32] Palmström, A. (2000). On classification systems. In: GeoEng2000 conference. URL: http://www.rockmass.net/misc/documents/onclasssystems.pdf.

[33] Palmström, A., Blindheim, O. T., Broch, E. (2002). Q-systemet - muligheter og begrensninger. In: Norwegian tunnelling conference, p 44. URL: http://www.rockmass.net/misc/documents/qsystem.pdf.

[34] Palmström, A., Broch, E. (2006). Use and misuse of rock mass classification systems with particular reference to the Q-system. In: Tunnelling and underground space technology, Vol 21, No 6, p 575-593. ISSN 0886-7798.

[35] Palmström, A., Stille, H. (2007). Ground behaviour and rock engineering tools for underground excavations. In: Tunnelling and underground space technology incorporating trenchless technology research, Vol 22, p 363-376. ISSN 0886-7798.

[36] Peck, R. B. (1969). Advantages and limitations of the observational method in applied soil mechanics. In: Géotechnique, Vol 19, No 2, p 171-187.

[37] Powderham, A. J. (1998). The observational method - application through progressive modification. In: Civil engineering practice, Fall/winter 1998, p 87-110. ISSN 0886-9685. URL: http://www.cepractice.org.

[38] Rabcewicz, L. (1964). The new Austrian tunnelling method, part one. In: Water power, November 1964, p 453-457.

[39] Rabcewicz, L. (1964). The new Austrian tunnelling method, part two. In: Water power, December 1964, p 511-515.

[40] Rabcewicz, L. (1965). Die neue Österreichische Tunnelbauweise. 1. Entstehung, Ausführungen und Erfahrungen. In: Der Bauingenieur, Vol 40, Heft 8 1965, p 289-301. ISSN 0005-6650.

[41] Rabcewicz, L. (1965). The new Austrian tunnelling method, part three. In: Water power, January 1965, p 19-24.

[42] Rabcewicz, L., Golser, J. (1973). Principles of dimensioning the supporting systems for the "New Austrian tunnelling method". In: Water power, March, 1973, p 88-93.

[43] Rawlings, C. G., Barton, N., Smallwood, A., Davies, N. (1995). Rock mass characterisation using the Q and RMR systems. In: VIII international congress on rock mechanics, Vol 1, p 29.

[44] Rocscience (2007). Computer software: RocLab, version 1.031. URL: http:/www.rocscience.com.

[45] Rostami, J., Ozdemir, L., Nilsen, B. (1996). Comparison between CSM and NTH hard rock TBM performance prediction models. In: ISDT Las Vegas 1996.

[46] Sandström, G. E. (1963). The history of tunnelling - underground workings through the ages. Barrie and Rockliff.

[47] Sauer, G. (1988). Further insight into the NATM. In: 23rd Sir Julius Wernher memorial lecture. URL: http://dr-sauer.com.cnchost.com/natm/f_inst.htm.

[48] Schubert, W. Grundlagen der New Austrian Tunnelling Method. Technische Universität Graz, Institut für Felsmechanik und Tunnelbau, LV: 220.001. URL: http://www.tunnel.tugraz.at/skriptum/Skriptum_NATM_2002.pdf.

[49] Seeber, G., Keller, S., Enzenberg, A., Tagwerker, J., Schletter, R., Schreyer, F., Coleselli, A. (1978). Methods of measurements for rock support and installations in road tunnels using the new Austrian tunnelling method. Bundesministerium für Bauten und Technik, Strassenforschung, Heft 133, 1978, p 200.

[50] Sjöström, Ö. (1993). Active tunnel design - a new concept for pre-design, contractual layout and construction management. In: Shotcrete for underground support V, p 90-102. ISBN 0-87262-944-9.

[51] Stille, H., Holmberg, M., Nord, G. (1989). Support of weak rock with grouted bolts and shotcrete. In: International journal of rock mechanics and mining sciences & geomechanical abstracts, Vol 26, No 1, p 99-113. ISSN 0148-9062.

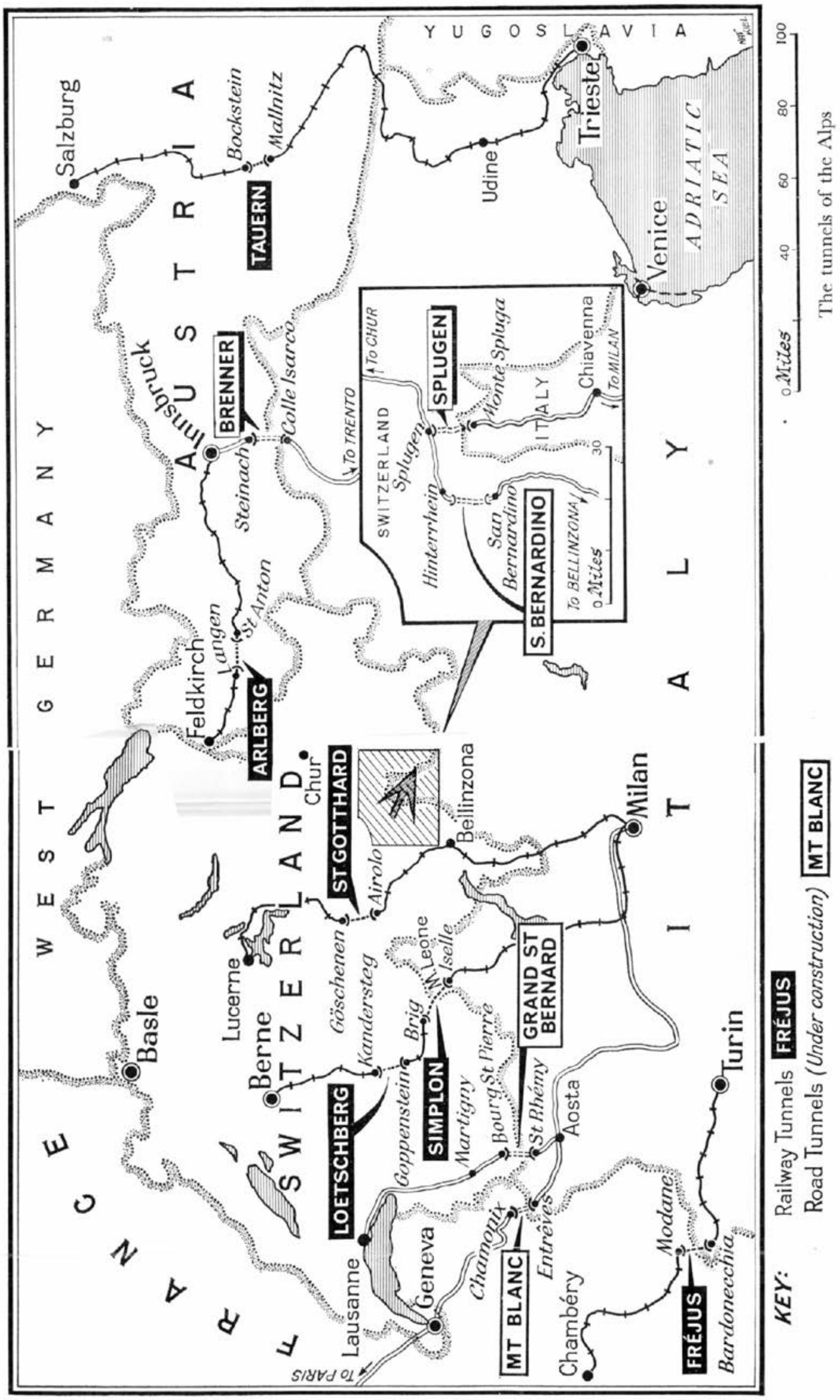
[52] Web site. Images from the Watson bridge book collection. Cleveland state university. URL: http://web.ulib.csuohio.edu/watson/albums.

[53] Zhang, L., Einstein, H. H. (2004). Using RQD to estimate the deformation modulus of rock masses. In: International journal of rock mechanics and mining sciences, No 41, p 337-341. ISSN 1365-1609.

Index

Appendix 1

The great early tunnels of the Alps [45]



The tunnels of the Alps

Appendix 2

Müller's 22 principles for NATM [30]:

1) Essentially, the bearing component part of a tunnel is the surrounding rock mass.
2) Therefore, it has to be one of the chief principles to preserve, as far as possible, the original mass resistance of the rock mass.
3) Loosening has to be prevented as far as possible; it leads to a considerable loss of strength.
4) Uniaxial and biaxial stress conditions ought to be avoided whenever the rock mass is scarcely able to bear them.
5) The rock deformations should be controlled so that, on the one hand. (by extending the rock mass toward the excavation) a protecting covering will be mobilized, and on the other hand, a loosening and decomposition can extensively be prevented. The better the success of this measure, the greater safety and economy.
6) For this purpose, the lining has to be timely placed, not too early and not too late , and the skin resistance has to be dosed accordingly - lining and support not too rigid, not too weak.
7) This requires a correct estimation of the specific time factor of the rock mass (or of the system lining and rock mass).
8) For the estimation of the time factor, preliminary laboratory tests as well as displacement measurements in the tunnel are performed; stand-up time, deformation velocity, and rock classification give an idea of this most important factor of influence.
9) In case larger deformations or loosening of the rock mass are being expected, the protection of the excavation has to act on the entire surface and has to be force grip. This can be reached best through shotcrete support.
10) The lining shall be slender and thus slack; in this way it minimizes the absorption of bending moment and the occurrence of bending fractures.
11) Strengthening of the lining, when necessary, is not done by increasing the thickness but by placing mesh reinforcements, tunnel ribs, and anchors.
12) Means and time of lining are determined on the basis of displacement measurements of the rock mass.
13) Statically, the tunnel is considered as a (thick-walled) tube consisting of a bearing ring of rock and the support or lining.
14) As a tube can statically act as a tube only if it is not slit, the closing of the ring (as far as the foundation rock does not act in this way itself) is of special importance.
15) The behaviour of the rock mass is essentially determined by the time required for closing the ring. Far advancing calottas extend the time and expose the cantilevering tunnel half-shell to undesirably great bending effects in the longitudinal direction of the tunnel; moreover, such calottas expose the rock mass below the toe of them to high loads.

16) For reasons of stress rearrangements, a heading in fu l profile is considered especially advantageous; heading in stages complicates and multiplies the stress rearrangements and damages the rock mass.

17) The mode of operation can be decisive for the safety of the structure as it influences the time factor of the rock mass. As a variation of the rounds of excavation on the time of lining and of closing the invert, the length of the calotta and the skin resistance are systematically used for controlling the procedure of stabilizing in the system rock mass plus lining support.

18) In order to prevent stress concentrations destroying the rock mass, corners of the profile shall be avoided and rounded shapes of the cross section shall be aimed for.

19) If the tunnel tube is designed in double shells, the inner shell is preferably slim. Force grip with the external shell is desired, not friction grip.

20) The total rock mass plus shell shall essentially be stabilized by the (preliminary) lining. The inner shell then serves to increase safety. (If aggressive ground water exists, the inner shell has to have total stabilization.) The anchors can be considered a permanent part of the system only if they are protected against corrosion to an extent determined by the rock mass.

21) Measurements of concrete stresses and of contact stresses between shell and rock mass serve as the control and dimensioning of the whole structure, with the continuation of the measurement of movements during construction.

22) The seepage pressure in the rock mass as well as the static pressure on the lining are relieved by drainage systems (e.g., Sika-hose-method).

Summary of solutions to the axisymmetric tunnel problem (GRC) [10]:

Author and year	**Strength/yield criterion**	**Stress-strain model**	**Treatment of plastic volumetric strain**	**Special features of analysis**
Fenner, 1938	Mohr-Coulomb	Elastic-plastic	None	
Kastner, 1948	Mohr-Coulomb	Elastic-plastic	None	Non hydrostatic stress field
Labasse, 1949	Mohr-Coulomb with zero cohesion	Elastic-plastic	Evaluated an average volumetric strain in the plastic zone	Non hydrostatic stress field
Morrison & Coates, 1955	Mohr-Coulomb peak and residual with constant Φ, zero residual cohesion	Elastic-brittle-plastic	None	Corrected error made by Fenner
Hobbs 1966	Non-linear power law with reduced strength in plastic zone	Elastic-brittle-plastic	None, but different E, ν used for plastic zone	
Bray, 1967	Mohr-Coulomb	Elastic-plastic	None	Slip on log spiral surfaces in plastic zone
Diest, 1967	Mohr-Coulomb with zero residual strength	Elastic-strain softening	None	
Salencon, 1969	Trasca and Mohr-Coulomb	Elastic-plastic	Used associated flow rule; rate of plastic volume change independent of strain	
Daemen & Fairhurst, 1971	Bilinear with different peak and residual strengths	Elastic-strain softening	Plastic volume change constant or varying linearly with radial strain	Closed-form solutions not presented
Lombardi, 1970	Mohr-Coulomb with different peak and residual cohesion and Φ	Elastic-brittle-plastic	Average volumetric strain in plastic zone estimated; different E, ν in plastic zone	

Author and year	**Strength/yield criterion**	**Stress-strain model**	**Treatment of plastic volumetric strain**	**Special features of analysis**
Henderdon & Aiyer, 1971	Mohr-Coulomb with constant Φ and either constant, varying or zero cohesion in plastic zone	Elastic-plastic, elastic brittle-plastic and a special case of elastic-strain softening	Associated flow rule applied over entire plastic zone; different E, ν in plastic zone in some solutions	Several different cases solved; all features not included in one solution
Ladanyi, 1974	Non-linear Fairhurst criteria for original and broken rock in short and long term	Elastic-brittle-plastic	Associated flow rule applied over limited range of post peak strain	
Egger, 1974	Mohr-Coulomb peak and residual with constant Φ, zero residual cohesion	Elastic-strain softening	Major and minor principal plastic strains linearly related related by variable parameter Φ	
Panet, 1976	Mohr-Coulomb peak and residual with constant Φ, zero residual cohesion	Elastic-strain softening	Major and minor principal plastic strains linearly related by variable parameter Φ	Allows for influence of tunnel face
Korbin, 1976	Piecewise linear Coulomb approximation of non-linear Mohr envelope	Non-linear strain softening	Uses Hendron and Aiyer’s application of the associated flow rule	
Kennedy & Lindberg, 1977	Piecewise linear Coulomb approximation of non-linear Mohr envelope	Elastic-plastic	Associated flow rule applied over entire plastic zone, an alternative incompressible flow solution presented	

Author and year	**Strength/yield criterion**	**Stress-strain model**	**Treatment of plastic volumetric strain**	**Special features of analysis**
Florence & Schwer, 1978	Mohr-Coulomb	Elastic plastic	Associated flow rule applied over entire plastic zone	Allows for influence of axial stress resulting in up to three different plastic zones depending on values of ν and Φ and relative magnitudes of tangential, radial and axial stresses
Nguyen Minh & Berest, 1979	Mohr-Coulomb peak and residual with constant Φ	Elastic-strain softening with possible class 2 behaviour	Major and minor principal plastic strains linearly related by variable parameter β	Allows for influence of axial stress resulting in two different plastic zones depending on relative magnitudes of tangential, radial and axial stresses
Schwartz & Einstein, 1980	Mohr-Coulomb	Elastic-plastic	Zero total volume change in plastic zone (non-associated flow rule)	Allows for influence of tunnel face
Hoek & Brown, 1980	Empirical non-linear peak and residual criteria	Elastic-brittle-plastic	Associated flow rule applied over limited range of post peak strain	Calculation steps given for complete ground support interaction calculations
Kaiser, 1980	Rate-dependent Mohr-Coulomb peak and residual	Elastic-brittle-plastic; stiffness and strength loss rate dependent	Major and minor principal plastic strains linerly related by variable parameter β	
Brown et al, 1982	Empirical non-linear peak and residual criteria	Elastic-brittle-plastic and elastic-strain softening	Post peak strain increments in two different regimes related by experimental parameters or by the associated flow rule	Closed form solution for simpler case; stepwise numerical solution for more complex material behaviour model

Essential features of NMT (Barton et al [6])

1) Areas of usual application:

   Jointed rock; harder end of scale (UCS = 3 to 300 MPa)

   Claybearing zones, stress slabbing (Q – 0.001 to 10)

2) Usual methods of excavation

   Drill and blast, hard rock TBM, hand excavation in clay zones.

3) Temporary support and permanent 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)

   - temporary support forms part of permanent support
   - mesh reinforcement not used
   - dry process shotcrete not used
   - steel sets or lattice girders not used; RRS used in clay zones
   - 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

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

Appendix 5

Rock Mass Rating (Bieniawski 1989)

RMR classification parameters and their ratings									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength (MPa)	>8	4-10	2-4	1-2	Use of UCS is preferred		
		Uniaxial compressive strength	>250	100-250	50-100	25-50	5-25	1-5	<1
		Rating	**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	RQD		Excellent	Good	Fair	Poor	Very poor		
			90-100	75-90	50-75	25-50	<25		
		Rating	**20**	**17**	**13**	**8**	**3**		
3	Spacing of discontinuities		Very wide	Wide	Moderate	Close	Very close		
			>2	0.6-2	0.2-0.6	0.006-0.2	<0.006		
		Rating	**20**	**15**	**10**	**8**	**5**		
4	Condition of discontinuities		Very rough surfaces. Not continuous. No separation. Unweathered wall rock.	Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls.	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls.	Slickensided surfaces. - or - Gouge < 5 mm thick. - or - Separation 1 - 5 mm. Continuous.	Soft gouge > 5 mm thick. Separation > 5 mm. Continuous.		
		Rating	**30**	**25**	**20**	**10**	**0**		
5	Groundwater	General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
		Inflow per 10 m tunnel length (l/min)	None	< 10	10-25	25-125	>125		
		Joint water pressure / major principal stress	0	<0.1	0.1-0.2	0.2-0.5	>0.5		
		Rating	**15**	**10**	**7**	**4**	**0**		

RMR; rating adjustment for discontinuity orientations							
			Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
6	Joint orientations, strike & dip	Tunnels and mines	**0**	**-2**	**-5**	**-10**	**-12**
		Foundations	**0**	**-2**	**-7**	**-15**	**-25**
		Slopes	**0**	**-5**	**-25**	**-50**	

RMR; assessment of joint orientation effects on tunnels				
	Dip	**45°-90°**	**20°-45°**	**0°-20°**
Strike perp. to tunnel axis	**Drive with dip**	Very favourable	Favourable	⇩
	Drive against dip	Fair	Unfavourable	⇩
Strike parallel to tunnel axis		Very unfavourable	Fair	⇩
Irrespective of strike				Fair

RMR; guidelines for excavation and support of 10 m span rock tunnels						
Rock mass class		**RMR**	**Excavation**	**Rock bolts***	**Shotcrete**	**Steel sets**
I	**Very good rock**	**81-100**	Full face, 3 m advance	Generally no support required except for spotbolting		
II	**Good rock**	**61-81**	Full face, 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None
III	**Fair rock**	**41-61**	Top heading and bench. 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None
IV	**Poor rock**	**21-40**	Top heading and bench. 1-1.5 m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V	**Very poor rock**	**<20**	Multiple drifts. 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after excavation.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

* Ø 20 mm. Fully grouted.

Appendix 6

NGI's Q-system (2002)

Table A1

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

Notes:
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.

Table A2

Joint set number		$\mathbf{J_n}$
A	Massive, none or few joints	0.5 - 1
B	One joint set	2
C	One joint set plus random	3
D	Two joint sets	4
E	Two joint sets plus random	6
F	Three joint sets	9
G	Three joint sets plus random	12
H	Four or more joint sets, random, heavily jointed, 'sugar cube' etc.	15
J	Crushed rock, earth like	20

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

Table A3

Joint roughness number		Jr
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
G	Slickensided, planar	0.5
c) No rock wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock wall contact	1
J	Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	1

Notes:

i) Descriptions refer to small-scale features and intermediate scale features, in that order.

ii) Add 1.0 if the mean spacing of relevant joint set is greater than 3 m.

iii) Jr = 0.5 can be used for planar, slickensided joints having lineation, provided the lineation is oriented for minimum strength.

iv) Jr and Ja 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 $\tau \approx \sigma n \tan\text{-}1\ (Jr/Ja)$).

Table A4

Joint alteration number		Φ_r (°)	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
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25 - 30	2
D	Silty or sandy clay coatings, small clay fraction (non-softening)	20 - 25	3
E	Softening or low friction clay mineral coatings, i.e., kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays.	8 - 16	4
b) Rock wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25 - 30	4
G	Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5 mm in thickness)	16 - 24	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, < 5 mm in thickness)	12 - 16	8
J	Swelling clay fillings, i.e., montmorillonite (continuous, < 5 mm in thickness). Value of J_a depends on percentage of swelling clay-sized particles, and access to water, etc.	6 - 12	8 - 12
c) No rock wall contact when sheared			
K L M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 - 24	6 8 8 - 12
N	Zones or bands of silty or sandy clay (non-softening)		5
O P R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6 - 24	10 13 13 - 20

Table A5

Joint water reduction factor		Approx. water pressure (kg/cm^2)	J_w
A	Dry excavations or minor inflow, i.e., 5 litres/minute locally	< 1	1
B	Medium inflow or pressure occasional out-wash 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 out-wash 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

Notes:

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.

iii) For general characterisation of rock masses distant from excavation influences, the use of J_w =1.0, 0.66, 0.5, 0.33, etc. as depth increases from say 0-5, 5-25, 25-250 to > 250 metres is recommended, assuming that RQD/J_n is low enough (e.g., 0.5-25) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterisation values of SRF. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

Table A6

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 ≤ 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 ≤ 50 m)			5
F	Single shear zones in competent rock (clay-free) (depth of excavation > 50 m)			2.5
G	Loose open joints, heavily jointed or "sugar cubes", etc. (any depth)			5
b) Competent rock, rock stress problems				
		σ_c/σ_1	σ_Θ/σ_c	
H	Low stress, near surface open joints	> 200	< 0.001	2.5
J	Medium stress, favourable stress conditions	200 - 10	0.01 - 0.3	1
K	High stress, very tight structure (usually favourable to stability, may be unfavourable to 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 deformations in massive rock	< 2	> 1	200 - 400

Stress reduction factor			SRF
c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressure			
		$\sigma_\Theta/\sigma c$	
O	Mild squeezing rock pressure	1-5	5 - 10
P	Heavy squeezing rock pressure	> 5	10 - 20
d) Mild swelling rock pressure			
R	Mild swelling rock pressure		5 - 10
S	Heavy swelling rock pressure		10 - 15

Notes:

i) Reduce these SRF values by 20-50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterisation.

ii) For strongly anisotropic stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to 0.75 σ_c, when $\sigma_1/\sigma_3 > 10$, reduce σ_c to 0.5 σ_c (where σ_c is unconfined compressive strength, σ_1 and σ_3 are major and minor principal stress, and σ_Θ. the maximum tangential stress (estimated from elastic theory)).

iii) Few case records available where depth of crown below surface is less than span width. Suggest an SRF increase from 2.5 to 5 for such cases (see H).

iv) Cases L, M and N are usually most relevant for support design of deep tunnel excavations in hard rock massive rock masses, with RQD/J_n ratios from about 50-200.

v) For general characterisation of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0 and 0.5 is recommended as depth increases from say 0-5, 5-25, 25250 to >250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterisation values of J_w. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these where developed.

vi) Cases of squeezing rock may occur for depth $H > 350 \times Q^{1/3}$. Rock mass compression strength can be estimated from $SIGMA_{cm} \approx 5 \times \gamma \times Q_c^{1/3}$ (MPa) where γ is the rock density in t/m^3, and $Q_c = Q \times \sigma_c/100$.

Appendix 7

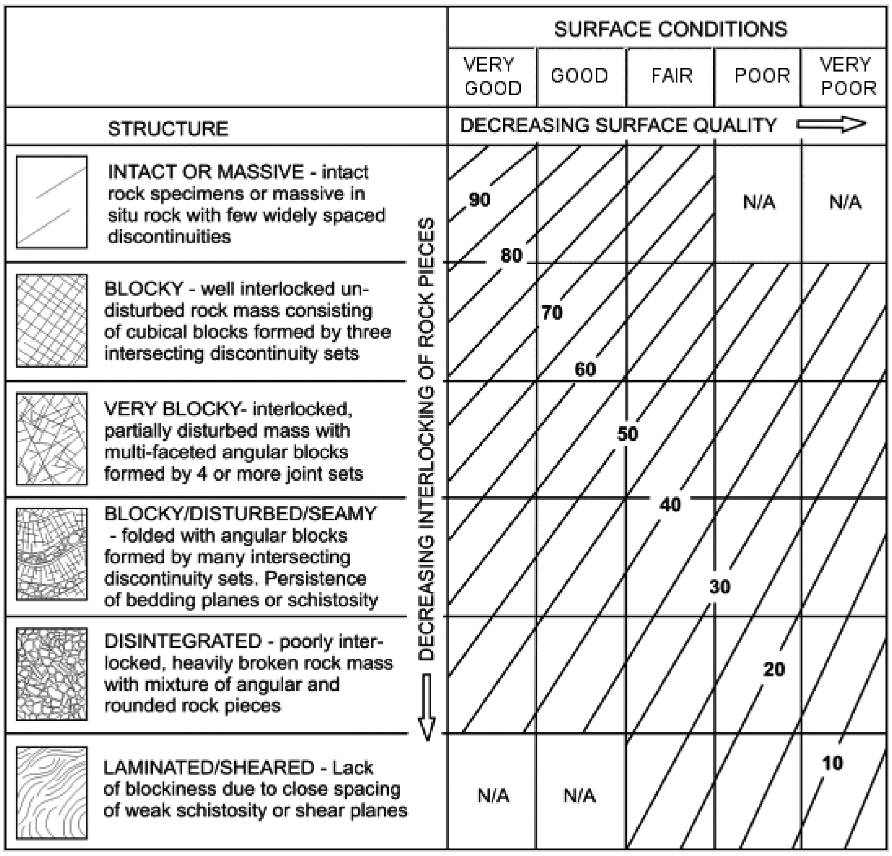
GSI

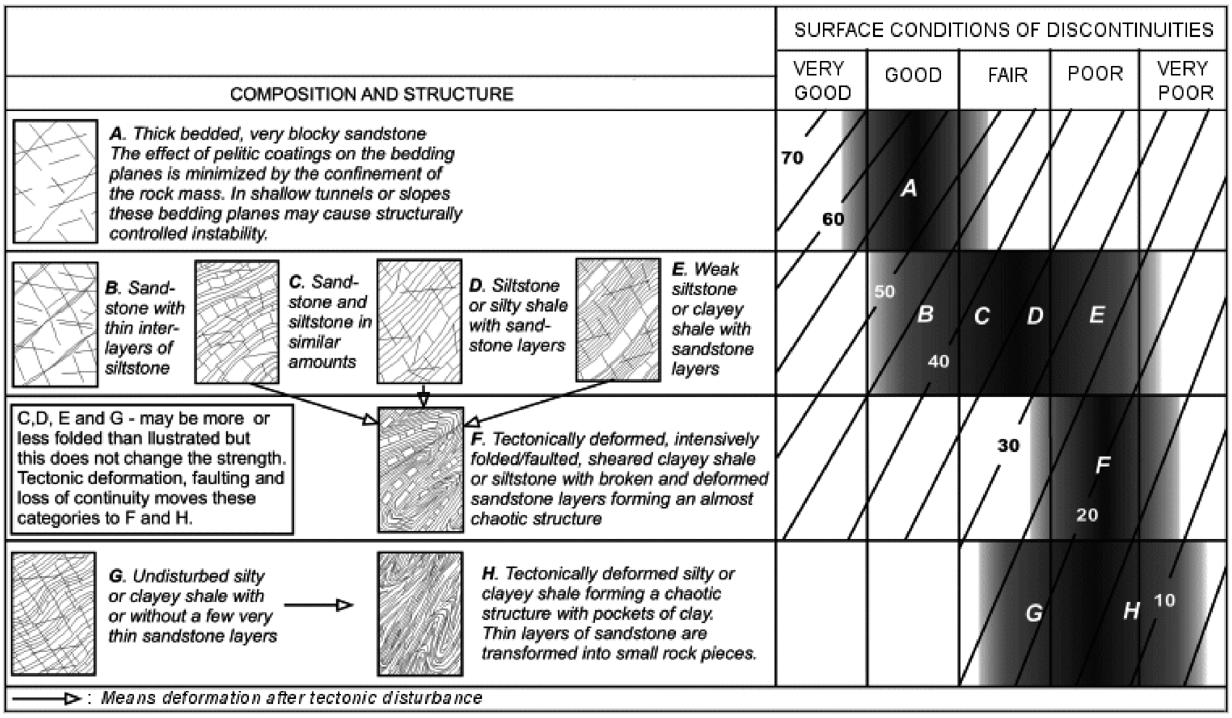
Values of the constant m_i for intact rock by group. Note that values in parenthesis are estimates.

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerates (21 ± 3) Breccias (20 ± 2)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-clastic	Carbonates	Crystalline limestone (12 ± 3)	Sparitic limestones (10 ± 5)	Micritic limestones (8 ± 3)	Dolomites (9 ± 3)
		Evaporites		Gypsum (10 ± 2)	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
Metamorphic	Non-foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 29 ± 6		
	Foliated*		Gneiss 28 ± 5	Schists (10 ± 3)	Phyllites (7 ± 3)	Slates 7 ± 4
Igneous	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 3)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

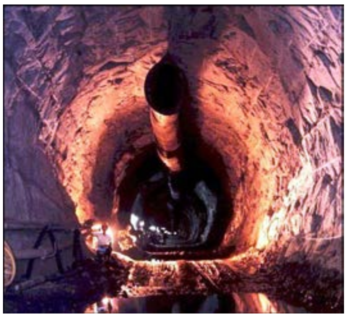
* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Selection values for GSI (estimates) [43].





Guidelines for estimating the disturbance factor *D*.

Appearance of rock mass	**Description of rock mass**	**Suggested value of D**
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	D = 0 D = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	D = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	D = 0.7 Good blasting D = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	D = 1.0 Production blasting D = 0.7 Mechanical excavation

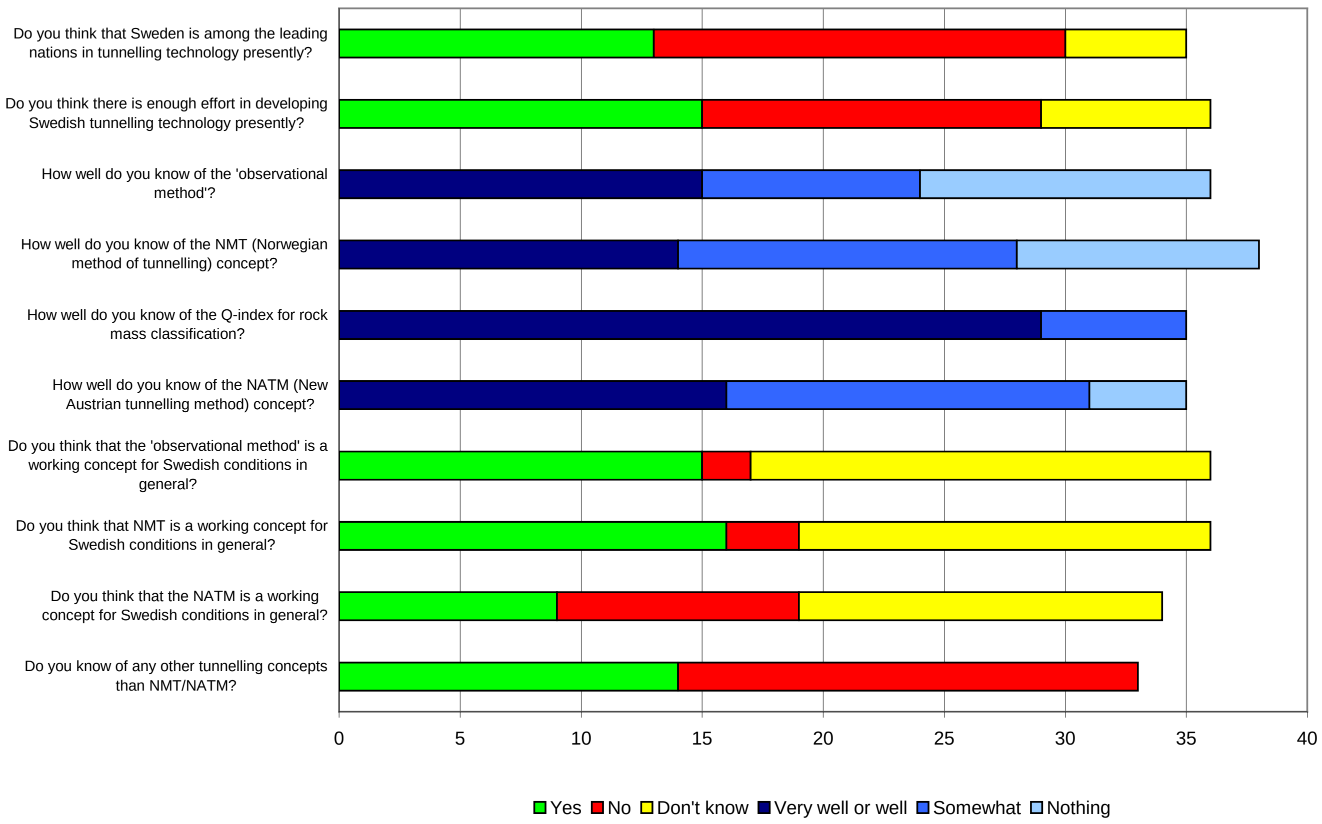
Appendix 8

Results and charts from the survey addressed to members of the Swedish society of engineering geology (BGS, Byggnadsgeologiska sällskapet) in February 2007.

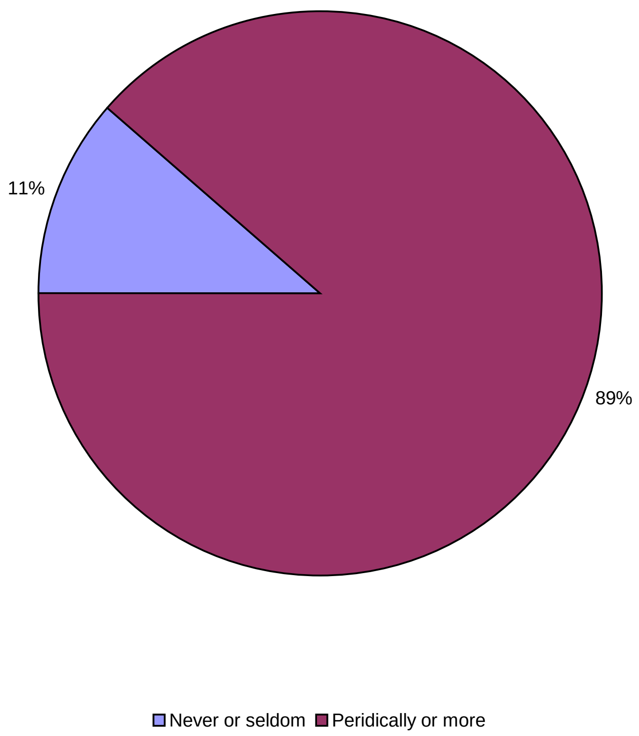
The questions:

No.	Question	Answers	Count	Percentage
1	In what area is your main occupation presently?	Civil engineering	8	21%
		Rock engineering	11	29%
		Engineering geology	9	24%
		Geology	2	5%
		University/college	2	5%
		Other	6	16%
2	How much do you work with tunnelling matters?	Full time	11	31%
		Daily	11	31%
		Periodically	9	26%
		Seldom	2	6%
		Never	2	6%
3	Do you think that Sweden is among the leading nations in tunnelling technology presently?	Yes	13	37%
		No	17	49%
		Don't know	5	14%
4	Do you think there is enough effort in development of Swedish tunnelling technology presently?	Yes	15	42%
		No	14	39%
		Don't know	7	19%
5	How well do you know of the 'observational method'?	Nothing	12	33%
		Somewhat	9	25%
		Well	10	28%
		Very well	5	14%
6	Do you think that the 'observational method' is a working concept for Swedish conditions in general?	Yes	15	42%
		No	2	6%
		Don't know	19	53%
7	How well do you know of the NMT (Norwegian method of tunnelling) concept?	Nothing	10	26%
		Somewhat	14	37%
		Well	13	34%
		Very well	1	3%
8	How well do you know of the Q-index for rock mass classification?	Nothing	0	0%
		Somewhat	6	17%
		Well	12	34%
		Very well	17	49%
9	Do you think that NMT is a working concept for Swedish conditions in general?	Yes	16	44%
		No	3	8%
		Don't know	17	47%
10	How well do you know of the NATM (New Austrian tunnelling method) concept?	Nothing	4	11%
		Somewhat	15	43%
		Well	12	34%
		Very well	4	11%
11	Do you think that the NATM is a working concept for Swedish conditions in general?	Yes	9	26%
		No	10	29%
		Don't know	15	44%
12	Do you know of any other tunnelling concepts than NMT/NATM?	Yes	14	42%
		No	19	58%

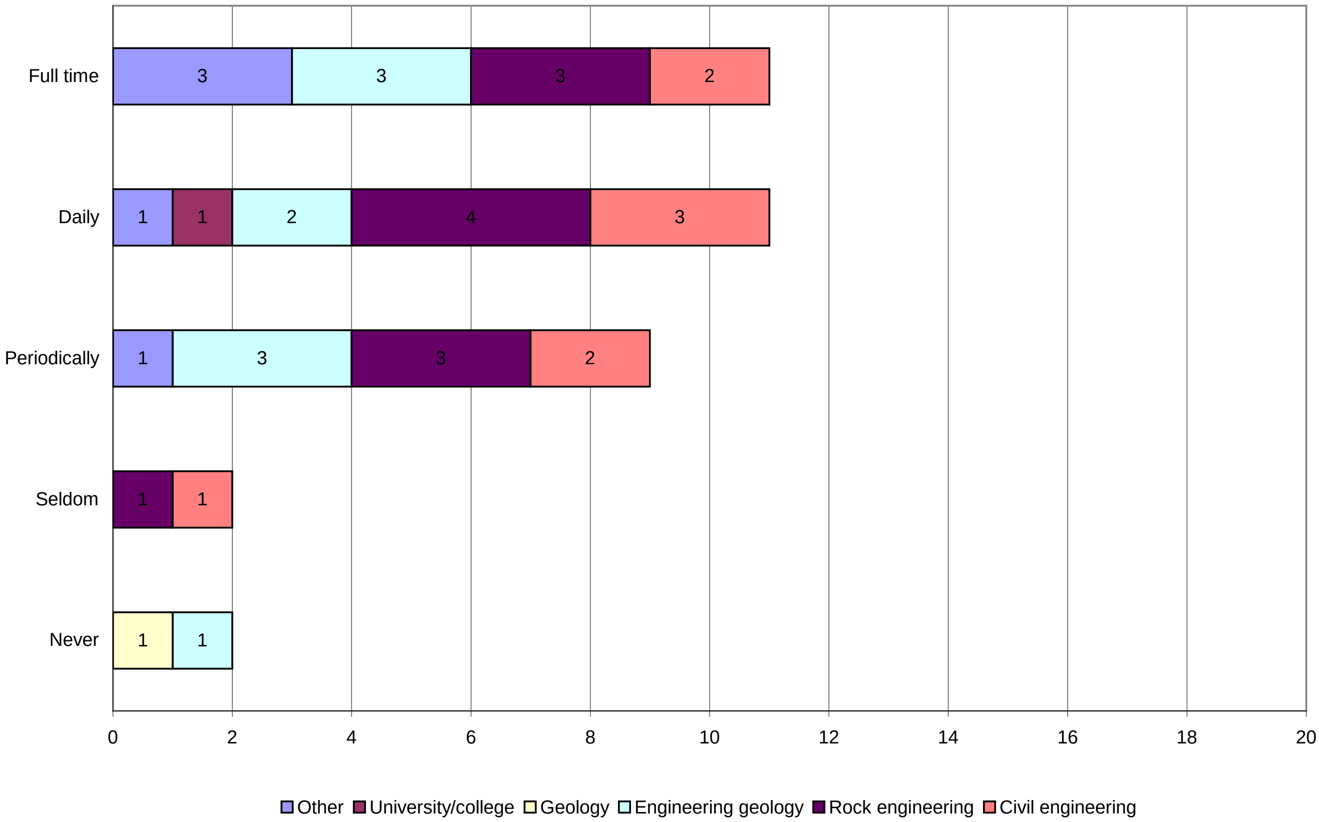
Summary of questionnaire responses.



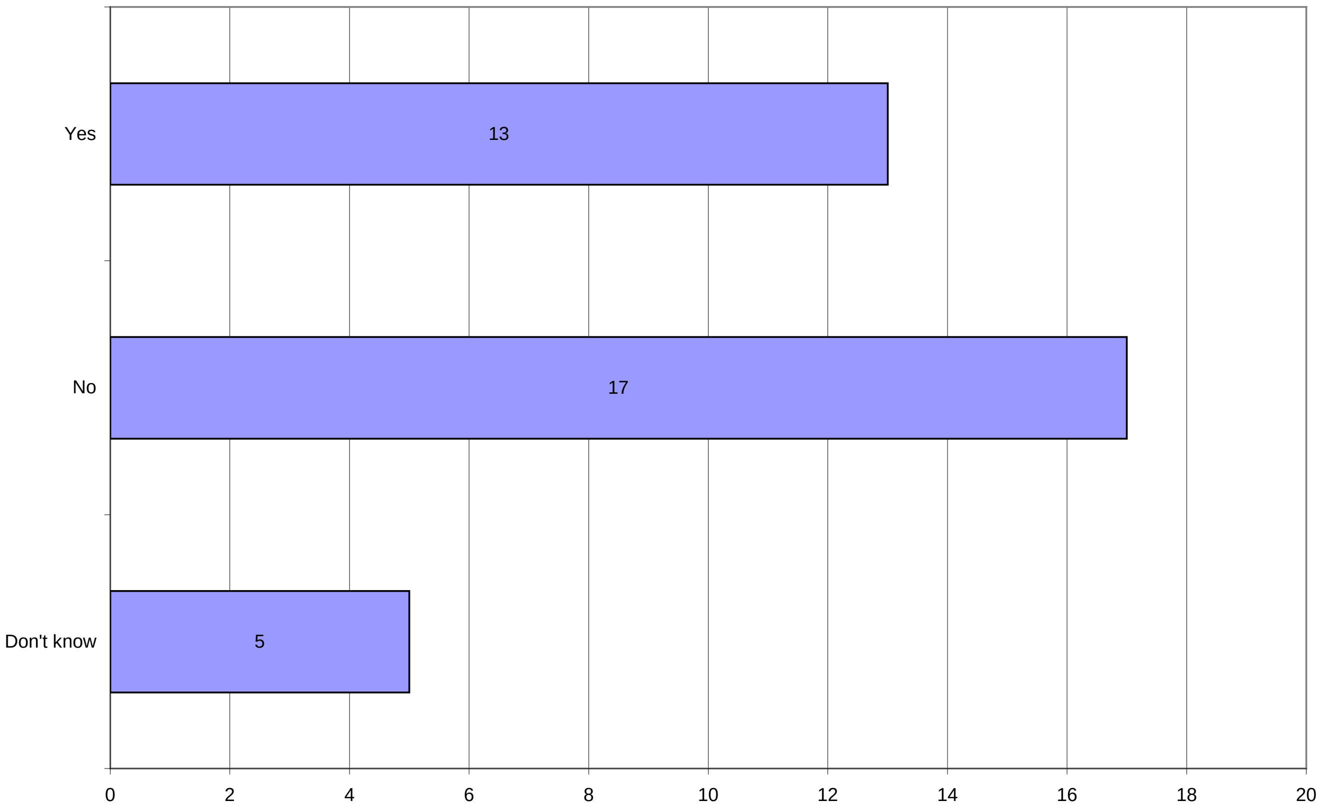
Question "How much do you work with tunnelling issues?" (2) categorised by work allocation (2).



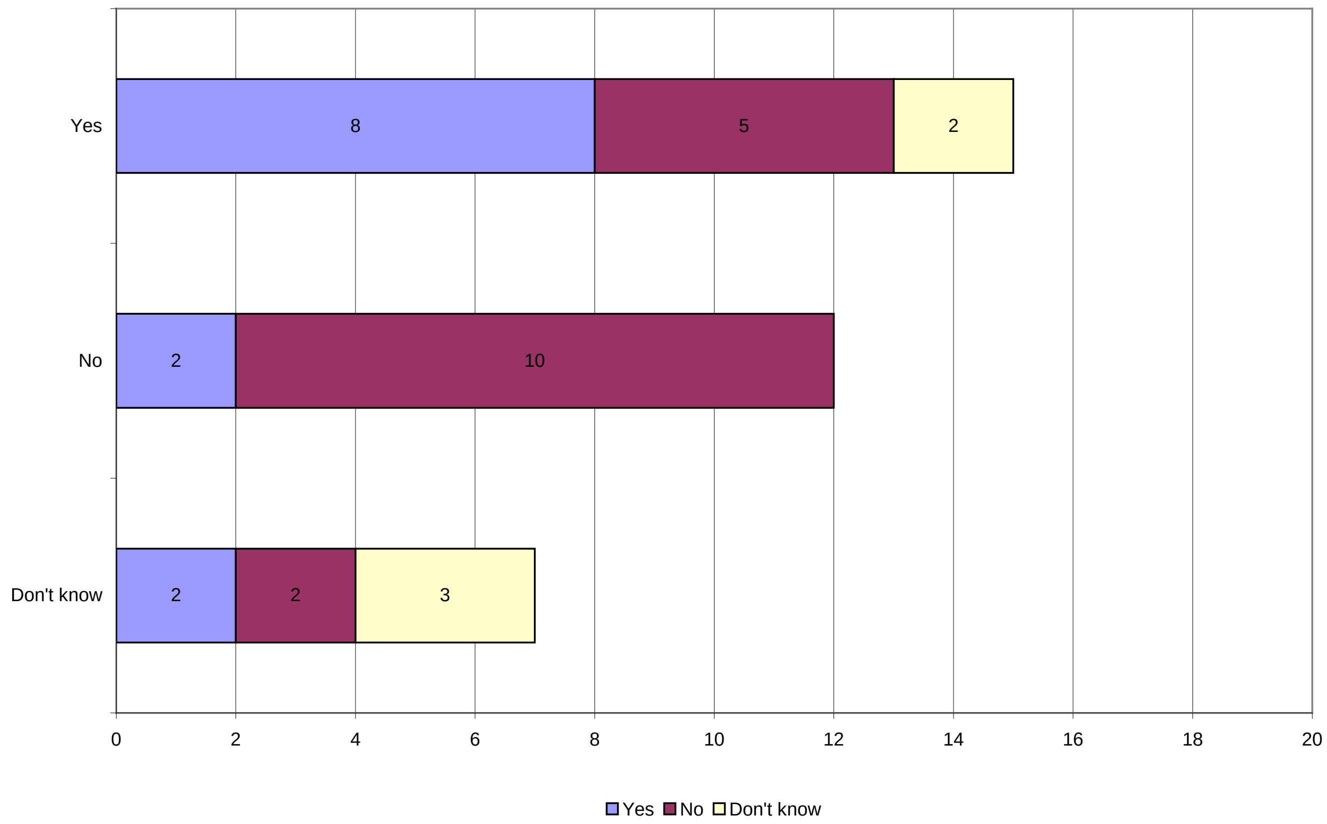
Question "How much do you work with tunnelling issues?" (2) categorised by occupational area (1).



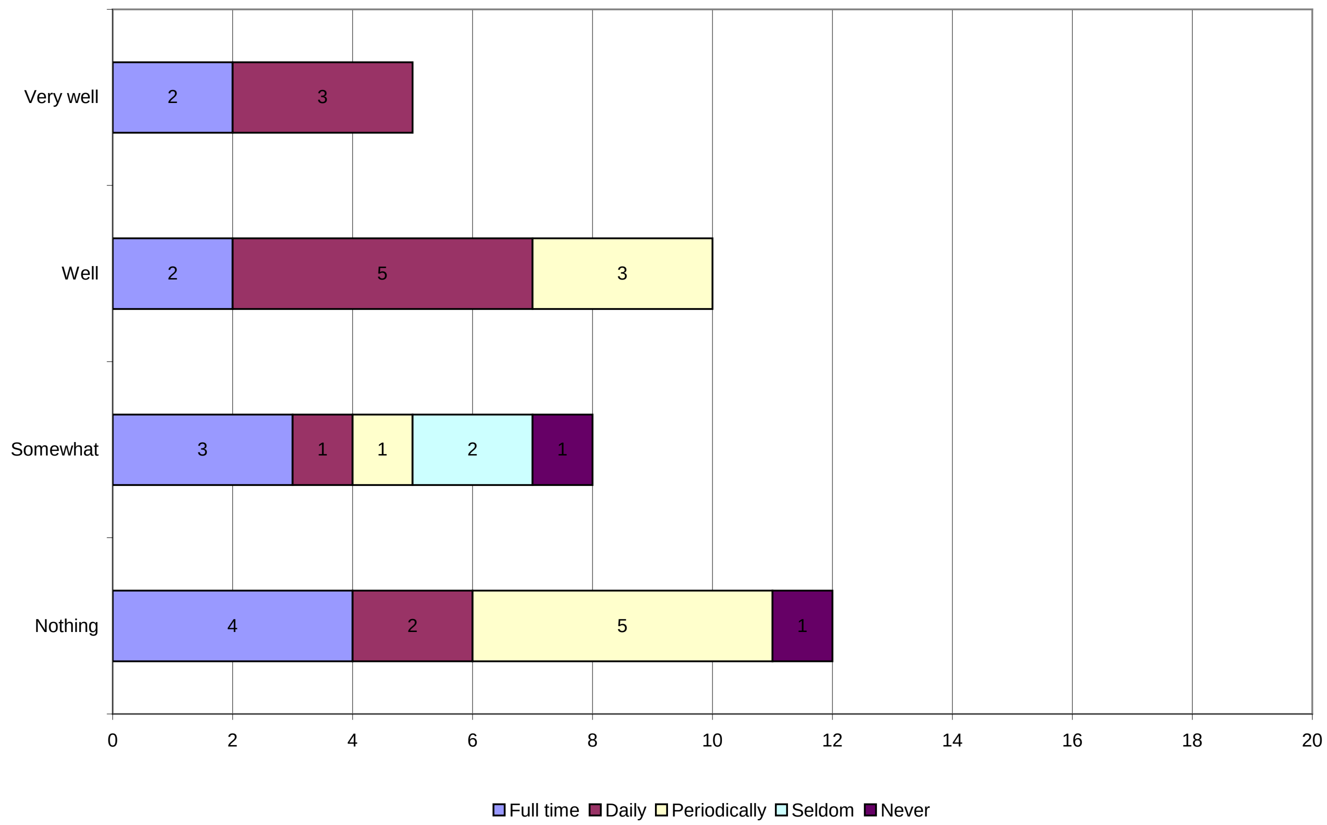
Question "Do you think that Sweden is among the leading nations in tunnelling technology presently?" (3)



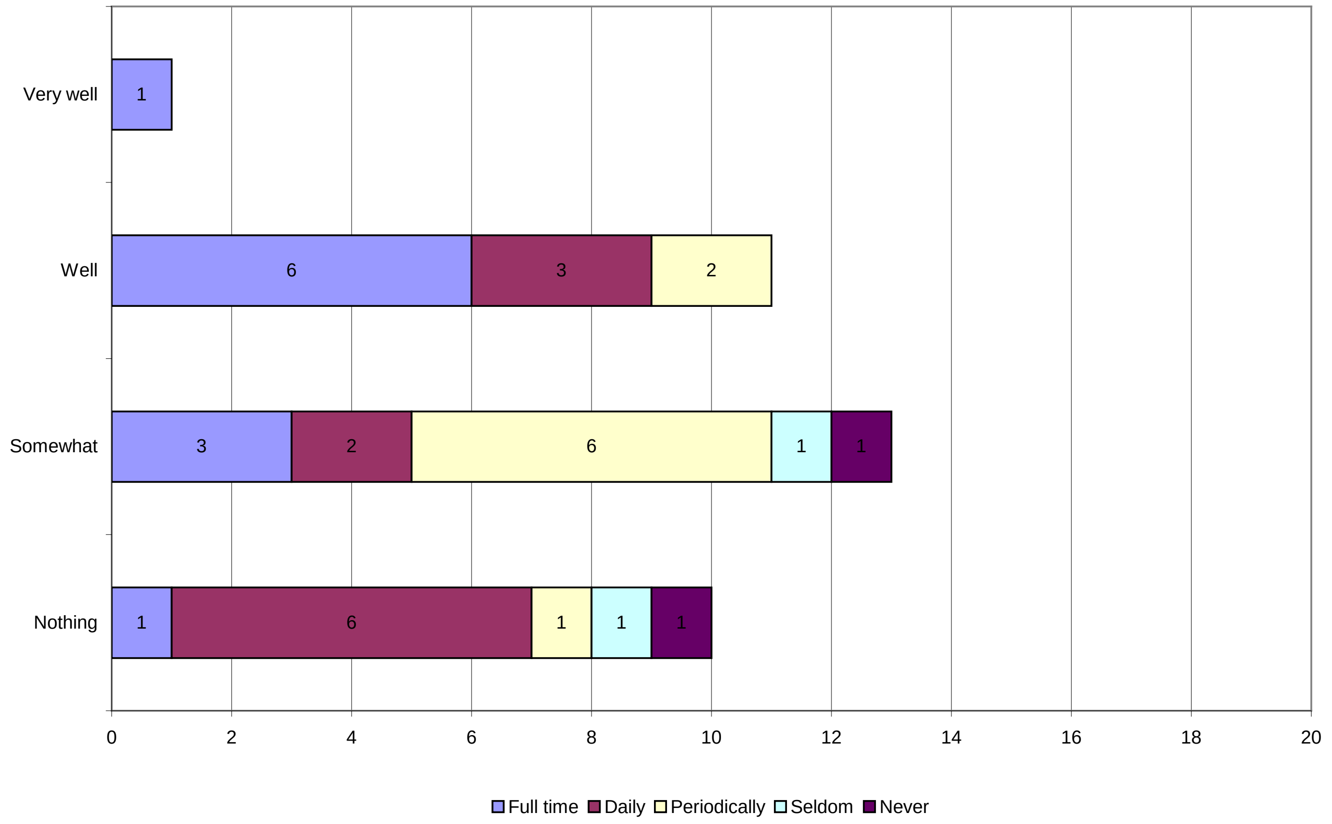
Question” Do you think there is any development of Swedish tunnelling technology presently?” (4).



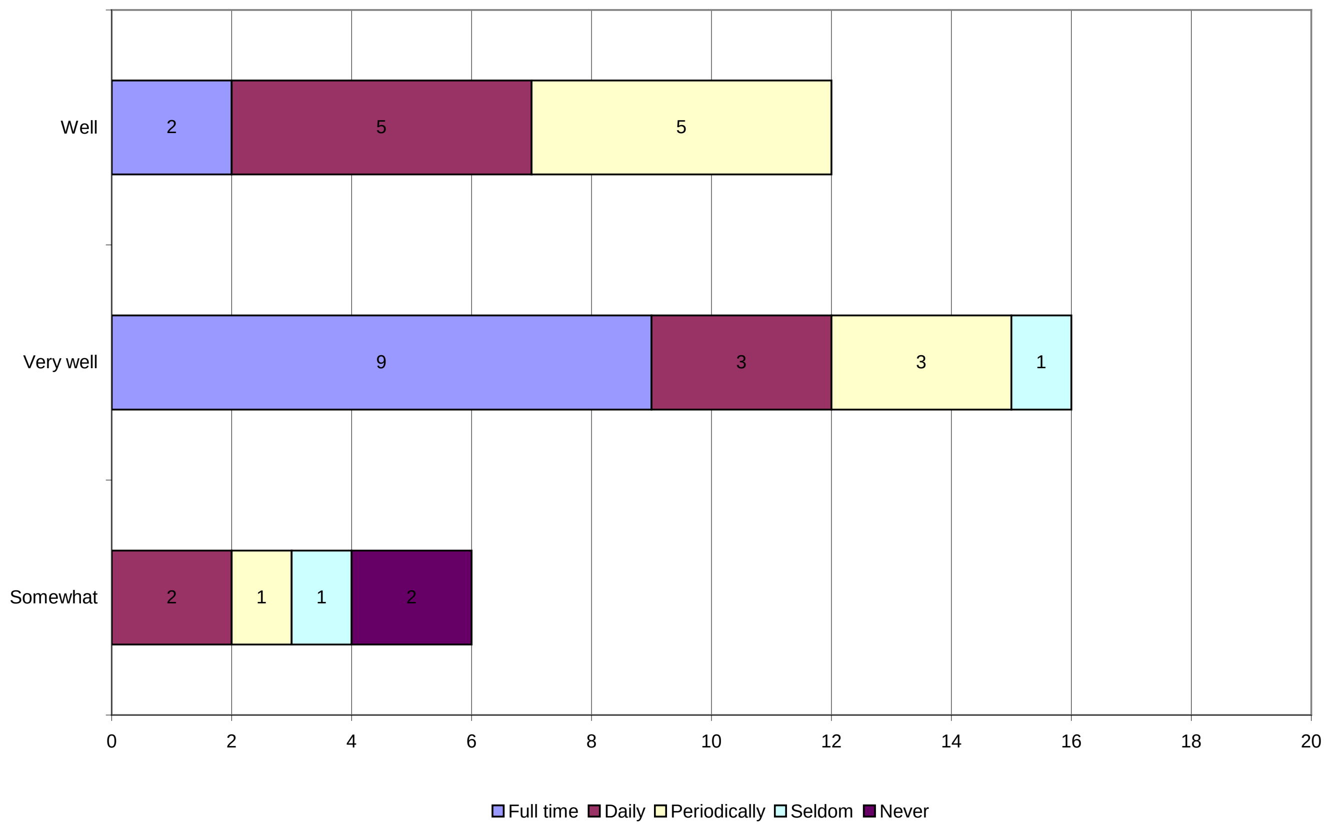
How well do you know of the 'observational method'? (5) categorised by work allocation (2)



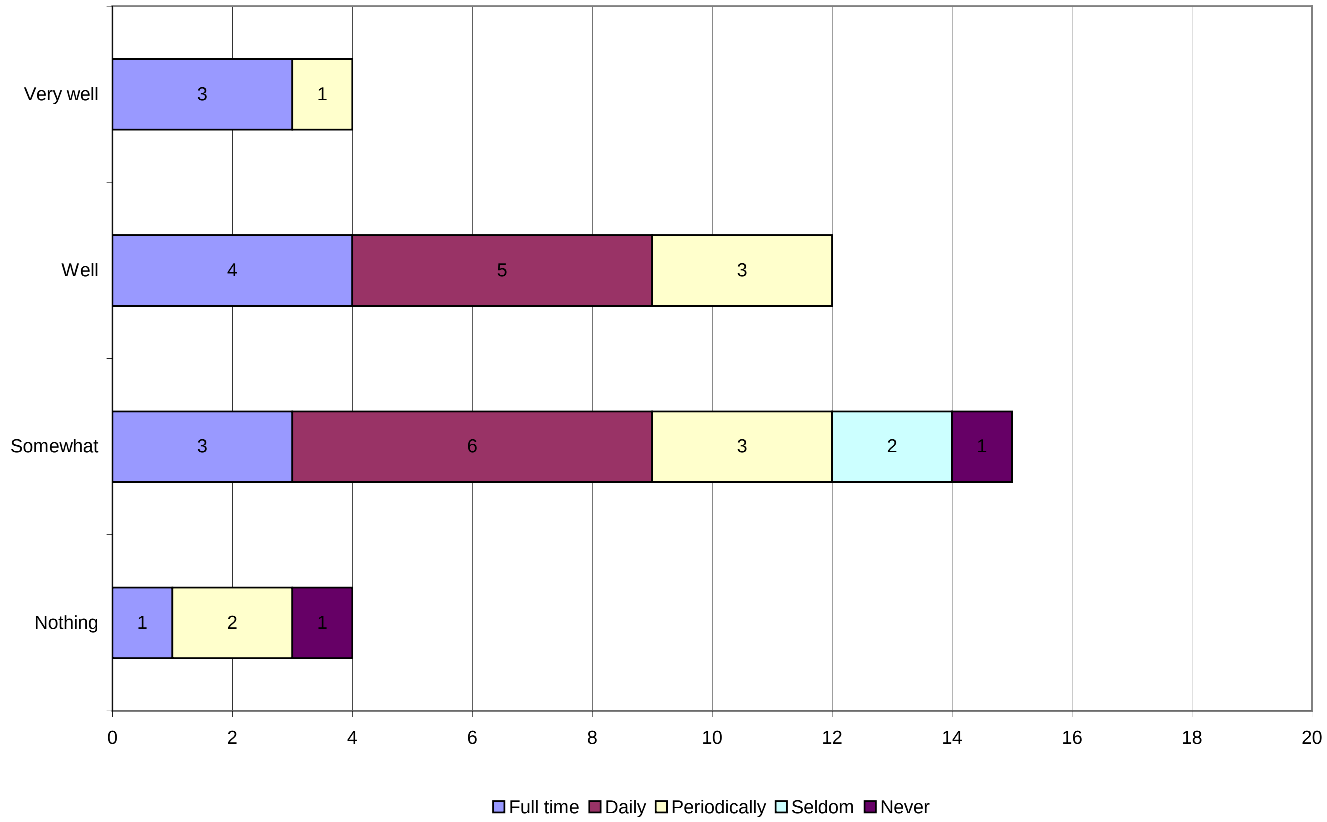
Question "How well do you know of the NMT (Norwegian method of tunnelling) concept?" (7) categorised by work allocation (2)



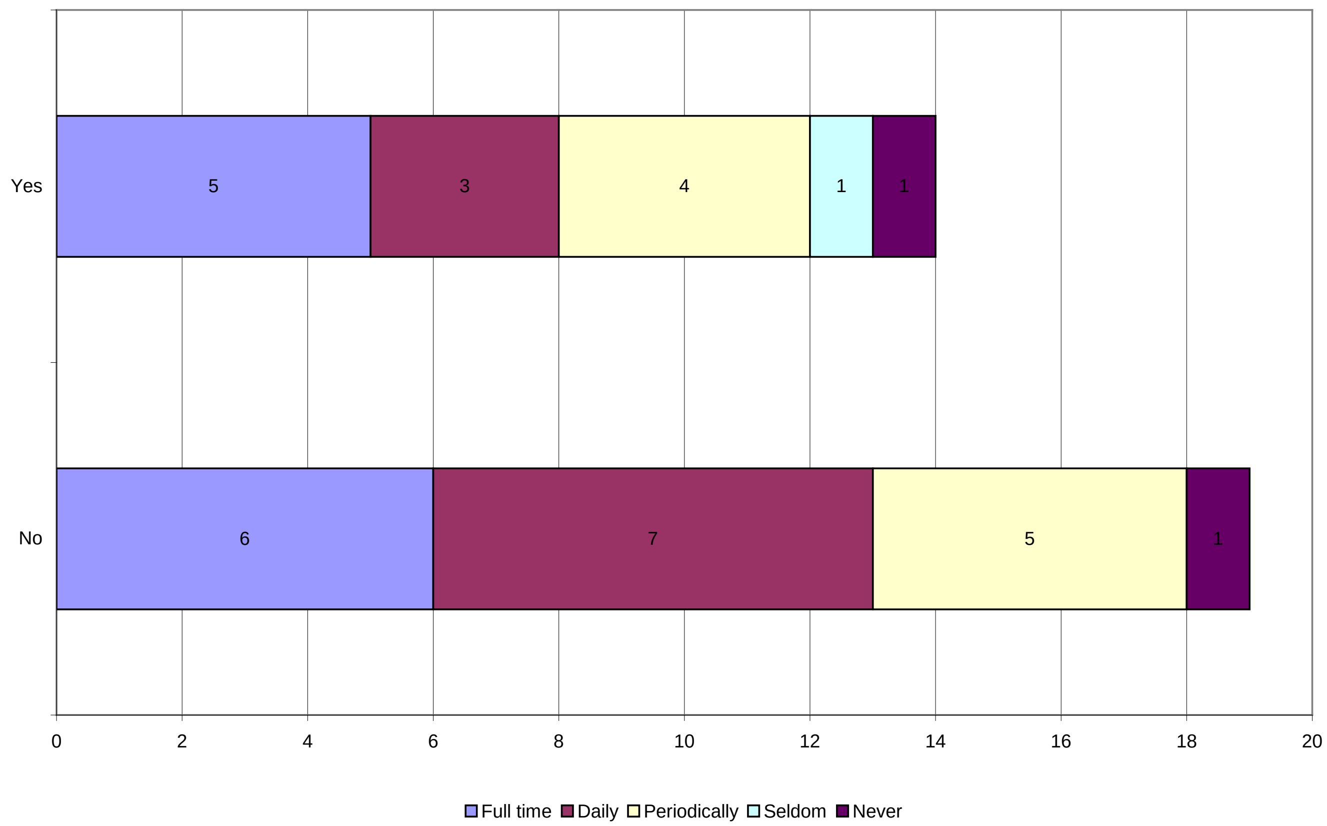
Question "How well do you know of the Q-index for rock mass classification?" categorised by work allocation (2).



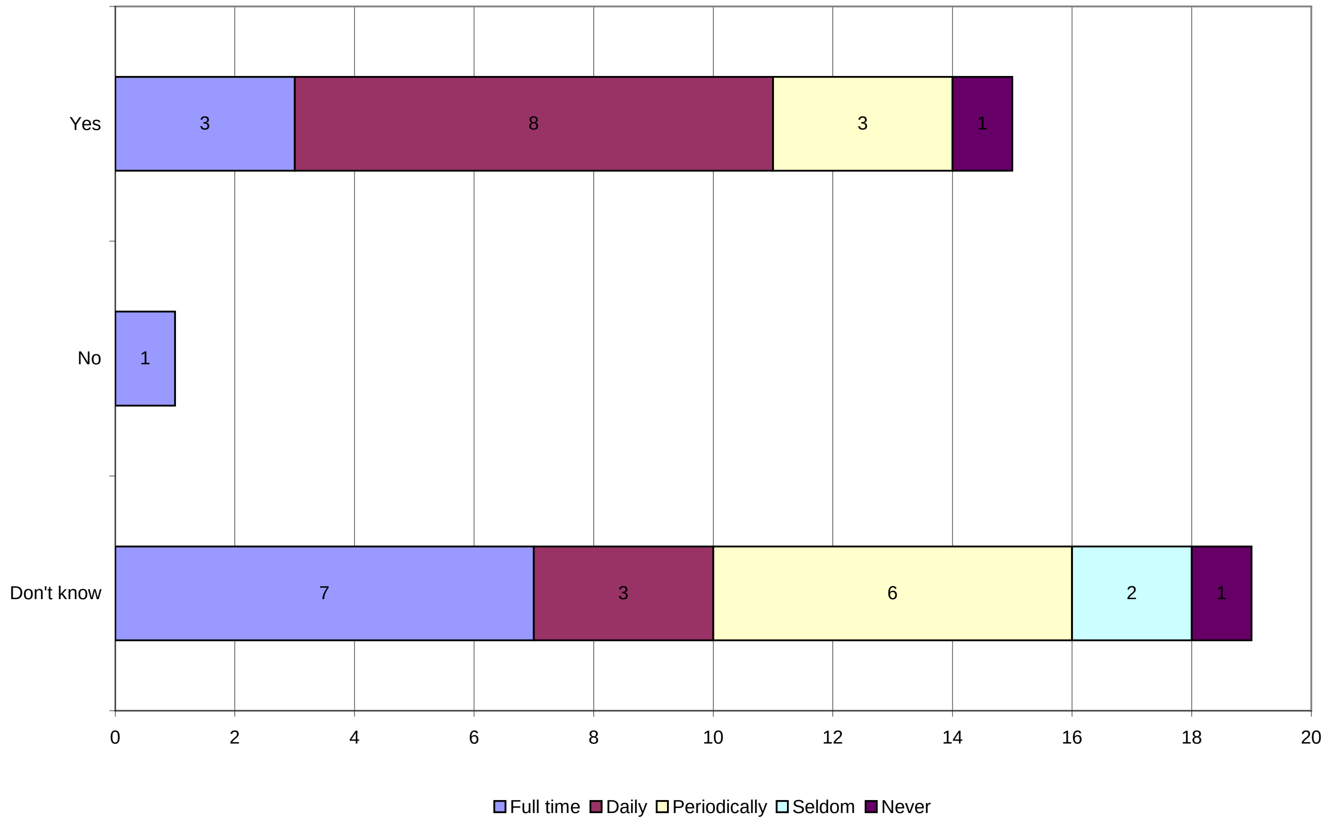
Question "How well do you know of the NATM (New Austrian tunnelling method) concept?" (10) categorised by work allocation (2).



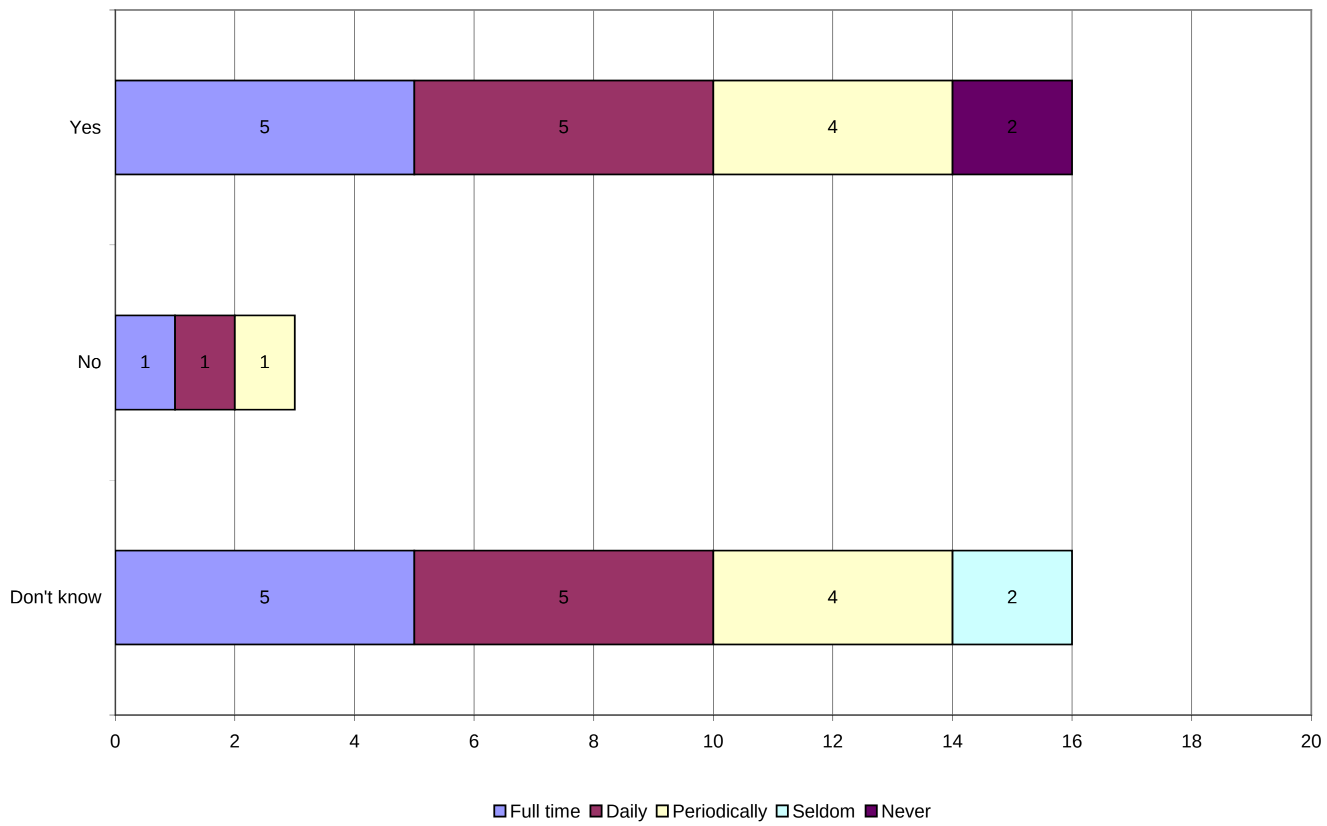
Question "Do you know of any other tunnelling concepts than NMT/NATM?" (12) categorised by work allocation (2).



Question “Do you think that the 'observational method' is a working concept for Swedish conditions in general?” (6) categorised by work allocation (2).



Question “Do you think that NMT is a working concept for Swedish conditions in general?” (9) categorised by work allocation (2).



Question "Do you think that NMT is a working concept for Swedish conditions in general?" (11) categorised by work allocation (2).