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Strategy for a Rock Mechanics Site Descriptive Model

Development and testing of the empirical approach

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Summary

This Report presents the results of one part of a wide Project for the determination of a methodology for the determination of the rock mechanics properties of the rock mass for the so-called Äspö Test Case. The Project consists of three major parts: the empirical part dealing with the characterisation of the rock mass by applying empirical methods, a part determining the rock mechanics properties of the rock mass through numerical modelling, and a third part carrying out numerical modelling for the determination of the stress state at Äspö. All Project's parts were performed based on a limited amount of data about the geology and mechanical tests on samples selected from the Äspö Database. This Report only considers the empirical approach.

The purpose of the project is the development of a descriptive rock mechanics model for SKBs rock mass investigations for a final repository site. The empirical characterisation of the rock mass provides correlations with some of the rock mechanics properties of the rock mass such as the deformation modulus, the friction angle and cohesion for a certain stress interval and the uniaxial compressive strength.

For the characterisation of the rock mass, several empirical methods were analysed and reviewed. Among those methods, some were chosen because robust, applicable and widespread in modern rock mechanics. Major weight was given to the well-known Tunnel Quality Index (Q) and Rock Mass Rating (RMR) but also the Rock Mass Index (RMi), the Geological Strength Index (GSI) and Ramamurthy's Criterion were applied for comparison with the two classical methods.

The process of: i) sorting the geometrical/geological/rock mechanics data, ii) identifying homogeneous rock volumes, iii) determining the input parameters for the empirical ratings for rock mass characterisation; iv) evaluating the mechanical properties by using empirical relations with the rock mass ratings; was considered. By comparing the methodologies involved by the application of different classification systems, advantages and disadvantages of each method could be highlighted. Moreover, a comparison of the results of the methods could be made, so that the differences in the output parameters were studied and explained. This comparison also allowed establishing a range of possible values of the output parameters. Through this process, a suitable determination method was chosen for each rock mechanics property required as outcome of the Project.

Based on the critical analysis and comparison of the different rock mass classification systems, a series of recommendations was provided concerning: i) the quality/quantity of the geological/rock mechanics input data; ii) the technique for partitioning of the rock mass in homogeneous domains; iii) the sensitivity and subjectivity of the empirical methods; iv) some limits of the methods; v) some warnings about difficulties, misleading techniques, and; vi) about the need of more studies for the validation of the empirical methods against results from case histories.

Sammanfattning

Denna rapport presenterar resultaten av en del i ett större projekt vars mål varit att ta fram en metod för att bestämma bergmassans egenskaper för det sk Äspö-testet. Projektet består av tre huvuddelar: en empirisk del som behandlar karaktäriseringen av bergmassan genom att tillämpa empiriska metoder, en andra del där de bergmekaniska egenskaperna bestämts genom numeriska modeller och en tredje huvuddel där spänningssituationen vid Äspö bestämts med hjälp av numerisk modellering. Alla projektdelarna är utförda och baserade på en begränsad mängd geologisk data och mekaniska tester på prover utvalda från Äspös databas. I denna rapport beskrivs den empiriska projektdelen.

Målet med projektet är att utveckla en beskrivande bergmekanisk modell för SKBs bergundersökningar för slutförvar. Den empiriska karaktäriseringen av bergmassan har syftat till att bestämma några av bergmassans bergmekaniska egenskaper så som deformationsmodulen, friktionsvinkeln och kohesionen för vissa spänningsintervaller samt den enaxiella tryckhållfastheten.

För karaktärisering av bergmassan har flera olika empiriska metoder analyserats och studerats. Av dessa metoder utvaldes några som är robusta, tillämpliga och frekvent använda i modern bergmekanik. Stor vikt lades därför på de välkända Tunnel Quality Index (Q) och Rock Mass Rating (RMR), men också Rock Mass Index (RMi), Geological Strength Index (GSI) och Ramamurthy's kriterium användes för att jämföras med de två klassiska metoderna.

Arbetsmetoden har omfattat följande moment: i) sortering av geometrisk/geologisk/bergmekanisk data; ii) identifiering av homogena bergvolymer; iii) bestämma indataparametrar för de empiriska metoderna för karaktärisering av bergmassan; iv) beräkning av mekaniska egenskaperna genom att använda empiriska relationer från karaktäriseringen.

Genom att jämföra metodologin, med tillämpning på olika klassificeringssystem, kunde för- och nackdelar för varje metod ingående studeras.

Jämförelse av resultaten från de olika klassificeringssystemen har också medfört att olikheter i parametervärden kunnat studerats och förklarats. Jämförelsen medförde också bestämning av en mängd möjliga parametervärden. Baserad på denna process valdes en lämplig metod för bestämning av varje bergmekanisk egenskap, vilket var ett krav som slutresultat.

Baserad på de kritiska analyserna och jämförelsen mellan de olika bergklassificeringssystemen har resultatet lett till en serie rekommendationer som omfattar: i) kvalitet/kvantitet på geologisk/bergmekanisk indata; ii) tekniken för att indela bergmassan i homogena domäner; iii) empiriska metoders känslighet och subjektivitet; iv) något om metodernas begränsningar; v) en del varningar om svårigheter, missledande teknik; vi) nödvändigheten av mera studier vad gäller validering av empiriska metoder mot resultat från utförda byggprojekt.

Contents

1 Introduction

1.1 Objectives

This progress report presents the evaluation of empirical methods for rock mass characterisation and classification based on empirical rock mass rating systems. The use of empirical rock mass rating systems is one part of the main project "Site investigation strategy for development of a Rock Mechanics Site Descriptive Model" /Andersson et al, 2002/. The two other parts comprises of the development of a theoretical and a stress model /Hakami et al, 2002; Staub et al, 2002/. The objective of the main project is to develop a combined rock mechanical model from those three models.

The basic scientific aim of the empirical approach using rock classification systems in the SKB Rock Mechanics-model project is to establish the methods and procedures for deriving representative mechanical properties, concerning strength and deformability, to characterize the rock mass quality during site investigation, the site selection stages and for the design, construction and performance assessment of the underground nuclear waste repositories.

An important, and also most challenging task of using empirical models for design and performance/safety assessment is to estimate the overall (equivalent, effective) properties of fractured rocks as continua for numerical modelling. Due to non-existence of closed-form solutions and difficulties in numerical homogenisation models, the rock classification systems, typically Q and RMR, are very often used as a means to estimate a first hand estimation of some mechanical properties (such as deformation modulus) and strength parameters (such as internal friction angles and cohesion) of the rock mass, based on basically engineering experiences and judgement. The applicability of such rating systems is not based firmly on basic laws of physics (such as conservation laws) and rigorously defined constitutive models in the frame of thermodynamics. The applicability is based on their successes in many real projects, without limitations such as existence of REV (representative elementary volume), reliable constitutive models for fractures and intact rocks and properly defined boundary/initial conditions, which are required for numerical homogenisation schemes. It is in this regard that the rock classification systems can be used as an empirical approach of homogenisation and up-scaling for deriving equivalent properties of fractured rocks.

The strategy for the development of this empirical methodology consisted of two parts. The first part included general review and understanding of the classification systems in use and evaluation of a methodology. In a second phase some rating systems were chosen for application on a selected part of Äspö i.e. Äspö Test Case (ÄTC). The Äspö Test Case includes using the methodology in two rock volumes. One model with volume of 500 x 500 x 500 m, and a smaller region near the Prototype repository area between the level of –380 to –500 m, called the Test Case area. This volume was subdivided into cubes with a dimension of 30 m.

The basic empirical rock classification systems of Q and RMR were primarily used. Complementary classifications systems, as RMi, were also applied to calculate rock mass properties and compare the results with the classical methods; both GSI and Ramamurthy's Criterion were also tested.

This report is a summary of the strategy and implementation procedures developed for characterising the mechanical properties of the rock mass using various classification systems. We also have given recommendations about how to collect and use the geological data, how to deal with uncertainties involved in the processes of collection and interpretation, and how to take into account the effect of rock stresses. We finally give an overview about the treatment of the data with statistical tools and about the reliability of the values of the properties obtained from the empirical relations.

The methodology for the Empirical Methods' part is applied for determining the mechanical properties of the rock mass at Äspö. In this exercise, called the Äspö Test Case /Hudson, 2002/, the classification systems are used as tools for determining those properties, and the results from the different classification systems are compared and discussed. This mirrors in the structure of the report that collects all relations for a certain mechanical property given by different classification systems under the same heading.

1.2 Short review of the classification systems

The aim for a classification system is to adequately and as simply as possible describe rock masses of various complexity. The system shall also include understandable and meaningful parameters that could easily be measured or determined in the field or from bore holes. Classification systems were developed to be used in estimating the tunnel support loads to be supported.

It is out of the scope in this report to give a deep review of the various approaches but a brief historical summary is given below and a more detailed description in chapter 2. Overviews can be found in the following books /Singh and Goel, 1999; Hoek et al, 1995; Bieniawski, 1989/. A key journal publication is given by /Hoek and Brown, 1997/.

One of the first and simplest rock mass classification system was proposed by /Terzaghi, 1946/, mainly based on physical model tests to be used for steel arch support. Other systems were proposed by /Stini, 1950; Lauffer, 1958/. A relationship between the engineering quality of the rock mass and the Rock Quality Designation (RQD) was proposed by /Deere, 1968/.

Later the CSIR classification system by /Bieniawski, 1973, 1976/ was introduced, later named RMR based on five parameters, strength of the intact rock, RQD (Rock Quality Designation), spacing of the joints, condition of the joints and ground water conditions. A sixth parameter accounts for the relative orientation of the joints with respect to the tunnel axis. Based on the first five parameters of RMR, /Stille, 1982/ designed an alternative classification system that also considers the number of joint sets in the rock mass (Rock Mass Strength, RMS) and was mainly used in Sweden.

The Q-index, a tunnelling quality index, is based on a large amount of case histories of underground excavation stability mainly in hard rock /Barton et al, 1974/. The system comprises of six parameters, which are divided into three groups that describe the rock mass block size, joint condition and active stress.

Both RMR and Q-index have for long time been applied for design of rock tunnels and excavations, estimation of ground support, choice of support system, selection of direction of tunnel axes, etc and a number of case histories have been published, however mainly for shallow excavations. Both systems provide a realistic assessment of the factors that influences the stability of the rock mass.

Recently /Palmström, 1995, 1996a,b/ has suggested the RMi –classification system based on a jointing parameter and the intact rock strength.

The rock mass properties as deformation modulus and rock mass strength, sometimes given as the uniaxial compressive strength, can be evaluated from the rating systems by empirical relations.

For determination of the rock mass strength, a criterion based on GSI, Geological Strength Index, was proposed by /Hoek, 1994; Hoek et al, 1995/. The GSI-value can also be obtained knowing the RMR of the rock mass.

/Ramamurthy, 1995/ suggested that the strength of the jointed rock mass and the deformations modulus can be determined from a joint factor. The strength and the deformation modulus of the rock mass are calculated through reduction factors applied to the uniaxial compressive strength and Young's modulus of the intact rock respectively.

Both RMR and Q-index have been correlated with the seismic P-wave velocity in the rock mass and both the deformations modulus and strength of the rock mass can be indirectly determined from the geophysical data. However, the correlation might be site specific so care must be taken.

Besides the presented rating systems above, there are several others more or less in use and also empirical relations for determining the rock mass properties.

1.3 Databases of the RMR and Q systems

Because of their empirical nature, all the classification systems are based on databases of real case histories.

Database for the RMR System /Bieniawski, 1993/

In the version of RMR in /Bieniawski, 1989/ adopted in this work, 351 case histories were analysed. Among them, about 11% of the cases were in rock masses with 71<RMR<80 and totally about 16% with RMR>71 (Figure 1-1). The depth of the excavation was shallower than 150 m for about 43% of the cases and between 150 and 500 m in 45% of the cases.

The equation relating the deformation modulus of the rock mass with RMR was also determined based on conspicuous number of case histories (Figure 1-2, /Bieniawski, 1993/).

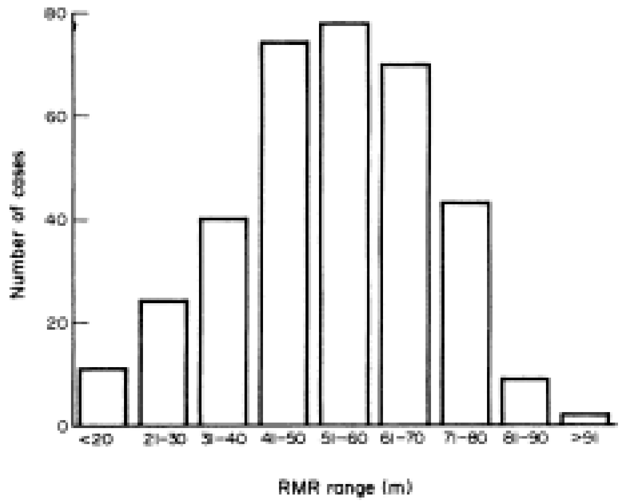


Figure 1-1. *Frequency distribution of the values of RMR in the case histories reported by /Bieniawski, 1989/.*

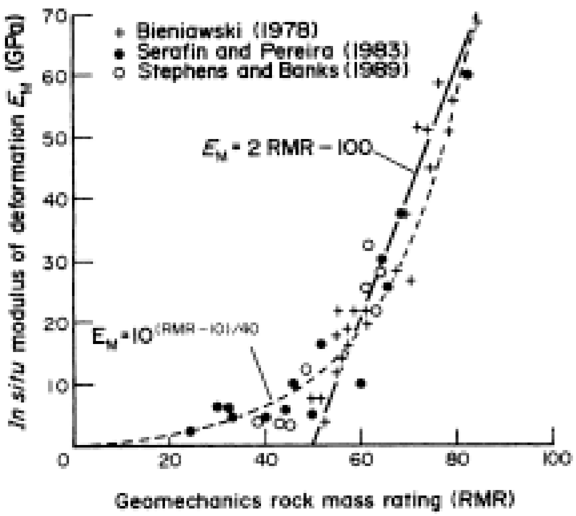


Figure 1-2. *Correlation between the in-situ deformation modulus of the rock mass and the Rock Mass Rating /Bieniawski, 1993/.*

Database for the Q System /Barton, 1988/

The Q-system by /Barton et al, 1974/ was based on 212 case histories. For about 50% of the cases the depth of the tunnels was smaller than 100 m, and for 34% of them between 100 and 500 m (Figure 1-3). For about 55% of the analysed cases, Q was in the range 1 to 40; 22% of the tunnels were excavated in granite and diorite. 1050 cases were later added to the Q-system database by /Grimstad and Barton, 1993/, and a new set of SRFs was then proposed.

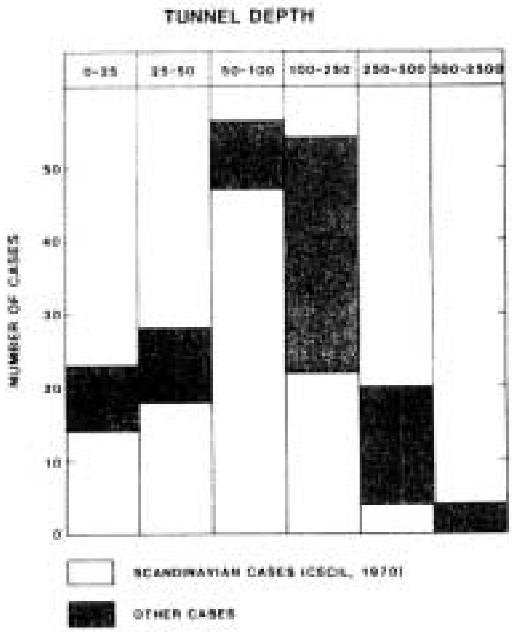


Figure 1-3. Frequency distribution of the tunnel depth for the 212 cases in the Q-system database /Barton, 1988/.

1.4 Recent application at Yucca Mountain

A circular tunnel through the Yucca Mountain was modelled by continuous and discontinuous models /Holland and Lorig, 1997/. The hoop pressure in the lining was considered as reference parameter for the comparison. The mechanical parameters of the intact rock were chosen as for the tuff at Yucca Mountain, while the properties of the fractures for the discrete modelling by UDEC were varied within certain assigned intervals. The pattern of the fractures was also changed so that 336 models were set up under seven stress boundary conditions. Under the same boundary conditions, continuous modelling by FLAC was carried out with parameters obtained from the rock mass characterisation by RMR (cohesion and friction angle from /Bieniawski, 1989; Hoek and Brown, 1980; Serafim and Pereira, 1983/. Models with RMR varying between 50 and 70 were considered (62<GSI<82).

The conclusions of the study were that:

- In most of the cases RMR gave reasonably conservative results except in cases were the boundary conditions or the fracture network caused the model to behave anisotropically. In those cases, RMR overestimated the numerical results;
- The stronger the rock mass, the closer the discontinuous and continuous model results were, and tended to converge to the elastic solution. For RMR>70 the authors found the effect of the joints negligible;
- The particular location of the fractures did not affects markedly the rock mass behaviour;

- The relation by Serafim and Pereira provides a good upper bound for the stresses in the liner obtained by discontinuous modelling;

- Bieniawski's recommendations for rock mass cohesion and friction angle are more conservative than the rock mass strength envelope proposed by Hoek and Brown, and both are more conservative than the discontinuous modelling;

- It was found that it is not the orientation of the tunnel axis with respect to the fracture sets, but the orientation of the fracture sets with respect to the direction of the major principal stress that influenced the hoop stress.

1.5 Classification for characterisation and design

The development of the various rock mass rating systems as described in Sec. 0 has been that the systems started with classification for use in design. Later also the systems have, by different modifications, been applied for characterisation during site investigations.

/Palmström et al, 2001/ in a discussion at the GeoEng2000 Workshop have presented a general approach for a clear definition of the terms characterisation and classification. The term characterisation should only be applied for the interpretation of the data for the site and site conditions. The term classification should be preferably used for the design of the excavation as the rating systems are design tools. A flow chart for rock mass characterisation and classification from /Palmström et al, 2001/ is presented in Figure 1-4. However, classification is also the act of applying the classification systems, thus in this Report instead of referring to classification we will often refer to design, so that the expressions "classification for characterisation" and "classification for design" gain their meaning.

The rock mass classification methods have been applied in rock mechanics and rock engineering for two main purposes:

a) CHARACTERISATION: The estimation of the physical properties of fractured rock masses has been performed using empirical relations between the indices of rock classification systems (e.g. Q, RMR, GSI, RMi, Ramamurthy's criterion) and some rock mechanical properties concerning deformability and strength. These properties have sometimes been used as rock mass parameters, without resorting to theoretical/numerical analysis methods for design or homogenisation/up-scaling methods. In this way, the characterisation is kept separated from design and design-related safety factors and construction solutions, geometry and techniques.

   An advantage of the empirical approach is that it is convenient to represent the variability of the rock mass properties. This can be done by statistically treating the ratings and/or the mechanical properties derived from the characterisation for determine ranges of variation and spatial trends. To achieve acceptable reliability of the results, it is important that enough data from surface and underground mapping and experimental measurement (both geological, geophysical and mechanical) are gathered so that a too pessimistic or optimistic evaluation of the rock conditions is avoided;

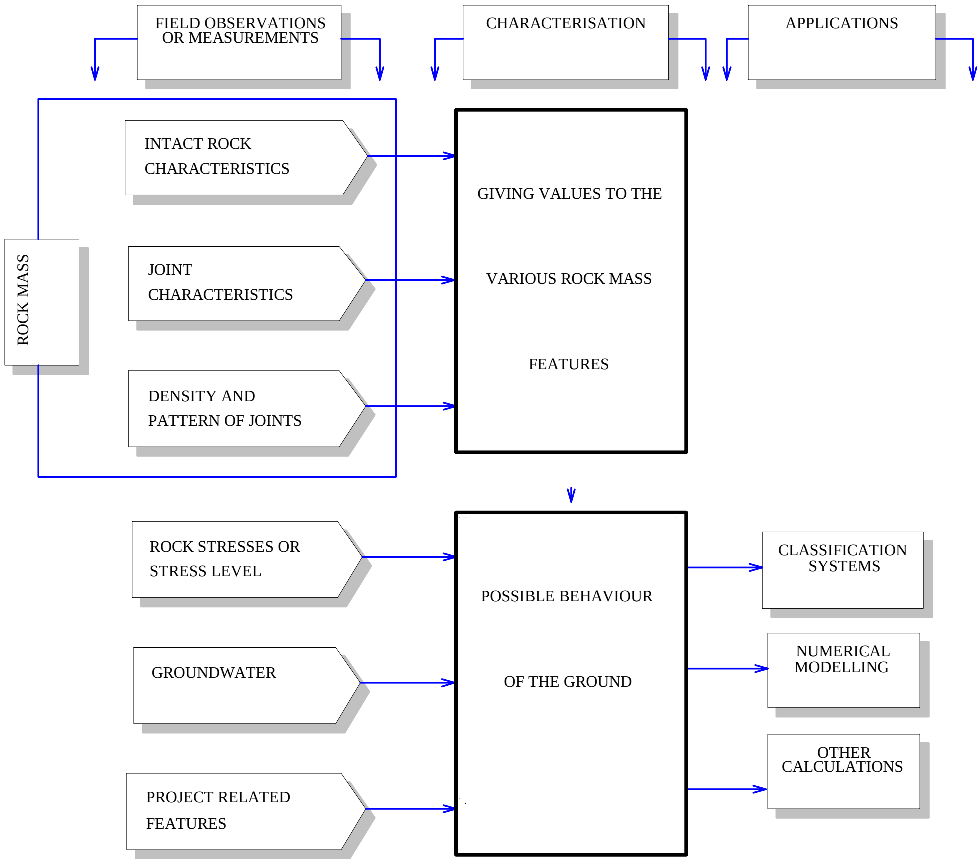


Figure 1-4. *Flow chart for rock mass characterisation and design /Palmström et al, 2001/.*

b) DESIGN: The rock classification systems were originally developed, and have been successfully applied, for design of rock engineering works, especially for tunnelling and underground construction, concerning dimensioning, layouting, and supporting. For design, it is important to know the local rock matrix and fracture conditions, and the geometry and orientation of the excavation. Irrespective of the excavation method and rock support, the properties of the best and worse encountered rock sections and the loading conditions (e.g. stress and water pressure), the design has to be reasonably conservative and cost effective. Thus, the classification of the rock mass and derived mechanical properties requires providing information about the most critical conditions with respect to construction technique, economy and safety /Palmström et al, 2001/. Through the classification some rock mass mechanical properties can be derived. They describe the near-field rock conditions at the scale of interactions with construction and include safety margins due to uncertainty, rather than evaluate the actual quality of the rock mass.

It is therefore important to note that requirements for rock classification for characterization and design are different, therefore require different treatment of parameter values and their weights to the overall rating indices. It is also important to note that, up to now, the main field of application of rock classification systems is design, not the characterization. The later started to appear in the rock engineering field more recently and with very limited number of case histories, due mainly to the fact that characterizing large scale rock masses in terms of mechanical properties became important only recently for large scale underground constructions of environmental importance, such as nuclear waste repositories, not for much smaller scale applications such as conventional tunnelling. The subject is relatively new and certain degree of risks about the validity and reliability of the methods and results must be taken in these regards.

1.6 Strategy for characterisation of rock masses

The strategy for the development of an empirical methodology comprised of two parts. The first part included general review and understanding of the classification systems in use and evaluation of a methodology. In a second phase some rating systems were chosen for application on a selected part of Äspö i.e. Äspö Test Case (ÄTC). The Äspö Test Case consists in applying the empirical methodology to two rock volumes. One model with volume of 500 x 500 x 500 m, and a smaller region near the Prototype repository area between the level of –380 to –500 m, called the Test Case area. This volume was subdivided into cubes with a length of 30 m. The basic empirical rock classification systems Q and RMR were primarily used together with complementary classifications systems like RMi, GSI and Ramamurthy's Criterion.

Besides the strategy and implementation procedures, we also have made recommendations on how to collect and how to use data, uncertainty treatment and effect of rock stresses.

2 Methods for rock mass classification

Some of the major rock classification systems, Q, RMR, GSI and RMi, have been well established in the field of rock engineering and were described systematically in a large number of books and articles, for both the principles, applications and developments. /Ramamurthy, 2001/ also proposed a more recent criterion and it is here considered because it has an independent background from the other systems. But first of all, it is important to explain how the rock mass at a site is conceptualised into a geological/geometrical model composed by rock units.

2.1 Input data and their treatment

The rock mass characterisation/classification is based on data from geology, rock mechanics, geophysics and hydrogeology collected from the field as well as from laboratory tests. The volume of input data will increase from the beginning of a site investigation to the final repository construction. The data points usually concentrate along boreholes and on surface mapping locations along tunnels.

2.1.1 Geological data

The rock types and structural features are the basis of subdivision of geological homogeneous domains, which is the first step required to perform characterisation/classification of the rock masses.

Geological data varies according to measurement techniques. The surface mapping depends on available outcrop areas, the borehole logging depends on the number, location and length of the available boreholes, and geophysical data depend on the available profiles of geophysical measurements.

The rock mass classification systems were developed by using tunnel/surface mapping data but have also been applied using borehole data. These approaches have respective limitations and advantages. The surface mapping gives more reliable information about fracture trace length than the other two techniques. Borehole information gives a continuous logging of the fracture frequency, fracture surface characteristics and orientation, but less information about trace length. Oriented diamond-drilled boreholes should be used to have acceptable quality of data for fracture set delineation and examination of fracture conditions. Tunnel mapping improves the determination of fracture set orientation, but gives limited improvement of the data for fracture trace length because of the limited dimensions. The best solution is to combine data from surface/tunnel mapping and core logging data. Even using such combined loggings, it is still very difficult to establish correlations between rock conditions at the surface and with depth.

The current practice for geological mapping and core logging should be improved for the needs of rock mass classification/characterisation. Special attention should be paid to quantify fracture properties, such as roughness, aperture, weathering degrees, fillings, etc, as they play a dominant role in all classification systems. Quantitative descriptions provide more objective determination of the ratings of the classification systems. If quantitative description is not possible, then qualitative description of the fracture conditions according to rock classification systems should be adopted.

2.1.2 Rock mechanics data

The classification systems normally include rock mechanics parameters as intact rock strength and there are correlations between the empirical ratings, field and laboratory observations. It is of importance that measured data are incorporated in an investigation systematically in order to be used as check points of the ratings but also for the decision of a certain parameter. Simple test devices can be applied in the field, as it is more important to collect more data with less accuracy than a few measurements with high accuracy. Complementary laboratory tests are used for checking and if special parameters are needed.

Simple test devices are point load tester for strength determination on cores or lump samples, Schmidt hammer for strength tests on surfaces and devices for estimation of roughness on fracture planes. The friction angel on fractures may be determined by tilt tests. The last test needs normally a core.

For the design stage it might be of importance to have a better understanding of the fracture parameters from the various sets as strength characteristics, stiffness and friction angle. In such cases separate samples must be taken and the tests performed in a large shear testing device under controlled conditions.

More advanced tests can be performed in the borehole and of special interest is the determination of the deformation modulus of the rock mass, which can be evaluated from various types of pressiometer tests.

The rock mass absolute principal stresses must be determined. The stress magnitudes are important and used in the Q-system. The orientation is important for the design in order to orient the rooms properly. The stresses should be measured and calculated with depth as many of the rock mechanical parameters are stress dependent.

2.1.3 Geophysical data

The geophysical data are unfortunately a limited source for rock mass rating, but are used for evaluation of the elastic parameters, the fracture intensity and the Q-system and RMR. The dynamic rock mass parameters can be evaluated if the P-wave and S-wave velocities are known.

There are only very few correlations made between geophysical data and rock mass ratings. Q-index has been correlated with the P-wave velocity, therefore it can be used as an alternative method for Q-value determination.

It is recommended that rock mass rating based on the geophysical data should be cross checked with ratings evaluated from geological information within the same area and that the geophysical ratings are used for extrapolation between outcrops and also as a help for checking homogeneity in areas where there are sparse outcrops, and for strength properties for checking the spatial variability.

2.1.4 Hydrogeological data

The effect of groundwater is important for the characterisation and design. Therefore, groundwater pressure and/or the flow rate are important parameters included in the rating systems of RMR and Q. In our opinion, the effect of water, either by pressure or flow rate, on the mechanical properties of rock masses can only be objectively considered based on properly formulated constitutive laws of fractured rocks based on thermodynamics. It is thus difficult to consider the water effects on mechanical properties of rock masses by classification systems.

2.2 Conceptualisation of the rock mass – The Rock Unit System

The first step for any rock classification system is the division of the rock units of qualitative lithological and structural homogeneity, which will be delineated using the main geological and geometrical information (Figure 2-1). The rock mass is divided into a number of units by the following structural features and mechanical properties:

- Lithological Contact zones that divide the rock mass into a number (N) of Rock Formations, in both vertical and horizontal directions. This division is given by geological model of the particular site.

- Major Fracture Zones (D1-D2) larger than 500 meters in length that divide each Rock Formation into a number (K1…KN) of basic Rock Units (U1-1…UN-KN), in both vertical and horizontal directions. This division in Rock Units is also a direct input from the geological model.

- Fracture Zones, due to their large size and possible large width, with probably complex internal structural, mineralogical and mechanical compositions and properties, are treated as independent basic units. The geometry of the Fracture Zone is also given by the geological model.

- Major differences in fracture density or fracture set number might make necessary to divide individual Rock Units into Subunits by Subunit Boundaries (SD1…SDJ). These can be identified by analysing the borehole logging data (RQD, set number and orientations along depth or borehole length, surface and shaft mapping results, DFN data at the test area). This is a subjective measure of choice based on intuitive understanding and experiences typical for rock classification systems.

- Major differences in representative mechanical properties of rock matrices and fractures (such as the uniaxial compressive strength of the intact rock, σ_c, and the residual friction angle of the fractures, ϕ) may also produce the division of basic Rock Units into Subunits.

- Differences in the state of stress can also introduce new boundaries between the Rock Units. These boundaries can identify zones with homogeneous stress constrain about a certain nominal stress level or with the same spatial law of variation of the stresses.

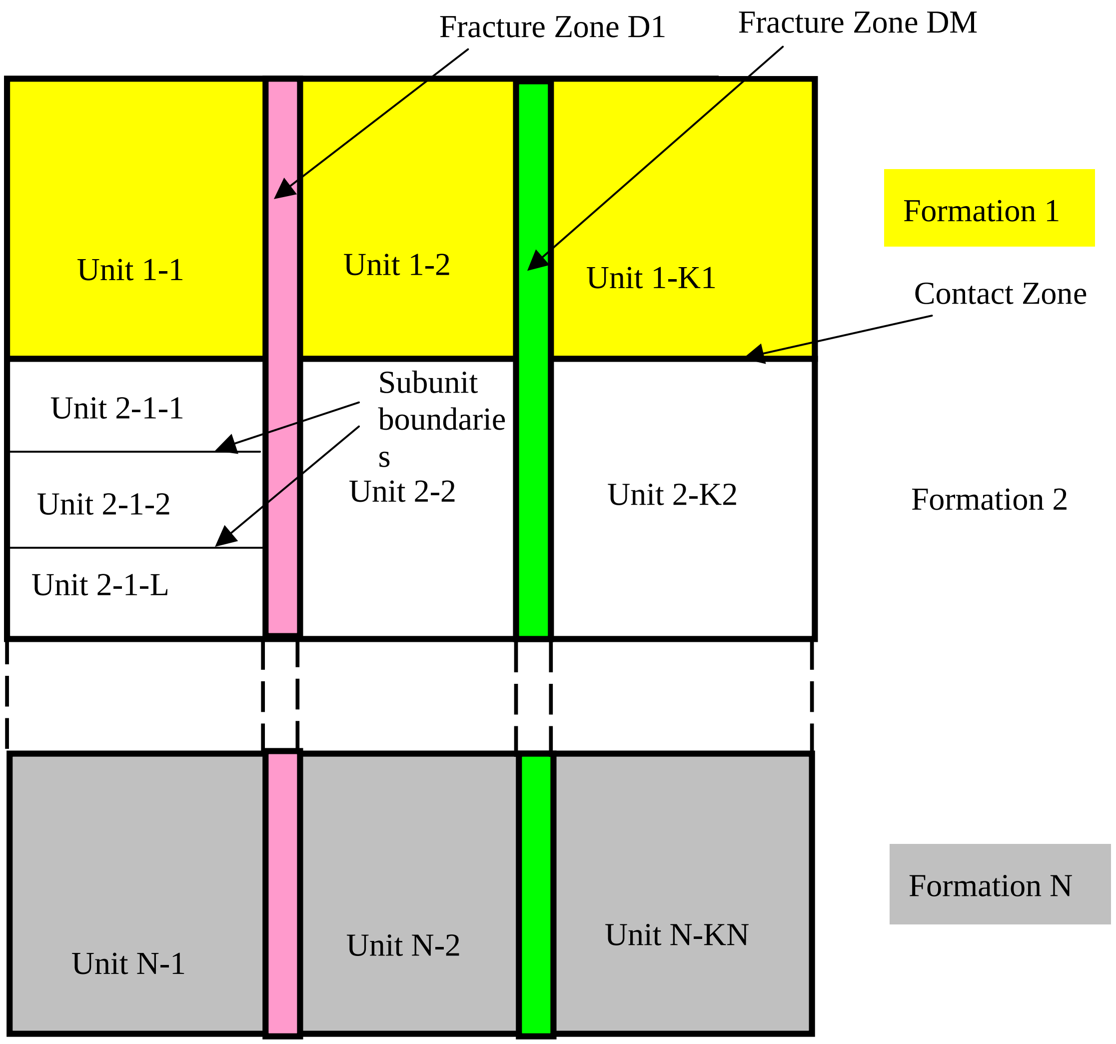


Figure 2-1. *The conceptual geological model of the site by Rock Unit System by dividing the rock mass into Rock Units (U1-1…UN-KN), Lithological Rock Formations (1…N), major Fracture Zones (D1…DM) each of them with rather homogeneous fracture properties (fracture set number, RQD, roughness, aperture, etc) and mechanical properties of rock matrix (E, ν,* σ_c *).*

The above unit delineation will divide the rock mass of the site into a number of working units (basic Rock Units and Subunits) of homogeneity in terms of lithology, structure, main mechanical properties and sometimes stress levels. These Units will serve as the objects for implementing the empirical model using Q, RMR, GSI and RMi rating systems and Ramamurthy's Criterion.

2.3 Main rock mass classification systems

Among the existent characterisation and classification systems, some were chosen for their historical value, robustness and widespreading. A classification based on RQD is illustrated for is its simplicity and because it constitutes the base of the Q- and RMR-system. The Q- and RMR-system are also described since they are the most used in rock engineering practice. Much literature is available on these two systems that were created in mid 70'. More recently, GSI, RMi and Ramamurthy's criterion were developed either as evolution of the former classification systems or as new concepts.

2.3.1 RQD and the engineering quality of the rock mass

/Deere, 1968/ proposed the classification of the rock mass quality based on RQD described in Table 2-1. This classification can be useful for identifying roughly homogeneously fractured rock on which to apply the other classification systems.

Table 2-1. Engineering classification of rock mass quality according to /Deere, 1968/.

RQD	Rock mass quality
90–100	Excellent
75–90	Good
50–75	Fair
25–50	Poor
<25	Very poor

2.3.2 Tunnelling Quality Index (Q-system)

The Q-classification system developed first by /Barton et al, 1974/ is given by the relation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{1}$$

where the rating of the parameters are:

RQD (0–100%) – Rock Quality Designation;
J_n (0.5–20) – Joint set number;
J_r (0.5–4) – Joint roughness number;
J_a (0.75–20) – Joint alteration number (related to friction angle);
J_w (0.05–1) – Joint water reduction number;
SRF (1–400) – Stress Reduction Factor.

The ratings of the Q-system for design have been updated and revised by /Grimstad and Barton, 1993/.

A subset of the Q, called the modified Tunnelling Quality Index, Q', was used in practice to characterize rock mass qualities without considering effects from water and stress, written as /Hoek et al, 1995/:

$$Q' = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \tag{2}$$

/Barton, 2001, personal communication/ has proposed that the Q-equation used for characterisation should have relevant values on SRF (low stress 0–25 m depth: 2.5; medium stress 25–250 m depth: 1.0; high stress 250–500 m depth: 0.5).

Sensible values for Jw for the Äspö Test Case at depth (450 m) are 0.5 and 0.66. Those are recommended for classification of competent rock. The rock mass rating based on Q is presented in Table 2-2.

Table 2-2. Classification of rock mass based on Q-value.

Q-value	Rock mass classification
400–1000	Exceptionally good
100–400	Extremely good
40–100	Very good
10–40	Good
4–10	Fair
1–4	Poor
0.1–1.0	Very poor
0.01–0.1	Extremely poor
0.0001–0.001	Exceptionally poor

2.3.3 Rock Mass Rating (RMR)

This rock mass classification method was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by /Bieniawski, 1973/. It was based on experiences on shallow tunnels in sedimentary rocks. A series of improvements, upgrades and modifications of this classification method has undergone during the years /Bieniawski, 1976, 1984, 1989/. Thus, it is important to add which version of the RMR geomechanics classification is adopted for a certain investigation site. The RMR-rating is given as the sum of ten components:

$$\begin{aligned} RMR = {} & RMR_{\substack{strength \\ of\ intact\ rock}} + RMR_{RQD} + RMR_{\substack{fracture \\ spacing}} + RMR_{\substack{fracture \\ lemgth}} + RMR_{\substack{fracture \\ weathering}} \\ & + RMR_{\substack{fracture \\ aperture}} + RMR_{\substack{fracture \\ roughness}} + RMR_{\substack{fracture \\ infilling}} + RMR_{water} + RMR_{\substack{fracture \\ orientation}} \end{aligned} \quad (3)$$

according to the RMR definition by /Bieniawski, 1989/:

$RMR_{\substack{strength \\ of\ intact\ rock}}$ (0–15) – Rating for intact rock strength using point load test index and σ_{ci} data from laboratory test results;

RMR_{RQD} (3–20) – Rating for RQD (from RQD <25% to RQD =90–100%);

$RMR_{\substack{fracture \\ spacing}}$ (5–20) – Rating for fracture spacing (spacing <60 mm to >2 m);

$RMR_{\substack{fracture \\ weathering}}$ (0–6) – Rating for fracture weathering condition;

$RMR_{\substack{fracture \\ length}}$ (0–6) – Rating for fracture length;

$RMR_{\substack{fracture \\ aperture}}$ (0–6) – Rating for fracture aperture (width);

$RMR_{\substack{fracture \\ roughness}}$ (0–6) – Rating for fracture roughness;

$RMR_{\substack{\text{fracture}\\ \text{infilling}}}$ (0–6) – Rating for fracture in-filling condition;

RMR_{water} (0–15) – Rating for groundwater (inflow rate (from 0 to 125 l/m) and pressure (from 0 to 0.5 of pressure /major principal stress ratio)). The inflow-rate rating needs tunnel, and may or may not be applicable. Pressure needs local or regional groundwater table information from hydro-geological information for rating;

$RMR_{\substack{\text{fracture}\\ \text{orientation}}}$ (–12–0) – Rating from very unfavourable to very favourable fracture orientation relative to tunnel orientation. Needs tunnel orientation for definite rating.

The rating and the classification with RMR is according to Table 2-3.

Table 2-3. Rock mass classification based on the RMR-value.

RMR rating	100-81	80-61	60-41	40-21	20-0
Rock class	I	II	II	IV	V
Classification	Very good	Good	Fair	Poor	Very poor

In case of non-uniform conditions, the “most critical condition” should be considered according to /Bieniawski, 1989/. In case two or more clearly distinct zones are present at small scale (e.g. tunnel front) through a unit to be considered homogeneous, then the overall weighted value based on the area of each zone in relation to the whole area should be considered.

2.3.4 Correlations between RMR and Q

Several empirical correlations between Q and RMR ratings have been reported in literature concerning case histories in Scandinavia, New Zeeland, USA and India. Some of those relations are listed below:

$RMR = 9 \ln Q + 44$ /Bieniawski, 1976/ (4)

$RMR = 5.9 \ln Q + 43$ /Rutledge and Preston, 1978/ (5)

$RMR = 5.4 \ln Q + 55.2$ /Moreno, 1980/ (6)

$RMR = 5 \ln Q + 60.8$ /Cameron-Clarke and Budavari, 1981/ (7)

$RMR = 10.5 \ln Q + 41.8$ /Abad, 1984/ (8)

$RMR = 15 \log Q + 50$ /Barton, 1995/ (9)

It should be noted that those correlations are only based on a statistical basis and their physical grounds are different. Caution should be taken when applying these relations for different rock conditions.

The first attempt of correlating RMR with Q values was carried out by /Bieniawski, 1976/ who analysed 117 case histories (68 in Scandinavia, 28 in South Africa, and 21 in USA). That study resulted in the classical relation in Eq. (4). Although this relation has been widely used in practice, several other relations were suggested in the following years. This depended on the fact that, not only such kind of relation is site sensitive, and thus not suitable for generalisation, but also that the two ratings are not equivalent because they take into account different rock mass parameters (e.g. uniaxial compressive strength of the intact rock and orientation of the rock fractures in the RMR system; and the stress influence in the Q system). The correlation should then be calculated between the reduced values of RMR (*RCR* with no intact rock strength and orientation rating) and the reduced Q (*N* with SRF=1)/Goel et al, 1995/. The relation between RCR and N was then obtained based on 36 case histories from India /Hoek and Brown, 1980/, 23 from /Bieniawski, 1984/, and 23 cases from /Barton et al, 1974/, as follow:

$$RCR = 8 \ln N + 30 \qquad (10)$$

This indicates that when subset of the classical classification ratings are considered, the relation in Eq. (7) does not necessarily apply (Figure 2-2), as it was demonstrated for the characterisation at the Äspö Test Case.

However, it is advisable not to rely on such ready-to-use relations, but to apply independently at least two classification systems and derive a site-specific correlation between the two, or even a simplified site-related characterisation system. In fact, the standardization of the classification system has been found often to be undesirable and impracticable /Bieniawski, 1988; Palmström et al, 2001/.

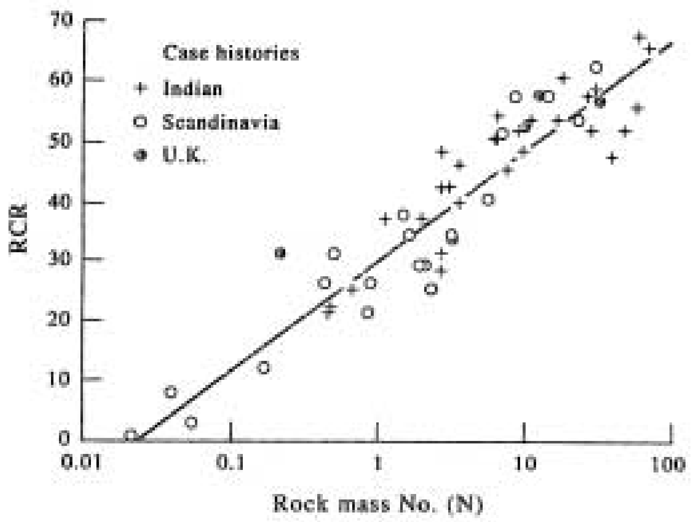


Figure 2-2. *Correlation between the reduced RMR (RCR) and Q (N) /Goel et al, 1995/.*

2.3.5 Geological Strength Index (GSI)

The Geological Strength Index (GSI) was introduced by /Hoek, 1994, 1995; Hoek and Brown, 1997/. GSI provides a strength index based on the geological conditions identified by field observations. The characterisation is based upon the visual impression of the rock block structure and the condition of the rock fractures (roughness and alteration). Based on the rock mass description, GSI is estimated from the contours in Figure 2-3.

A series of empirical relations were also proposed to relate the GSI-values with the strength /Hoek and Borwn, 1997; see Sec. 2.5.1/. A conversion equation between GSI and RMR (in the version proposed by /Bieniawski, 1989/) was also provided as:

$$GSI = RMR - 5 \text{ for } RMR > 23 \qquad (11)$$

where RMR is evaluated in dry conditions (rating for water = 15) and with favourable orientation of the tunnel with respect to the fracture orientation (rating for orientation=0) /Hoek et al, 1995/. GSI is related to the rock mass deformation modulus by empirical relations (c.f. Sec 2.5.3).

A feature of the GSI system is that it can be used for very preliminary estimations without quantitative data for geometrical and mechanical properties of rock and fractures other than the observational description of the block structure formations, which can be estimated readily from surface surveying at selected outcrops, without using borehole information.

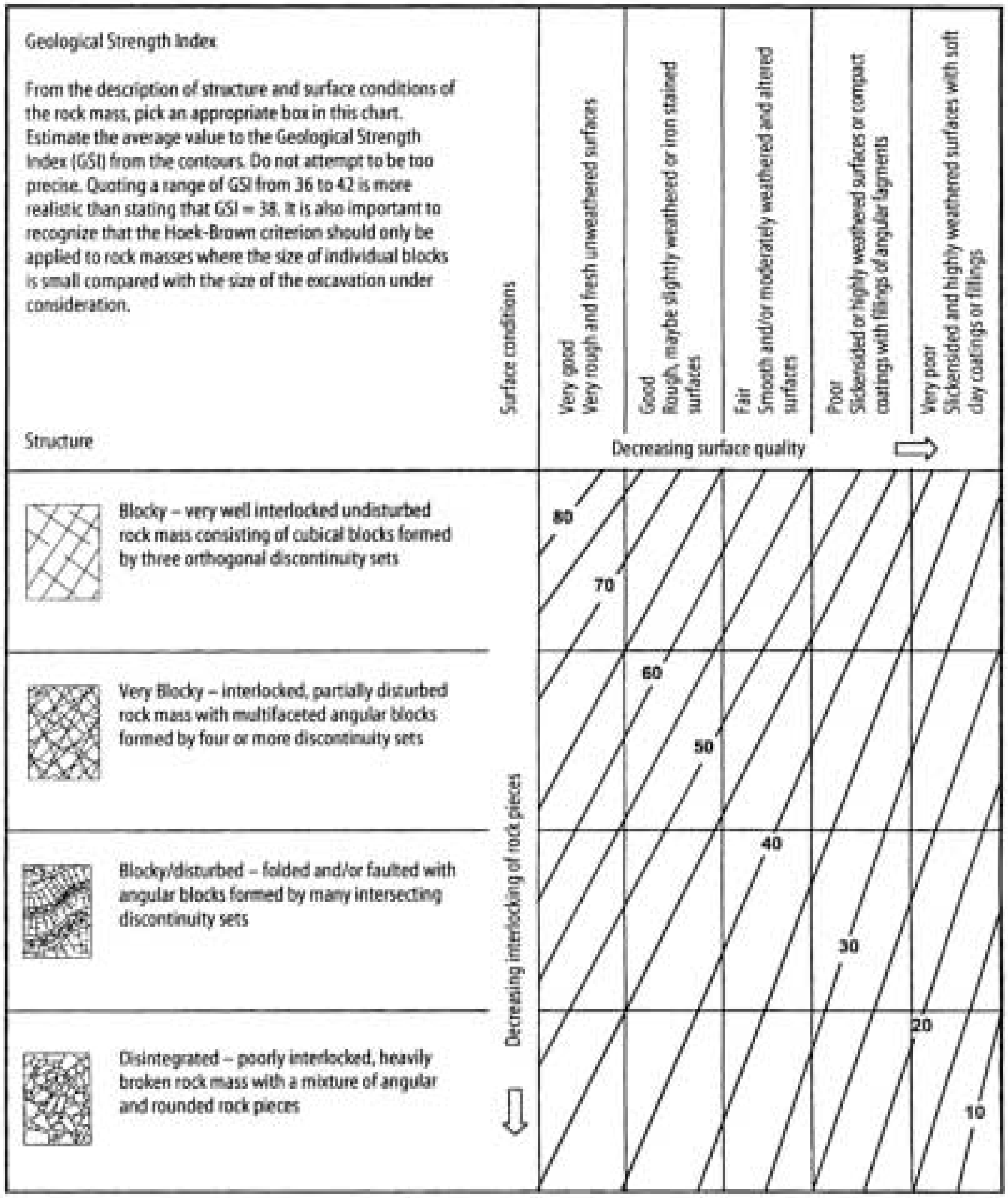


Figure 2-3. *Geological Strength Index (GSI): description of the rock mass quality based upon the interlocking of the rock block and the conditions of the fractures.*

2.3.6 Rock Mass Index (RMi)

The RMi classification system was developed for the need of a strength characterization of the rock mass and for improving the rock mass description /Palmström, 1995, 1996a,b/. The RMi-index uses the following input parameters:

- Uniaxial compressive strength of rock matrix;
- Block volume: the size of the blocks delineated by the joints;
- Joint characteristics as joint alteration, joint roughness and joint length.

The uniaxial compressive strength of the rock mass is expressed by the RMi-value in MPa and is obtained as:

$$RMi = \sigma_{ci} \cdot JP \tag{12}$$

where:

σ_{ci} = the uniaxial compressive strength of intact rock measured on 50 mm samples;
JP= the jointing parameter which is a reduction factor representing the block size and condition of its surfaces as represented by the friction properties. Additionally a scale factor for the size of the joints is also included.

The jointing reduction factor is given by:

$$JP = 0.2 jC^{1/2} Vb^{D} \tag{13}$$

where:

Vb= the block volume in m^3
jC= the joint condition factor expressed as:

$$jC = jL(jR/jA). \tag{14}$$

The exponent D in Eq. (13) is given as a function of jC:

$$D = 0.37 jC^{-0.2} \tag{15}$$

where:

jL = joint length and continuity factor
jR = joint wall roughness
jA = joint wall alteration factor

The ratings jR and jA are almost the same as Jr and Ja defined in the Q-system and are given as tables.

The value of JP varies from 0 for crushed to 1 for intact rock. The JP value can be determined by using several correlations and a special monogram has been developed for the method /Palmström, 1996a,b/.

The following options are given for evaluation of jC:

Block volume (Vb) and jC
Volumetric joint account (Jv) and jC
Average joint spacing and jC
RQD and jC

The volumetric joint account is calculated by the equation:

$$J_v = 35–0.3\ RQD \qquad (16)$$

Block volume, Vb, is one of the most critical parameters and it has a significant impact on the RMi-value. Various methods for determining the Vb value have been recommended.

The block size is mainly defined by small and medium-sized joints in the rock mass. The joint spacing defines the size of the block. Random joints may also have an influence on the size. Significant scale effects are generally involved when the sample size is enlarged. RMi is related to large-scale samples where the scale effect is included in jP values. The joint size factor jL is also a scale-dependent variable.

For a massive rock where the joint parameter jP=1 the scale effect for the uniaxial compressive strength must be accounted for as it is related to a 50 mm sample size. The scale effect of the uniaxial compressive strength can be described by the equation reported by /Palmström, 1996a,b/:

$$\sigma_{cm}=\sigma_{ci}(0.05/Db)^{0.2} \qquad (17)$$

where:

σ_{cm} = uniaxial compressive strength of rock mass
Db= block diameter measured in metre, which may be derived from $Db=Vb^{1/3}$ or in cases with pronounced joint set Db=S= joint spacing of the set. The equation is valid for block sized varying from sample diameters up to some metres. From Table 2-4, it appears that the RMi system might be used to classify extremely weak rock to extremely strong rocks.

Table 2-4. Classification according to RMi.

Terminology		RMi-value [MPa]
RMi	Related to rock mass strength	
Extremely low	Extremely weak	<0.001
Very low	Very weak	0.001–0.01
Low	Weak	0.01–0.1
Moderate	Medium	0.1–1
High	Strong	1–10
Very high	Very strong	10–100
Extremely high	Extremely strong	>100

2.3.7 Ramamurthy's Criterion

/Ramamurthy, 2001/ suggested that the rock mass strength and deformation modulus are related through a joint factor to the strength and Young's Modulus of the intact rock. The definition of the joint factor in Ramamurthy's Criterion is based on laboratory tests on mainly small samples with various adjustments for jointing, joint orientations and loading direction. Actually, this criterion is not classification system in the classical sense that no classes of rock quality are provided. However, its importance lies in the fact that the mechanical properties of the rock mass can be directly be obtained.

The joint factor Jf is obtained from the following equation:

$$Jf = Jn/(n \cdot r) \tag{18}$$

where:

Jn = number of joints per meter in the direction of the loading/major principal stress
n = inclination parameter depending on the orientation of the joint
r = is the roughness or the frictional coefficient on the joint or joint set of greatest potential for sliding.

The Jf-factor combines the joint frequency, inclination of the joints with respect to the loading direction and the shear strength of the joints. The joint with an inclination angle closer to (45°–ϕ/2) to the load direction will be the first one to slide, and ϕ is the friction angle of the joints. This orientation should be considered if several joint sets exist. The r-value could also be obtained from shear tests along the joint and is given by:

$$r = \tau_j/\sigma_{nj} \tag{19}$$

where:

τ_j =the shear strength of the joint;
σ_{nj} = the normal stress on the joint.

2.4 Classification by using geophysical techniques

Geophysical methods can contribute to a continuous overall assessment of the rock conditions at a site. Several methods are available and can be subdivided into surface and subsurface methods. In this Section, some methods correlating rock mass classification with indirect determination of rock mass properties (deformation modulus, Poisson's ratio and fracture frequency) are discussed. In Section 5.9.8, a comparison is given between the results obtained from the characterisation of the rock mass with the values of Q and of the rock mass deformation modulus obtained from the P-wave velocity along vertical seismic cross sections.

It is well documented in the literature that results from dynamic and static testing of the seismic waves on same samples of intact rock often have significant differences /McCann and Entwisle, 1992/. The greatest difference will occur in soft rock while often in dense rock the correlation is better. According to McCann and Entwisle, the two methods are equally valid under different circumstances- depending on if the results apply to near surface or deep excavations. Therefore it might be argued that properties derived from dynamic methods are more pertinent to use for deep excavations.

Seismic and sonic methods have been applied on surface and subsurface measurements of rock mass parameters. So far seismic methods have been correlated with rock mass classification in hard rocks, but seismic data also will contribute to assess other important conditions of the rock mass, such as degree of fracturing, location of weakness zones, etc.

The sonar technique is used in boreholes. The P-wave will be affected by the presence of fractures and fracture zones with a high angle to the measuring direction relative to the borehole. The combined use of the P-wave and the S-wave can be applied to infer the fractured parts of the rock mass with good precision. In addition, if the density of the formation is known or is measured by gamma-gamma log, the elastic parameters can also be evaluated.

2.4.1 Dynamic rock mass parameters

If the rock is considered isotropic, homogeneous and elastic, then the following equations can be used for calculation of the rock properties:

Bulk modulus: $K=\rho_b V^2_s [a^2-4/3]$ (20)

Shear modulus: $G=\rho_b V^2_s$ (21)

Poisson's ratio: $\upsilon = 0.5 [a^2-2]/[a^2-1]$ (22)

Deformation modulus: $E= \rho_b V_s^2 [3a^2-4]/[a^2-1]$ (23)

where:

ρ_b = bulk density
V_p= P-wave velocity
V_s= S-wave velocity
$a=V_p/V_s$

2.4.2 Correlation with fracture frequency

/Sitharam TG, Sridevi J, Shimizu N, 2001. Practical equivalent continuum characterization of jointed rock masses, Int. J. Rock Mech. & Min. Sci, Vol. 38, pp. 437–448. et al, 1979/ gave a theoretical model for calculation of the average jointing frequency given by the equation:

$$N=V_n-V_p/V_n*V_p*ks \quad (24)$$

where:

N= number of joint/m
V_n= average, "natural" P-wave velocity in the rock mass or fracture zone
V_p= P-wave velocity in the actual section to be studied
ks=constant representing the actual in situ conditions

The data on the jointing can be calculated from observations of joint frequency along the seismic profiles, and/or logging data from nearby the boreholes. Data are required from two different locations. The number of joints per meter is best evaluated from calculation of two unknowns V_n and ks from two data sets of measured values of N and the corresponding V-values.

/Palmström, 1995/ has suggested the following relations for an approximate estimate of the joint frequency/number of joints per meter:

$$N=3[V_0/V_p]^{V_0/2} \tag{25}$$

where V_0 is the basic velocity (km/s) of the intact rock under the same condition as in situ i.e. humidity, in situ stress, etc.

2.4.3 Correlation with rock mass rating

A correlation between the seismic velocity Vp and Q-ratings has been proposed by /Barton, 1991/ for rock at shallow depth as:

$$Q = 10^{\frac{V_p-3500}{1000}} \tag{26}$$

For good quality of the rock (Q >4), a better correlation is obtained using the equation /Barton, 1991/:

$$Q = (V_p - 3600)/50 \tag{27}$$

The correlation is mainly based on near surface data. A simple correlation between Q and V_p is presented in Table 2-5 for non-porous rock.

Table 2-5. Approximate correlation between Q-value and V_p-velocity.

V_p (m/s)	1500	2500	3500	4500
Q	0.01	0.1	1	10

For the classification of the rock mass by means of seismic tomography, /Barton, 1995/ proposed a correlation between a new formulation of Q, Q_c, and the seismic velocity V_p, with additional parameters like depth and rock porosity (Figure 2-4), and where the uniaxial compressive strength of the intact rock is directly considered as:

$$Q_c = Q \times \frac{\sigma_c}{100} \tag{28}$$

The uniaxial compressive strength of the intact rock is given in MPa. The chart in Figure 2-4 reflects the influence of the compressive strength and porosity of the intact rock, and the influence of the depth on the seismic velocity. This relation was developed for taking into account the fact that, due to the stress level at depth, the rock mass deformation modulus increases and the porosity decreases depending on the strength of the intact rock.

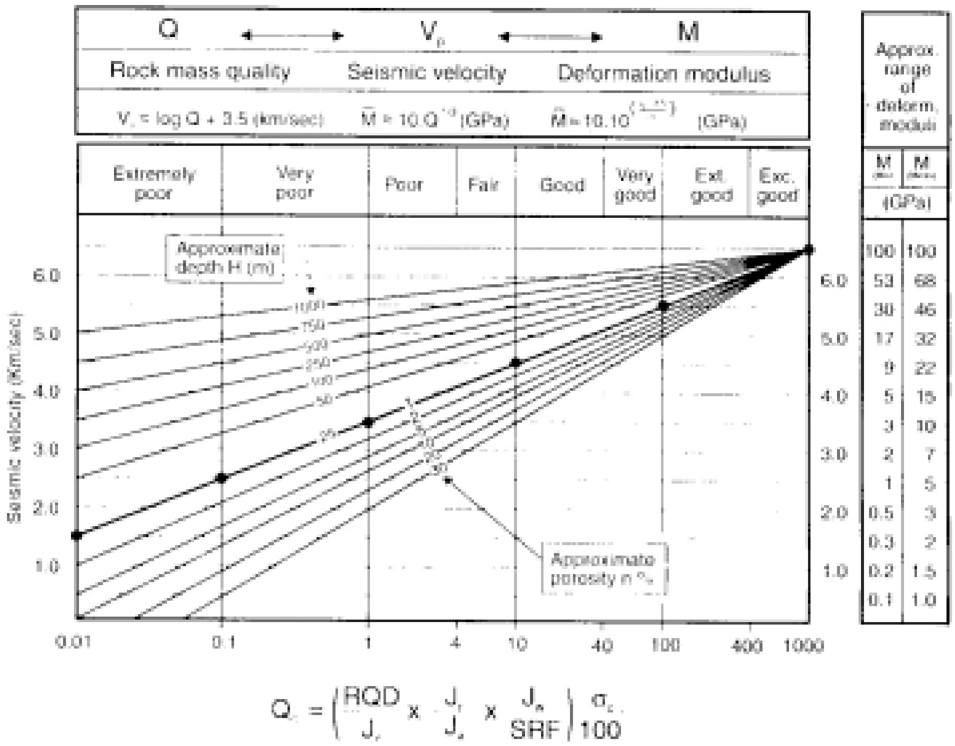


Figure 2-4. *Correlation between the classical and modified Rock Mass Quality Q and Q_c, respectively, and seismic velocity and deformation modulus for design purposes /Barton, 1995/.*

2.4.4 Velocity index

The squared ratio between the compressive seismic wave velocity as measured in the field (V_{pf}) and the sonic velocity measured on an intact rock sample in laboratory (V_{pl}) has been used as an index of rock quality. The ratio is squared for making it equivalent to the ratio between the deformation modulus in situ and the deformation modulus measured in laboratory. /Bieniawski, 1989/ suggested a rock mass quality description based on the velocity ratio according to Table 2-6 (Rock Mass Index).

Table 2-6. Velocity index and Rock Mass Index /Bieniawski, 1989/.

Velocity Index V_{pf}/V_{pl}	Rock Mass Index /Bieniawski, 1989/
<0.2	Very Poor
0.2–0.4	Poor
0.4–0.6	Fair
0.6–0.8	Good
0.8–1.0	Very Good

Vpf= Compressive wave velocity in the field
Vpl= Compressive wave velocity intact rock sample

2.5 Empirical equations for evaluation of the rock mass strength and modulus of deformation

2.5.1 Definitions

Rock mass deformation modulus: The deformation modulus of the rock mass E_m is defined as the ratio of the axial stress change to axial strain change produced by a stress change. The definition of deformation modulus for the intact rock implies no lateral confining pressure. For the rock mass, where there always is some level of confinement, this definition should be modified to take into account the influence of the confining pressure on the deformation modulus. Due to anisotropies, the deformation modulus normally depends on the direction of loading.

Rock mass cohesion: As for intact rock, a rock mass strength criterion can be defined. This is the locus of all points of rock mass failure as a function of the stresses. The rock mass strength criterion is often assumed non-linear. Thus, for a certain stress value or stress interval, the curved strength criterion can be approximated by a line. In particular, if stresses are expressed by the shear and normal stress to a certain plane in the rock mass, the linear approximation can be characterised by two parameters according to the Mohr-Coulomb criterion: cohesion c and friction angle ϕ. The cohesion is thus the intercept of the linear fitting for a normal stress equal to zero. Because these two parameters depend on the stress level at which they are determined, they apply for a defined stress level and stress interval, and often cannot be extrapolated to different stress intervals.

Rock mass friction angle: The friction angle is related to the slope of the linear fitting of the rock mass failure criterion with a line (Mohr-Coulomb criterion). As for the cohesion, the friction angle depends on the stress level and stress interval on which it is calculated.

Uniaxial compressive strength of the rock mass: This definition derives from that of the uniaxial compressive strength of the intact rock, σ_{ci}. For the intact rock, the uniaxial compressive strength shall be calculated by dividing the maximum load carried by the specimen during the test by the original cross-sectional area /Fairhurst and Hudson, 1999/. The specimen is loaded with no lateral confinement. For the rock mass, the uniaxial compressive strength is given for a fictitious specimen when the confining pressure is set to zero. According to Hoek and Brown's definition, the uniaxial compressive strength of the rock mass $\sigma_{cm(H-B)}$ (the strength at zero confining pressure) is:

$$\sigma_{cm(H-B)} = \sqrt{s\sigma_{ci}^2} \qquad (29)$$

where s is a parameter that depends upon the characteristics of the rock mass, and σ_{ci} is the uniaxial compressive strength of the intact rock material making up the sample /Hoek and Brown, 1980/.

2.5.2 Rock Mass Strength

Using GSI and Hoek and Brown Strength Criterion

/Hoek and Brown, 1988, 1997/ proposed the descriptive classification system GSI (Geological Strength Index) for rock masses and some relations between this index and RMR. Through those relations, the parameters defining the rock mass strength envelope can be determined according to Hoek and Brown Strength Criterion.

GSI can directly be estimated when knowing RMR by /Bieniawski, 1989/ where the groundwater rating is set to 15 and the adjustment for orientation to zero. The generalised Hoek and Brown Criterion for jointed rock masses is defined by:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{30}$$

where σ_1 and σ_3 are the major and confinement pressure, respectively, σ_{ci} is the uniaxial compressive strength of the intact rock material, and m_b, s and a are specific parameters characterizing the rock mass. Thus, the parameters that describe the rock mass strength characteristics are:

$$m_b = m_i e^{\frac{GSI-100}{28}} \tag{31}$$

m_i is a dimensionless constant that depends on the intact rock type and can be found in tables in the literature. For rock masses of reasonably good quality (GSI>25), the original Hoek and Brown's Criterion can be applied with $a = 0.5$ and:

$$s = e^{\frac{GSI-100}{9}} \tag{32}$$

For rock masses of very poor quality, the modified Hoek and Brown's Criterion is more suitable with $s = 0$ and:

$$a = 0.65 - \frac{GSI}{200} \tag{33}$$

For determining the equivalent Mohr-Coulomb parameters, a certain stress interval has to be considered (which can be reduced to a single stress level). This is due to the fact that the linear failure criterion has to fit the curved one, whose curvature depends on the confining pressure σ_3. In terms of major and confinement pressures, the Mohr-Coulomb's Criterion can be written as:

$$\sigma_1 = \sigma_{cm(M-C)} + k\sigma_3 \tag{34}$$

From the uniaxial compressive strength of the rock mass $\sigma_{cm(M-C)}$ and the slope of the curve k, obtained from the fitting of the Hoek and Brown's Criterion, the equivalent friction angle and cohesion of the rock mass can be calculated according to:

$$\sin\phi' = \frac{k-1}{k+1} \tag{35}$$

and:

$$c' = \frac{\sigma_{cm}}{2\sqrt{k}} \tag{36}$$

Using Ramamurthy's Rock Strength Criterion

The strength criteria according to /Ramamurthy, 2001/ for a jointed rock mass is given by the equation:

$$(\sigma_1 - \sigma_3)/\sigma_3 = B_j\,(\sigma_{cm} / \sigma_3)^{\alpha j} \quad (37)$$

where:

σ_1 = major principal stress
σ_3 = minor principal stress
σ_{cm} = uniaxial compressive strength of the fractured rock mass
B_j and α_j = strength parameters

The values of B_j and α_j are determined from:

$$\alpha_j/\alpha_i = (\sigma_{cm} / \sigma_{ci})^{0,5} \quad (38)$$

and:

$$B_i / B_j = 0.13\ e^{[2.04(\alpha_j/\alpha_i)]} \quad (39)$$

The values of B_i and α_i are obtained from triaxial tests on intact rock specimens.

Based on the test results in the laboratory a relation was found between the uniaxial compressive strength of jointed samples and the joint factor Jf Eq. (18)). The curve for the mean values of the test data follows the equation

$$\sigma_r = \sigma_{cm}/\sigma_{ci} = \exp\{-0,008*J_f\} \quad (40)$$

where:

σ_r = strength reduction factor

The Ramamurthy's Criterion has been applied for both small and large scale problems /Sitharam et al, 2001/.

Using RMR-system

Among the other mechanical properties that can be estimated using RMR, also (see Table 4.1B in /Bieniawski, 1989/) the cohesion, C, and friction angle, ϕ, of the rock masses can be estimated from Table 2-7. The values are determined mainly on soft rock and the cohesion values given in this table are too low for hard rock.

Table 2-7. Cohesion and friction angles determined using RMR rating /Bieniawski, 1989/.

RMR	100–81	80–61	60–41	40–21	<20
C (KPa)	>400	300–400	200–300	100–200	<100
ϕ (°)	>45	35–45	25–35	15–25	<15

2.5.3 Rock mass deformation Modulus

Using the Q-system

/Barton, 1983; Grimstad and Barton, 1993/ gave some relations for determining the rock mass deformation modulus E_m; The modulus is estimated in the range:

$$E_m = (10 \sim 40) Log_{10} Q \text{ (GPa)} \tag{41}$$

when Q>1, with the mean value calculated as:

$$E_m = 25 Log_{10} Q \text{ (GPa)} \tag{42}$$

From the result of uniaxial jacking tests, for Q ≤ 1 (e.g. fracture zones), the elastic modulus of the fractured rock during unloading cycle, E_e, can be calculated as /Singh, 1997/:

$$E_e = 1.5 Q^{0.6} E_r^{\ 0.14} \text{ (GPa)} \tag{43}$$

where E_r is the elastic modulus of the intact rock (in GPa).

On the basis of the Q-index, the following approximation is proposed for estimating the mean value of the rock mass deformation modulus /Barton, 1995/:

$$E_m \approx 10\, Q^{1/3} \text{ (GPa)} \tag{44}$$

Equations (42) and (44) can be used for fractured hard rocks. For weak rocks, such as in fracture zones, either Eq. (43) or the following expressions for the deformation and shear moduli, under dry or nearly dry conditions and for the depth H, can be used /Singh, 1997/:

$$E_m = H^{0.2} Q^{0.36} \text{ (GPa)} \tag{45}$$

$$G = E_m / 10 \text{ (GPa)} \tag{46}$$

Using the RMR-system

The calculation of the deformation modulus using the RMR rating is given by /Bieniawski, 1978/ as:

$$E_m = 2RMR - 100 \text{ (GPa)} \tag{47}$$

for RMR >50; and /Serafim and Pereira, 1983/:

$$E_m = 10^{\frac{RMR-10}{40}} \text{ (GPa)} \tag{48}$$

An alternative correlation between deformation modulus and RMR ratings were also proposed by /Verman, 1993/:

$$E_m = 0.3 H^{\alpha} 10^{(RMR-20)/38} \text{ (GPa)} \tag{49}$$

where H is the overburden (in meters and ≥ 50m) and α = 0.16 or 3.0 (higher value for poor rocks), when $\sigma_c < 100$ MPa.

The above equations (47)–(49) are used to determine the deformation modulus of the rocks using RMR.

Using Ramamurthy's Criterion

The correlation of the mean deformation modulus of the jointed rock mass with the joint factor Jf Eq. (18)) according to /Ramamurthy, 1995/ follows the equation:

$$E_j = e^{\left(-1.15 \cdot 10^{-2} \cdot Jf\right)} E_i \qquad (50)$$

where:

E_i = tangent modulus at 50% of failure stress of the intact rock for zero confining pressure
E_j = tangent modulus at 50% of failure stress of the rock mass for zero confining pressure

Using the Rock Mass Index RMi

/Palmström, 1995/ also provided a relation between the deformation modulus of the rock mass and RMi, valid if RMi is larger than 0.1. Such relation is:

$$E_m = 5.6\ RMi^{0.375} \qquad (51)$$

3 Stress dependence of the mechanical properties

Most of the mechanical properties of the rock mass are stress, water content and temperature dependent, to certain degrees. The strength of the rock mass is a very well known stress dependent property, thus, it has to be specified for each stress level of concern. Due to the complexity of stress effects on mechanical properties of rock masses, it is difficult to represent them by a single parameter in an empirical classification system. Properly formulated constitutive models are needed with consideration of loading path effects. Therefore, a first estimation of rock mass properties without stress effect was carried out using classification systems (Q and RMR). A model for incorporating the stress effects on the mechanical properties is proposed and discussed in this section.

3.1 Stress dependence of rock mass strength

As defined in Eqs. (30), (34) and (37) the strength of the rock mass depends on the level of confinement stress σ_3 applied. Since the confinement stress in the rock mass often depends on the depth, thus also the strength of the rock mass increases with depth.

3.2 Stress dependence of rock mass deformability

Another parameter that is shown to be stress dependent is the deformation modulus of the rock mass. From several evidences, it appears that the deformation modulus increases by increasing the level of confinement of the rock mass, as it is often observed at depth. Some hard intact rocks do not exhibit any change in stiffness by increasing the level of confinement. This was reported for example by /Stagg and Zenkiewicz, 1975/ for a gabbro and by /Jaeger and Cook, 1976/ for a quartzite (Figures 3-1 and 3-2).

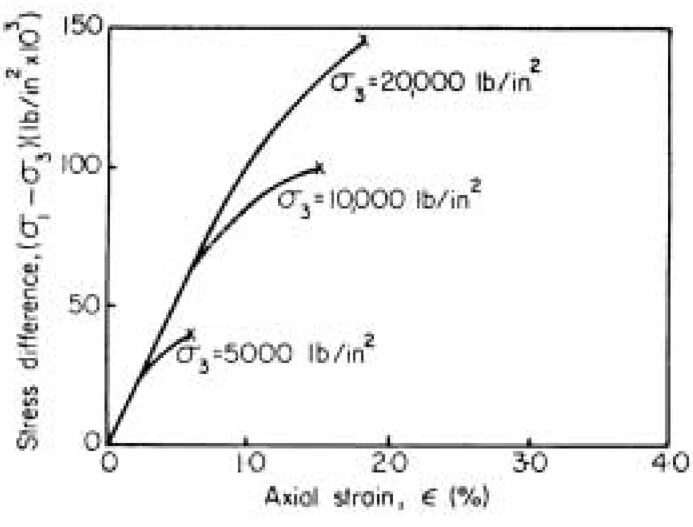


__Figure 3-1.__ Stress-strain curves for a gabbro at various confining pressures /Stagg and Zenkiewicz, 1975/.

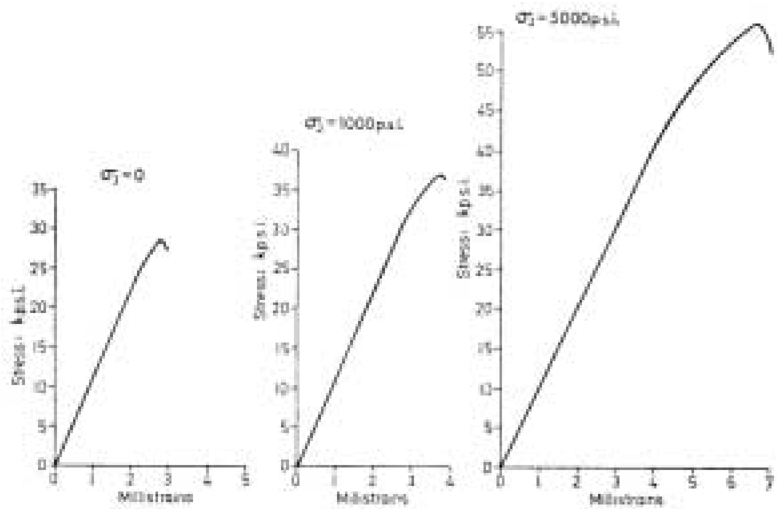
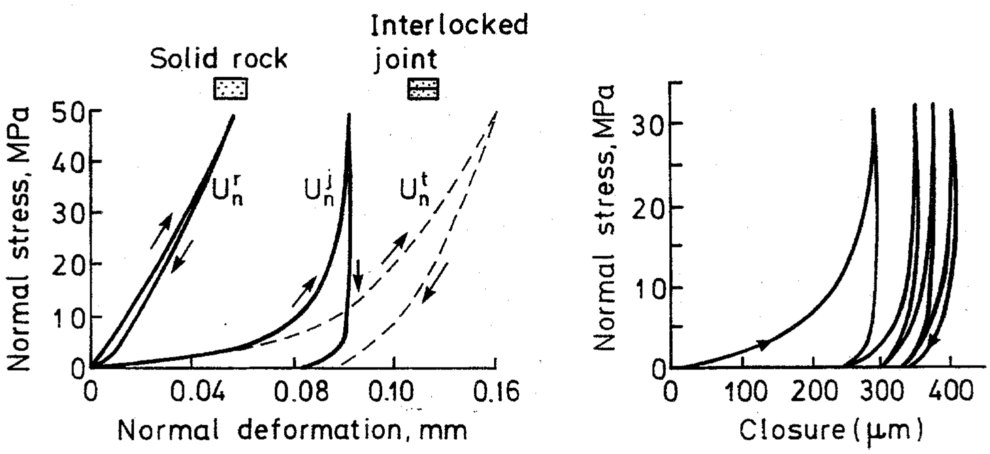


Figure 3-2. Stress-strain curves for Rand quartzite at various confining pressures /Jaeger and Cook, 1976/.

On the other hand, the fractures are responsible for most of the rock mass deformability and become stiffer with increasing confinement. The deformation of the fractures is not linear with stress increase, as it is observed from the deformation curve of natural rock fractures under normal loading (Figure 3-3).



(a) (b)

Figure 3-3. *(a) Normal stress versus fracture closure for solid rock, U_n^r, a sample with a fracture, U_n^t, and the difference between the fracture sample and intact rock deformation, U_n^j. (b) Cyclic loading of a fracture sample /Bandis et al, 1983; Barton, 1986/.*

According to some experimental results on samples from the Prototype Repository /Lanaro and Stephansson, 2001/, in a first approximation, the stiffness of the fractures can be assumed to increase almost linearly with normal stress, as it is shown in Figure 3-4.

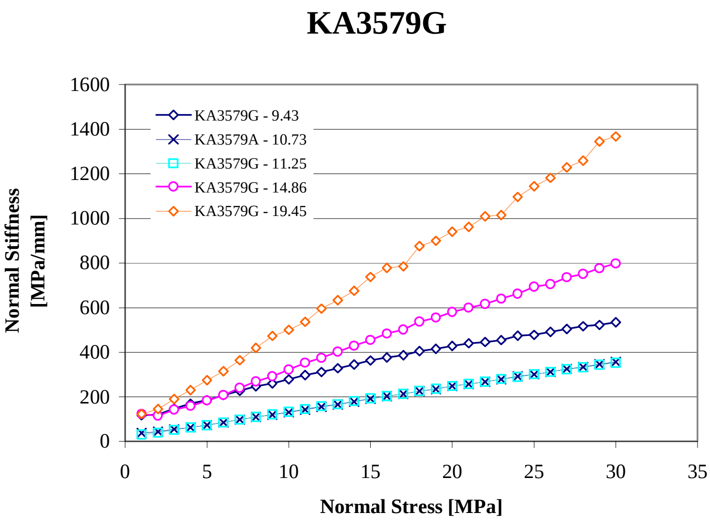


Figure 3-4. *Samples from sub-horizontal fractures (borehole KA3579G, Fracture Set 2): Secant normal stiffness versus normal stress starting from an initial stress of 0.5 MPa /Lanaro and Stephansson, 2001/.*

Because the stiffness of the rock mass cannot exceed that of the intact rock, in the engineering practice is sometimes assumed a non-linear relation between the deformation modulus and the depth z as (e.g. 3DEC User Manual):

$$E_m = E_{m0} + c\sqrt{z} \tag{52}$$

where E_{m0} is the deformation modulus for very low confinement pressure and c is a proportionality constant. If the confinement pressure were assumed to increase linearly with depth, thus the deformation modulus would not be linearly related to the confinement pressure.

The deformation modulus of the rock mass can also easily be calculated according to /Li, 2000/. He assumes a parallel system where the stiffness is given by the contribution of the intact rock and that of the fracture sets, as:

$$\frac{1}{E_m} = \frac{1}{E_i} + \sum_{j=1}^{N} \frac{\cos^2 \vartheta_j}{S_j} \left(\frac{\cos^2 \vartheta_j}{k_{nj}} + \frac{\sin^2 \vartheta_j}{k_{sj}} \right) \tag{53}$$

where E_i is the Young’s modulus of the intact rock, S_j, k_{nj}, k_{sj} and θ_j are the spacing, normal and shear stiffness, and the angle between the loading direction and the line orthogonal to the fracture planes for the j-th fracture set, respectively. This model takes into account the anisotropy of the rock mass due to the presence of N different fracture sets. Moreover, assuming that the normal and shear stiffness of the fractures is a function of the normal stress acting on the fractures (here also called confinement pressure σ_3), the variation of the rock mass deformation modulus with confinement pressure can be assessed. For example, for Block H, the parameters in Table 3-1 can be input (according to /Norlund et al, 1999; Lanaro, 2002a,b/.

Table 3-1. Input data for the determination of the anisotropy and stress dependence of the rock mass deformation modulus /Norlund et al, 1999; Lanaro, 2002a,b/.

Young’s modulus of the intact rock		73 MPa
SET2: orientation (strike/dip)		22/8
Mean spacing		0.66 m
Normal load	kn [MPa/mm]	ks [MPa/mm]
0.5 MPa	87	4.9
5 MPa	157	14.9
10 MPa	273	30.9
15 MPa	388	*(48)*
SET3*: orientation (strike/dip)		139/90
Mean spacing		1.51 m
Normal load	kn [MPa/mm]	ks [MPa/mm]
0.5 MPa	57	3.8
5 MPa	81	12.6
10 MPa	127	30.4
15 MPa	147	*(37)*

*) All vertical joints are assumed to have the same properties independently on the fracture set they belong to.

The deformation modulus in Eq. (53) can be studied along three orthogonal directions, one vertical and two horizontal directions (parallel and perpendicular to the major principal stress σ_1 at the Äspö area, which strikes 150°) so that the diagram in Figure 3-5 can be obtained. It can be observed how the deformation modulus increases by increasing the confinement pressure acting on the fractures. Furthermore, the orientation of the fractures with respect to the direction of loading seems to play an important role on the deformation modulus of the rock mass.

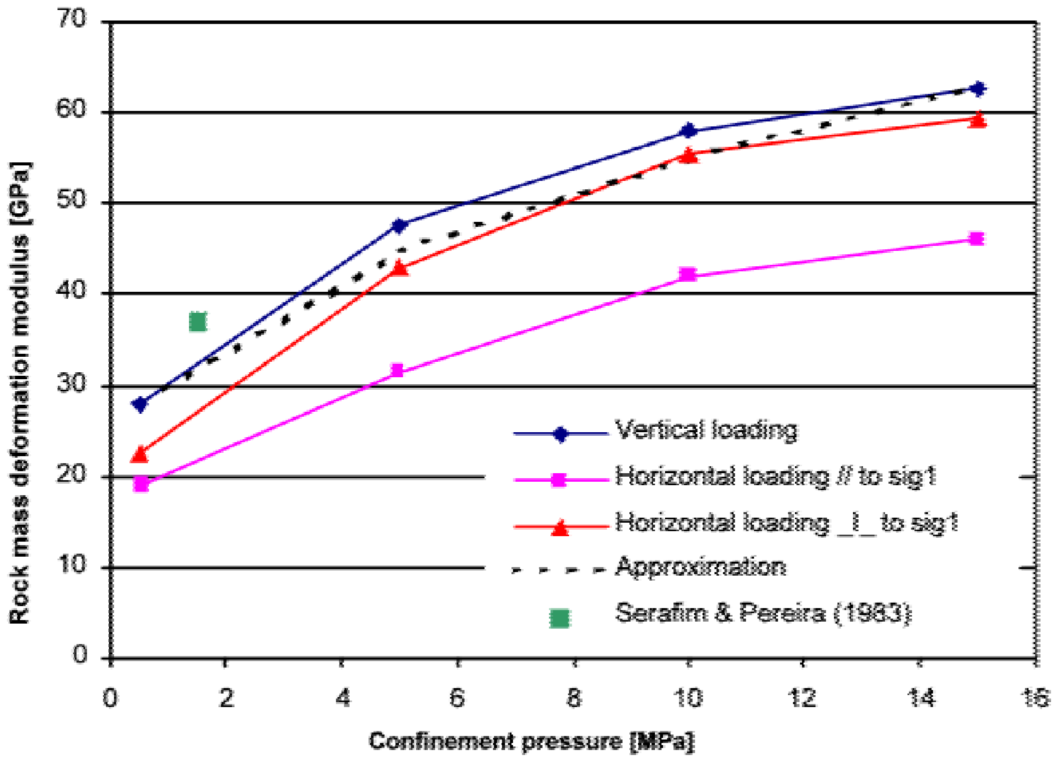


Figure 3-5. *Variation of the rock mass deformation modulus with the direction of loading and the level of confinement pressure (Eq. (53)) and comparison with the deformation modulus obtained by using the relation by /Serafim and Pereira, 1983/ (Äspö test case, Block H).*

We propose that the curves in Figure 3-5 can be approximated by a function of the confinement pressure very similar to Eq. (52):

$$E_m = E_{m0} + c'\sqrt{\sigma_3} \qquad (54)$$

where σ_3 is the confinement pressure. For a rock unit of the Äspö test case (see Chapter 5), the coefficients E_{m0} and c' were roughly evaluated in 20 GPa and 350 $GPa^{1/2}$, respectively, while the deformation modulus of the rock mass obtained by RMR-characterisation and the relation by /Serafim and Pereira, 1983/ was also determined for Block H was 37 GPa (see Table 5-13). As the empirical models are mainly based on case histories of superficial tunnels (about 50 m depth and 1.5 MPa confinement pressure), the mean deformation modulus for Block H can also be plotted in Figure 3-5. Even considering that the chosen level of confinement pressure of 1 MPa for plotting the empirical result is quite arbitrary, the agreement between the analytical solution and the empirical method is very satisfying.

The SRF factor in the Q-system is designed for making the rock mass quality to increase with increasing depth and, in turn, with stress. For this purpose, the values in Sec. 2.3.2 were suggested by Barton for the Äspö Test Case. In consequence of this rating choice, the parameters derived from Q would also present a stepwise variation with depth.

4 Statistical treatment of data and uncertainties

4.1 Statistical treatment of data

All parameter values, ratings and properties should be treated with statistical tolls for presentation of the characterisation results. Statistical tools and descriptors enable a string of numerical values for a given property to be summarised in a compact format that can readily be understood. In this study, unimodal statistics are mainly considered, which means that the data are usually interpreted as realisations of one single population. Appropriate statistical techniques might identify secondary population distributions when the overall distribution is bi- or multimodal.

For a given set of measured values, the experimental frequency distribution is often a powerful tool for recognizing the kind of theoretical statistical distribution that better approximates the real one. The experimental distribution can also be characterised by its principal statistical parameters. If the data population consists of N values x_1, x_2, …x_N, then the principal statistics are:

Mean: $\bar{x} = \frac{\sum_{i=1}^{N} x_i}{N}$ (55)

Standard deviation: $\sigma = \sqrt{\frac{\sum_{i=1}^{N} (x_i - \bar{x})^2}{N-1}}$ (56)

Two parameters often used in descriptive statistics are the mode and median; the mode is the value that occurs with the greatest frequency, while the median is the value for which there is an equal number of values greater and less in the data string x_1, x_2, …x_N. The median is more robust than the mean for non-Normal populations and the difference between the two provides a simple indicator of the skewness (or asymmetry) of the distribution.

Another useful statistical parameter for describing a population is the range R that is defined by:

Range: $R = x_{\max} - x_{\min}$ (57)

where xmax and xmin are the largest and smallest values in the population sample.

The spatial variability of the data can be significant. For data sets whose mean value and standard deviation do not vary significantly across the zone of interest, and for a considerably large zone, it is possible to incorporate the physical location, as well as the values observed, in a statistical analysis. This can help in making a reasonable estimation of a parameter's value assuming to know the value for geometrically close sample points. The spatial correlation between the values of the parameter can be quantified by using the semi-variogram. Let x(si) and x(si+h) be two values measured at a distance h. For all the n pairs of values taken at that distance, the semi-variogram γ(h) can be defined as:

Semi-variogram: $\gamma(h) = \frac{1}{2n}\sum_{i=1}^{n}\left[x(s_i) - x(s_i + h)\right]^2$ (58)

The semi-variogram usually increases for increasing spacing distance h until it comes to a sill that is equal to the sample variance (=standard deviation square). This property implies that there is some spatial correlation between the values when h is relatively small. The distance at which the spatial correlation is lost is called range of influence of the variogram. This range provides an indication of the distance below which it is possible to estimate the values of the parameter from the values at the locations that have been sampled.

4.2 Quantification of the parameter uncertainty

A technique was created to quantify the confidence in parameter values, which is directly related to quantification of uncertainty/variability. The principle of the technique is a ranking of the confidence in the rating parameter values according to the following influence factors:

- types and quality of the information (published SKB reports, data files, on site observations, personal communication, engineering judgement and reasoning);
- operational biases for a particular parameter (measurement techniques, personal perspectives, different time of measurement);
- size of sampling and data population;
- different evaluation techniques for related parameter (e.g. σ_{ci}, JCS, JRC, J_r, Q from seismic velocity, visual inspection, estimations during logging, availability of comments concerning logged parameters);
- confidence in the estimation of the difficult parameters (e.g. aperture, fracture surface roughness, coating, weathering, and trace length/persistence);
- ambiguity in the descriptions in certain classification codes;
- engineering/expert judgment for difficult situations, such as lack of data;
- mismatching between the geological and engineering definitions;

The ranking for confidence in a rating parameter can be different depending on the available data source quality and quantities for a particular rock unit. The following cases of data source availability are considered:

1. surface mapping + borehole logging + shaft + experimental result data;
2. surface mapping + borehole logging + experimental result data;
3. surface mapping + borehole logging;
4. surface mapping only;
5. no data available.

The confidence in a rating parameter changes from high for case 1) to low for case 5).

For the Q classification system, the ratings are ranked according to the following tables for surface and borehole data as examples:

Table 4-1. Surface mapping (for shallow rock units).

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain			X	X	X	X
Probable	X	X				
Guesswork						

Table 4-2. Borehole logging (for deeply buried rock units).

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain	X	X*				
Probable			X	X		
Guesswork					X	X

* When having several non-parallel boreholes in the same rock unit

For the RMR classification system, the ratings are ranked according to the following table for surface and borehole data as examples:

Table 4-3. Surface mapping (for shallow rock units).

RMR	$R_{strength}$	R_{RQD}	Rs_{pacing}	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X			X		X	X	X	X	
Probable		X	X							
Guesswork					X					X

Table 4-4. Borehole logging (for deeply buried rock units).

RMR	$R_{strength}$	R_{RQD}	Rs_{pacing}	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X	X	X							
Probable					X	X	X	X		
Guesswork				X					X	X

For rock units with information from different sources, the borehole data was ranked as of the highest level of confidence, followed by the surface and shaft/tunnel data, depending on the data availability and location of the rock unit. Considerations were also given to the positions of the boreholes with respect to each other and the size of the rock unit.

Using above confidence levels, three classes are defined for values of each *rating parameter* as:

- CERTAIN: the classification is done by using exactly the value of the parameter/rating supported by all or most of reliable sources with data availability cases 1–2: all variations in the data are accounted for spatial variation and sampling bias;

- PROBABLE: The classification is based on engineering judgement and reasoning to a certain extent, with very limited support from reliable data sources (cases 3–4). There is a possibility that the chosen classification class may have certain variation margins, but not more than one rank higher or lower than the estimated values;

- GUESSWORK: the classification is based basically on engineering judgement/reasoning without support of reliable data sources (cases 4–5), and variation of the rating/parameter could be large. A margin of two ranks higher and lower than the estimated values, within a reasonable limit, is given to quantify this class of uncertainty.

Assuming that the *rock units* can be treated as homogeneous entities, the uncertainty analysis is performed on the values of the rating parameters. The following strategy of quantification of the above uncertainty levels is adopted:

- CERTAIN: the mean value of the rating/parameter in different rating system is directly used without uncertainty margins;

- PROBABLE: the next upper and lower class of rating in different rating system is chosen as the uncertainty margin for the parameter, if they are within a reasonable limit permitted by the rating system.

- GUESSWORK: the next two upper and lower classes of the rating in different rating system is used as the uncertainty margin for the parameter, if they are within reasonable limits permitted by the rating system.

5 Methodology applied to the Äspö Test Case

The Äspö Test Case is a part of the project that practically tests the Empirical, Theoretical and Stress models. Each working teams reviewed similar limited data for the test. The test approach is described in this Chapter. The main goal was to determine the rock mass properties for strength and deformability for the "550 m" and "4–500 m" models. The two classification systems, RMR and Q were applied for characterisation and classification of the rock mass.

5.1 Site geology and model geometry

5.1.1 Regional structural geology at Äspö area

The geological model at the area of interest for the project contains five main deformation zones (EW1a and b, NE 1, NE 2 and EW3). The rocks are different types (granodiorite, fine grained granite/aplite, greenstone and mylonite). This volume of the model can be grossly divided into two domains: one north of and one south of deformation zones EW-1a and b. These two domains differ for the number of fracture sets and their orientation, and for some of the fracture properties. Very scarce information is available about the minor deformation zones EW 3 and NE 1.
Some seismic profiles were produced across the volume of interest.

5.1.2 The 550 m Model

Figure 5-1 illustrates the size and geometry of the large 550 m model, with the shaft and ramp systems. It contains the size and locations of the 4–500 m Target Area model between the levels of –380 m and –500 m. Figure 5-2 shows the 3D geometry and relative positions of the unit/block system, with blocks labelled from A to N defined by the fracture zones.

The 550 m model is only a structural model without explicit presentation of the lithology (rock types). The rocks within each unit/block are therefore mixtures of possible rock types without clearly defined boundaries. This lack of rock type representation, especially the geometry of main rock formations, such as granite, diorite, etc, appears to be a significant limitation for a geological model and may affect the rating implementation works to an unknown extent.

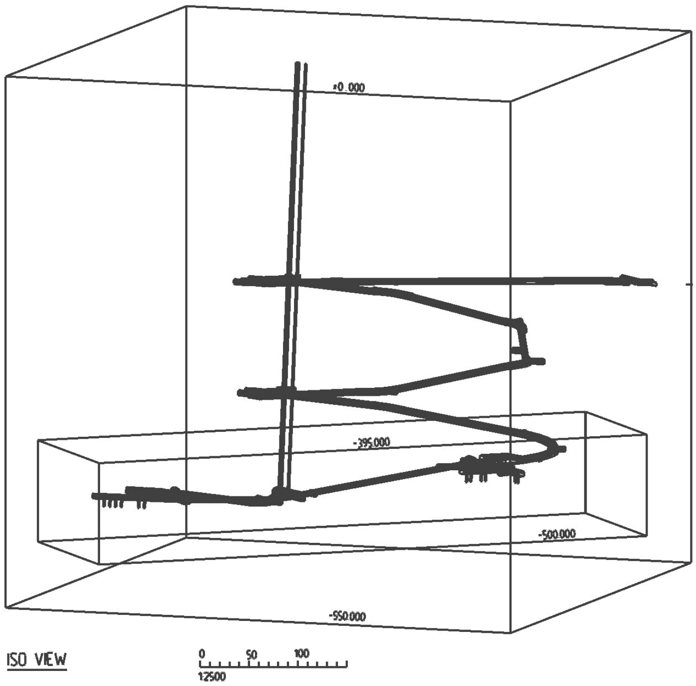


__Figure 5-1.__ The size and location of the 550 m model for the Test Case.

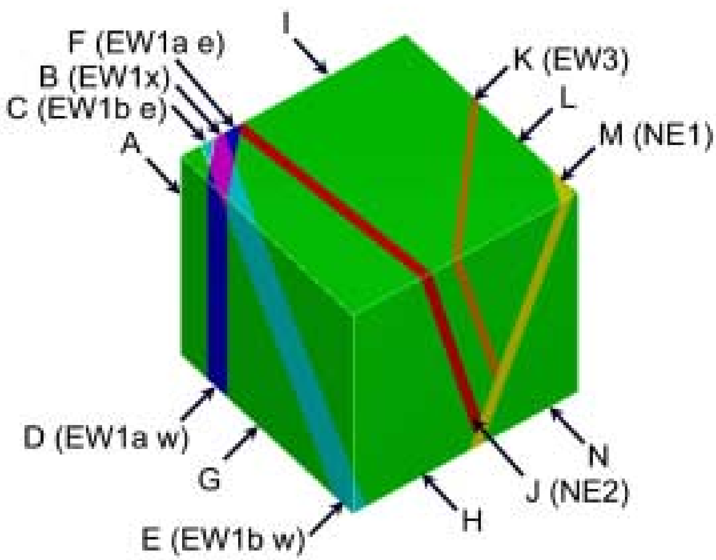


__Figure 5-2.__ Definition of the rock units for the 550 m model.

5.1.3 Target Area – The 4–500 m Model

Figure 5-3 illustrates the size of the Target Area (the 4–500 m model) and the relative locations of the shaft, ramps and Prototype Repository Area. The volume of the model was divided into four layers of equal thickness of 30 metres in the vertical direction. Each horizontal layer is then, in turn, divided into a grid of 30 × 30 × 30 m cubes (cells), with a sequential labelling from 1–120 for Layer 1 (–380 m – –410 m), 121–240 for Layer 2 (–410m – –440m), 241–360 for Layer 3 (–440m – –470 m) and 361–480 for Layer 4 (–470m – –500m), respectively. Figure 5-4 illustrates the location of boreholes together with the fracture zones.

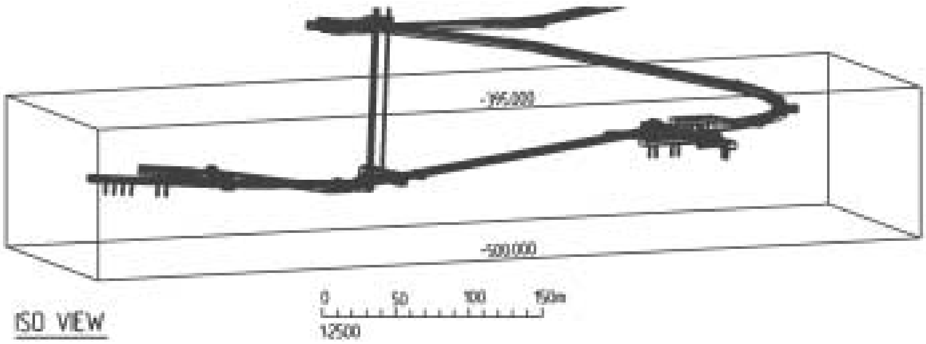


Figure 5-3. *The size and location of the 4–500 m model for the Test Case.*

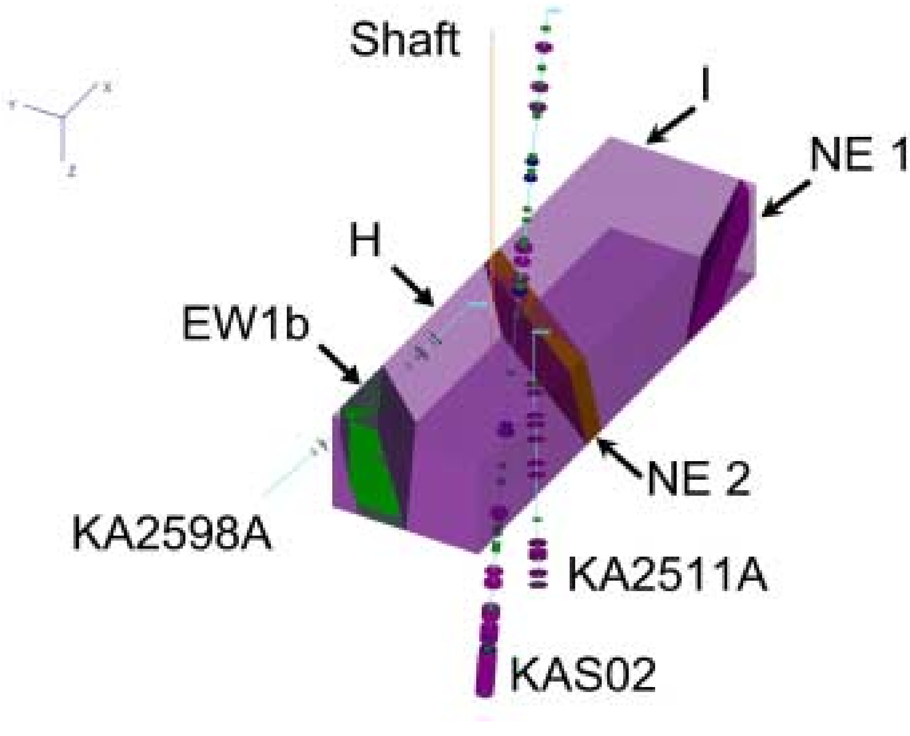


Figure 5-4. *The size and location of the 4–500 m model for the Test Case.*

5.2 Data from core logging, surface and shaft mapping

There are seven types of data sources available for the Q/RMR-rating systems:

1. The RVS (Rock Visualisation System) geology/geometry models for the block system definition, supplied by SKB for the project.

2. Borehole logging records for three boreholes: KAS02 which passes through Block H, I and J; KA2598A which passes through Block G, H and E, and KA2511A which passes through Block H, supplied by SKB for the project.

3. Surface mapping data about fractures orientations and trace lengths, which covers all blocks except for Block K and M, reported in /Ericsson, 1988/;

4. Mechanical shear test data with fracture samples taken from the Prototype Repository area, with samples taken from the cells 263, 264, 281–283, and 302–303, at Layer 2, and testing for uniaxial compressive strength of intact rocks with samples taken from cell 283, reported in /Lanaro and Stephansson, 2001; Lanaro, 2002a,b/.

5. Shaft mapping data containing orientation and traces of fractures of trace lengths larger than 1.0 m, located within block/unit H, supplied by SKB for the project.

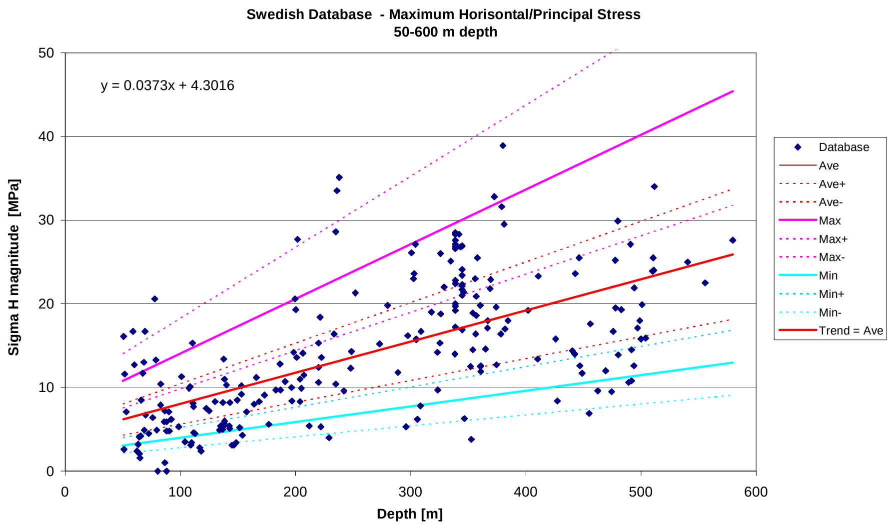
6. Mechanical testing of samples of intact rocks and fractures, reported in /Stille and Olsson, 1990/. The properties produced include uniaxial compressive strength, Young's modulus, Poisson's ratio of greenstone, aplite, diorite and granite, and the friction angles of steep and gently dipping fractures, with rock types not reported. The locations of the intact rock samples were also not reported.

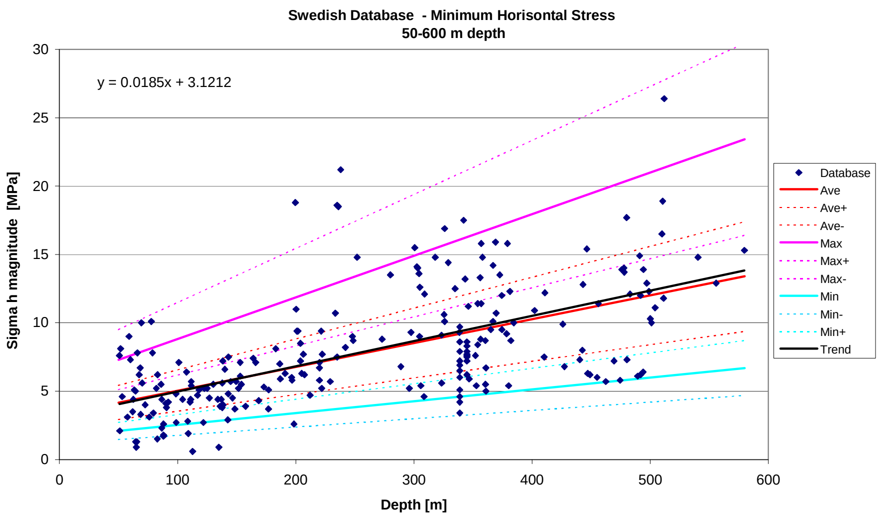
7. Mechanical testing of rock fractures and intact rock samples reported in /Nordlund et al, 1999/, with samples taken near the Prototype Repository area at Äspö. The rock type is diorite and the mechanical properties produced include the uniaxual compressive strength, tensile strength, Young's modulus, Poisson's ratio, cohesion, internal friction angle, strength parameter m in the Hoek-Brown's Strength Criterion, and other fracture related properties.

8. Measured in-situ stress results at Äspö area reported by /Hakami et al, 2002/.

5.3 Initial stress field and groundwater issues

Initial stress field and groundwater flow behaviour affects the Q and RMR ratings to a very significant extent, as represented by the ratio J_w/SRF in Q and RMR_{water} in RMR rating systems. The measured in situ stresses are used to determine the major and minor principal stresses according to the largest depth of the location under consideration, for example, at the bottom end of a core section that is used to determine Q and RMR ratings along a borehole (Figure 5-5). The stress field is therefore basically uniform without considering possible changes due to changes in rock types, and fracture zones.





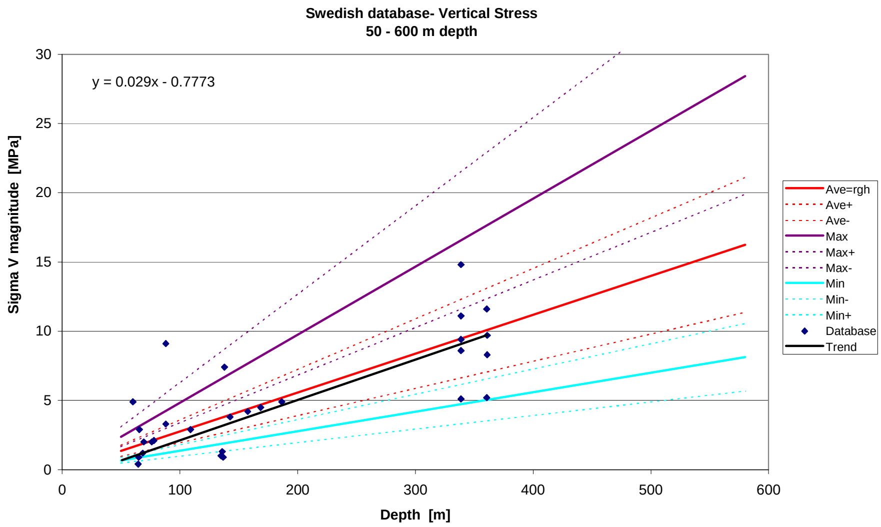


Figure 5-5. *Measured in situ stress used for the ratings (Swedish Stress Database, by /Hakami et al, 2002/.*

For the groundwater issue, it is assumed that a hydrostatic pressure field should be used to determine the water pressure at different depths, with zero pressure at the ground surface. In reality, water flow is controlled by connected fracture networks and is not uniform, as can be seen from the Äspö HRL tunnels. However, this effect cannot be incorporated properly with the rating systems at this stage of the project and the fracture system effect on water pressure has to be ignored. This may cause overestimated water pressure to some (unknown) locations, whose effect on the overall ratings cannot be properly evaluated.

Some inflow rate data is available for boreholes KA2598A and KA2511A. However, it is not so straightforward to transform these data into inflow rate per 10 m of a tunnel and therefore only the pressure is used for RMR ratings, and engineering judgement for the descriptive flow condition (dry, dripping, large flow, etc) for the Q ratings.

In this project, for characterization of the rock mass, we considered only mechanical properties under dry conditions as a start. This was also required in order to be compatible with the theoretical approach where coupling with water was not considered. In addition, hydrogeological effects were not defined as a part of this project at this stage. Therefore, the value of J_w for Q was set to 1.0, and RMR_{water} was set to 15, for characterisation.

5.4 Division of the core sections

Each borehole in each block unit was divided into a number of core sections along its length of homogeneous RQD and fracture frequency values. This is usually done in practice during classification of the rock mass by RMR and Q, of which RQD is an input parameter. This partitioning corresponds to a preliminary classification of the rock mass according to /Deere, 1968/. The difference in rock types is not considered in the current RVS geological unit system model. The difference in fracture set numbers was treated as variations within the block, without splitting blocks into sub-units. Figure 5-6 illustrates the technique of division of borehole sections and the data files created for each core sections.

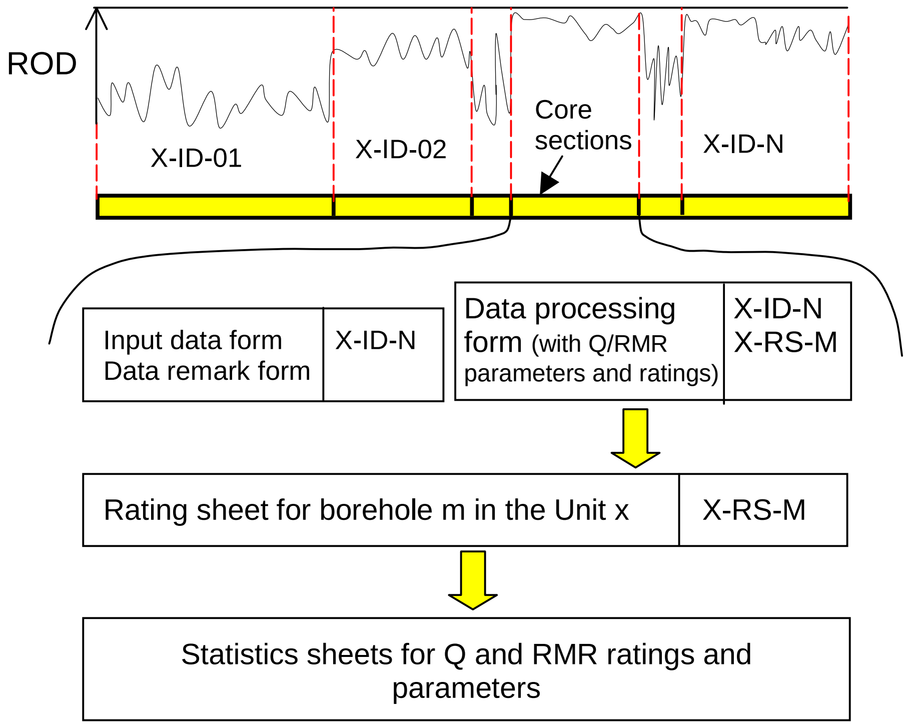


Figure 5-6. *Example of division of borehole sections for Q/RMR parameterisation.*

5.5 Data Processing Format

Some forms were designed for accommodating all information needed for the rock mass characterisation by RMR- and Q-systems and its results.

1. Input Data Form:

   This sheet contains all basic information for parameterisation according to Q and RMR systems, with a data file label "X-ID-N", where X is the block label (from A-N) and N is the sequential number of the section (1, 2 …10, 11, 12…). The σ_{ci}, Point Loads Strength, RQD, fracture spacing, water condition and all the fracture condition parameters are divided into several categories according to the definitions in Q and RMR.

   For the in situ stresses, a triple value set is created for both major and minor principal stresses in the form of x/+y/–z, where x is the mean expected value, y is the difference between the maximum and mean expected values and z is the difference between the mean and minimum expected values, respectively. These values can be used for estimating the SRF factors in Q.

   The total spacing of the fractures is calculated from the total frequency of all sets of fractures over the length of the domain.

2. Data remark form:

   This sheet contains the information and file sources and comments on the fracture conditions.

3. Data processing form:

   This sheet is the parameterisation form for both Q and RMR with all necessary parameters, category keyword and parameter values.

4. Rating sheet:

   The Q and RMR parameters and ratings from the data processing sheet for all core sections are collected into this form. From this form, statistical analyses of Q, Q' and RMR for characterisation and design and all associated summarised as:

   - Arithmetic and weighted (against core length) mean values of ratings;
   - Standard deviation of ratings;
   - Maximum and minimum ratings;
   - Mean, maximum and minimum values of parameters (RQD, …);
   - Histograms of all ratings and parameters.

5. Output parameter sheet:

   The rock mechanics parameters obtained from the ratings are here summarised by their statistics and histograms. Here, only one set of parameters is given according to the empirical relations that were judged most representative for each particular parameter.

The processing sheets described above form a nested data-file system for storage and cross-reference of all the input data, parameterisation, ratings, comments, sources and results for each rock unit, as shown in Figure 5-7.

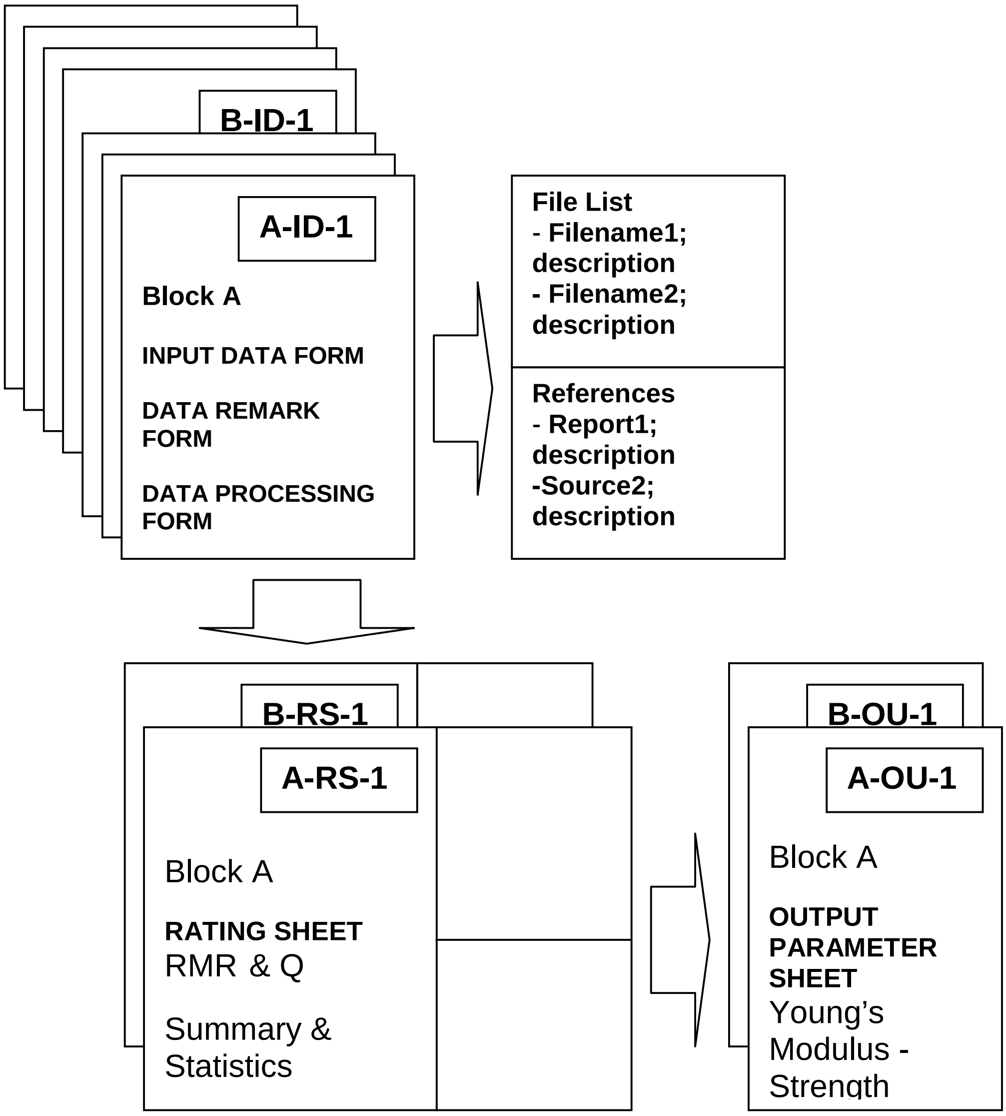


Figure 5-7. *The nested data processing forms for rock mass characterisation.*

5.6 Parameterisation for the Q-system

Techniques applied for determining the values of parameters and Q-ratings vary from parameter to parameter, depending basically on the quantity and quality of the source data. The parameterisation considers mainly borehole data, but tunnel/shaft mapping data should also be properly used if available.

5.6.1 RQD

The RQD values are mostly calculated directly for each core section from borehole logging records and are therefore most reliable for blocks with boreholes. For blocks where only surface mapping data are available, the RQD values were estimated using the fracture density data over the whole model site, which can also be regarded as reliable, but with much larger variability margins.

5.6.2 Jn

The Jn concerns with influence of the number of fracture sets, and was calculated for each borehole, using both borehole orientations and the fracture mapping results on the shaft walls, which is located near the centre of the model site. Because set delineation needs larger number of data, the Jn values were calculated for each borehole and were assumed to be valid for the whole unit/block.

The most frequent number of fracture set is 2 and 3, plus random fractures. As pointed out before, the stability conditions of underground works changes significantly from two sets to three sets of fractures. Therefore for units/blocks with different set numbers for different boreholes, such as the Block H, splitting of the block with different Jn values may be a more proper rating technique.

5.6.3 Jr

The Jr values, concerning the effect of fracture roughness, come mainly from three sources: JRC values determined by laboratory shear tests /Lanaro, 2002b/, borehole logging information and direct site observations by the team at Äspö site. All these sources point to the fact that fractures at the Äspö site are basically planar to undulating in a large scale and slightly rough at smaller scale, i.g. JRC <8 (see Figure 5-8). This observation was assumed to be generally valid for units/blocks where no borehole is available.

Figure 5-8. *Photographs showing the typical roughness and flow of fractures.*

5.6.4 Ja

The Ja parameter concerns the conditions of the fracture surfaces, mainly coating, infilling, shear history and the residual friction angle. With the shear history largely unknown, the parameter value was determined using mainly the residual frictional angle determined from laboratory tests of fractures and coating/infilling conditions from borehole logging records. Since coatings are the most frequent fracture feature (clay was rarely encountered), and $\phi_r = 25° \sim 30°$, the most frequent condition is Ja = 2 (category C) for the Q-rating system.

5.6.5 Jw

The parameter Jw concerns the effect of water, and should be determined according to inflow and water pressure data. Since the information for water inflow and pressure along most of the boreholes is largely not available, the Jw parameter values are estimated based on an assumed hydrostatic water pressure, varying linearly with depth and in fully saturated condition. This may cause significant errors in the Jw parameter values for fractured hard rocks since the flow and pressure conditions are often compartmentalized, determined by local fracture geometry. On the other hand, details for local flow and pressure cannot be obtained and the hydrostatic condition may still be a useful compromise. Jw = 1 can be assumed for near surface small blocks, such as A, B, C, D and F without borehole data.

For characterization of the rock mass Jw=1 and for the calculation of Q for design the value will vary depending on the hydraulic pressure and the estimated fracture geometry and condition according to the Jw rating table of the Q-system.

5.6.6 SRF

The SRF concerns the effect of stress and is perhaps the most difficult parameter to estimate, due to subjective descriptive classification and large stepwise jumps of SRF values. For characterization of the rock mass SRF=1.

5.7 Parameterisation for the RMR-system

The parameterisation for the RMR-system is similar, in technique, to that for the Q-system.

5.7.1 RMR for rock strength

The $RMR_{\text{strength of intact rock}}$ is determined for each core section using the uniaxial compressive strength (σ_{ci}) data from different sources: compression test results from the Prototype test area and Schmidt hammer tests on intact rock. The results reported by /Stille and Olsson, 1990/ are used for the rock units where there are not borehole and test data available.

5.7.2 RMR for RQD

The same RQD values used in Q-ratings are used to determine RMR_{RQD} rating, using Chart A in /Bieniawski, 1989/.

5.7.3 RMR for fracture spacing

The fracture spacing is not considered in Q-system, but is an important parameter in RMR. The total spacing of the fractures is calculated with the total frequency of the fractures for each core section if borehole logging data is available, and the surface mapping data /Ericsson, 1988/ which is assumed to be valid over the whole site area. $RMR_{fracture\ spacing}$ rating is determined using Chart B in /Bieniawski, 1989/.

5.7.4 RMR for fracture length

The fracture length is not used in Q, but is required in RMR. There are mainly two sources for length data: the surface mapping data /Ericsson, 1988/ and shaft mapping data. The former give a most reliable and direct estimate of the trace lengths of fractures mapped at surface exposures, roughly about 1 m. The latter provides the number of fractures with trace lengths larger than 4.5 m. Using these two data sources, the trace length of the fractures was grouped into three classes according to the ratio of the number of fractures of trace length larger than 4.5 m (found at shaft walls) over the total number of fractures found in the core sections, see Table 5-1. The trace length values from the table below were used to produce the $RMR_{fracture\ length}$ according to Chart E by /Bieniawski, 1989/.

Table 5-1. Persistence/trace length classification.

Persistence	Trace length	Ratio of large/all fractures
Very low	<1 m	<5%
Low	1–3 m	5–15%
Medium	3–10 m	>15%

5.7.5 RMR for fracture aperture

Fracture aperture was not measured in the reported surface mapping by /Ericsson, 1988/, but was recorded in borehole logging records and shear test by /Lanaro, 2002b/. The aperture value from these data sources can be divided into three classes: very tight fractures of aperture 0–0.1 mm, tight fractures with aperture = 0.1–0.5 mm and moderately open fractures with aperture 0.5–1 mm, respectively. These values are used to determine the $RMR_{fracture\ aperture}$ using Chart E in /Bieniawski, 1989/.

For units/blocks without borehole, such as blocks A, B, C, D and F, anmean value of aperture 0.1–0.5 mm was assumed since it is the most representative aperture value from the borehole logging records.

5.7.6 RMR for fracture roughness

The same fracture roughness estimation as that used for Q-ratings is used for the RMR-system, with the most representative category of “slightly rough” surface. This was used to estimate the $RMR_{fracture\ roughness}$ rating according to Chart E in /Bieniawski, 1989/.

5.7.7 RMR for fracture infilling

The RMR rating for fracture infilling has five classes (no infilling, hard infilling of <5 mm thickness, hard infilling of >5mm thickness, soft infilling of <5 mm thickness and soft infilling of >5mm thickness, respectively). From the surface mapping results and borehole logging records, the majority of the fractures have no infilling but only coating of clay-free minerals. Considering also that the apertures of the fractures are generally small, the class of "no infilling" was chosen to estimate the $RMR_{fracture\ infilling}$ rating (=6), for all units/blocks.

5.7.8 RMR for fracture weathering

As in the Q-ratings, the "slightly weathered" condition applies throughout the whole model site, and the value for $RMR_{fracture\ weathering}$ is thus determined as 5, according to Chart E in /Bieniawski, 1989/.

5.7.9 RMR for groundwater

For characterisation, the RMR_{water} parameter for groundwater is taken equal to 15. The RMR_{water} rating for design is determined using either inflow data or ratio of fracture water pressure over the major principal stress. Since inflow data is not available, the water pressure according to the assumption of a hydrostatic condition was used to calculate the pressure/stress ratio at each core section, using Table 4.1 in /Bieniawski, 1989/.

5.7.10 RMR for fracture orientation

The RMR rating for considering relative orientation of fractures with respect to tunnel orientations cannot be properly estimated without tunnels. For characterisation purposes, considering various possible tunnel orientations of a repository, a rough estimation of a "good" fracture orientation was assumed for the whole model and all units. This produces a constant value of $RMR_{fracture\ orientation} = 0$ for all units/blocks.

In Äspö, there are not very persistent fracture sets with parallel fractures, thus, in a first approximation, no particular unfavourable directions can be recognised for the excavation of the tunnels. In addition, in design of tunnels at depth, the directions of principal in situ stresses are also as important as the fracture orientation. Therefore a favourable tunnel orientation relative to fracture set and stress orientation might or might not be ensured in both design and characterisation.

5.8 Evaluation of the uncertainties

5.8.1 Main uncertainty issues and their treatment

The most important uncertainties are those related to the limitations of the empirical rating system themselves. The main limitations of the empirical approach are:

- The empirical approach of the rating systems makes it impossible to check whether they will obey basic laws of physics, such as conservation laws. The deformation modulus derived does not come from properly defined constitutive models, but an empirical estimate with possible assumption of equivalent elastic gross mass behaviour.

- Strength parameters in the RMR system based on no specific failure criterion, but possibly a Mohr-Coulomb shear failure criterion, which may or may not meet site-specific rock conditions.

- The estimated rock mass deformation modulus and strength parameters based on the empirical approaches cannot be explicitly made stress-dependent and fracture-system –geometry-dependent, such as orientation of fracture sets. The SRF factor in Q system provides a means to modify Q-values considering different stress effects, but cannot make the Q values as direct functions of stresses. The properties produced by the RMR approach are stress-independent.

These limitations must be considered when comparing results from different approaches.

The other main uncertainties related to the site conditions considered at present in this project are:

1. *Uncertain fracture length distribution*

   This uncertainty comes from the fact that most of the fracture information comes from three boreholes, KAS02, KA2511 and KA2598A, and the shaft near the centre of the 500 Model (see Fig. 5.1) and it is thus mainly fracture orientation information. The main source for fracture length/size, which is needed for RMR rating, comes from surface mapping results for regional structural geology purposes rather than fracture system characterization, with no information regarding unit/block structures. The treatment of this uncertainty is to assume that the mean value (about 1.0m) of the fracture length from the surface mapping is valid for the whole model area. This basic value is then modified locally at different borehole depths when calculating local $RMR_{\text{fracture length}}$ values along borehole sections. The modification is done with engineering judgement based on the numbers of larger fractures appearing on the shaft walls (whose size is certainly larger than 4.5m, the diameter of the shaft), where the shaft is in the concerned unit and core sections. This treatment puts the mean fracture length in the range of 1–3 m for most cases, and 3–10 m range in a few cases, according to the RMR rating system.

2. *Uncertain spatial distribution of mechanical properties of intact rocks and fractures*

   The required mechanical properties of intact rocks and fractures for using Q and RMR ratings are basically the Uniaxial Compressive Strength of the intact rock (σ_{ci}), and for the properties of the rock fractures: Joint Wall Compressive Strength (JCS), residual friction angle ϕ_r and Joint Roughness Coefficient (JRC). These properties were tested with samples from a number of rock units of the Äspö Test Case (see Table 5-2). Except for the shear tests reported in /Lanaro and Stephansson, 2001; Norlund et al, 1999/, sampling locations were not reported in other early technical reports, such as the rock mechanics testing and evaluation by /Stille and Olsson, 1990/. This lack of proper documentation makes the unit rating and estimation of spatial variation of the properties difficult. The treatment of this uncertainty is to assume that the mean values of these properties are valid for the whole unit where samplings and tests were reported.

Table 5-2. Available data of mechanical properties from boreholes or tests.

Block	Intact rock		Fractures			
	Rock type	σ_{ci} (mean/sdv/ No. Samples)	**Set** No.	**JCS** (mean/sdv/ No. Samples)	ϕ_r (mean/sdv/ No. Samples)	**JRC** (mean/sdv/ No. Samples)
A	Diorite	165/50/–	2	80	NA	NA
	Data estimated from surface mapping results		3	80	NA	NA
			4	80	NA	NA
B	Mylonite	137	2	NA	NA	NA
	From surface mapping		3	NA	NA	NA
C	NA	NA		NA	NA	NA
D	NA	NA		NA	NA	NA
E	Granite	137.04/–/1	2	108.6/33.41/4	31.69/2.23/4	8.28/4.91/17
	Diorite	138.98/33.41/6	3	NA	NA	3.75/0.87/3
F	NA	NA	NA	NA	NA	NA
G	Granite	130.98/–/1	1	83.44/31.72/3	28.75/1.93/3	5.16/1.93/12
	Diorite	141.44/4.38/2	3	112.8/23.8/3	31.02/0.86/3	6.54/2.4/9
H (from 3 boreholes)	Granite	124.01/–/1 128.64/22.04/2	1	83.96/34.3/24	31.11/3.29/4	6.49/1.44/16
	Diorite	91.57/31.2/5 138.04/22.04/12 147.9/52.6/5 218.75/17.5/4**	2	172.19/–/1 91.55/9.9/2 80–82/–/11*	28.26/–/1 34.23/1.8/2 27/–/11*	6.22/1.68/6 8.23/6.24/4 6.09/3.39/11*
	NA	NA	3	157.4/32.1/14 116.2/32.4/10 56.4/–/1 82–85/–/5*	28.03/3.38/14 25.79/–/1 32/–/5*	6.68/2.47/42 7.48/5.48/3 8.2/2.28/5*
I	Granite	164.83/50.3/5	1	NA	NA	5.05/0.54/3
	Amphibolite	114.87/–/1	3	80.54/23.57/5	25.84/3.19/5	8.58/3.28/16
J	Granite	140.64/1.09/2	1	121.3/33.45/3	32.67/1.16/3	6.35/2.64/10
K	Diorite	92.29/–/1	NA	NA	NA	NA
L	NA	NA	NA	NA	NA	NA
M	NA	NA	NA	NA	NA	NA
N	NA	NA	NA	NA	NA	NA

* data from shear tests of fractures /Lanaro and Stephansson, 2001/
** data from tests by /Nordlund et al, 1999/
NA = Not Available

3. *Uncertain block information*

This uncertainty comes from the fact that a large number of blocks, A, B, C, D, F, K, L, M and N are almost “blank” blocks where no data from borehole, shaft, or mechanical testing are available. The only data source available is the surface mapping data about fracture sets and trace lengths, and indications from seismic velocity records over the whole Äspö site, which is divided into two areas, the North Domain area (covering Blocks A, B, C, D, E and F) and the South Domain Area (covering Blocks H, I, L and N). The blocks K and M are totally “blank” blocks where no information is available at all (see Table 5-3).

Table 5-3. Information sources for the blocks/rock units.

Unit (Block)	Types of information
A	Surface mapping*
B	Surface mapping*
C	Surface mapping*
D	Surface mapping*
E	Surface mapping* KA2598A
F	Surface mapping*
G	KA2598A
H	Surface mapping** KAS02 KA2598A KA2511A
I	Surface mapping ** KAS02
J	KAS02
K	NA
L	Surface mapping **
M	NA
N	Surface mapping**

* Surface mapping data from the North Domain
** Surface mapping data from the South Domain
NA = Not Available

Block E, H, I, J and G have borehole data support. The rest have not and they can be divided into three categories: small surface blocks (Blocks A, B, C, F) formed by fracture zones EW1a and EW1b; large but thin blocks formed by fracture zones (Blocks D, K and M) and massive blocks (Blocks L and N). Normal rating systems cannot be applied to these blocks because of the lack of data. They are treated differently according to their size and locations.

For units/blocks A and B, the σ_{ci} from /Stille and Olsson, 1990/ and the surface fracture mapping results from /Ericsson, 1988/ were used because no other information is available.

Block C is a small block near ground surface without borehole and mechanical test, and is located close to Blocks A and B, see the RVS model. Therefore the rating and properties of Block C are assumed the same as those of Block B.

Block D is a large thin block with large depth and is formed by a fracture zone EW1b, similar to Block E that was formed by fracture zone EW1a. It is assumed that the ratings and properties of the Block D are the same as those of Block E, whose rating is supported by borehole data. Block F is a small surface block near Block B, whose rating and properties are assumed to be the same as those of Block B, based on the same reasoning as mentioned above.

Blocks K and M are formed by fracture zones EW3 and NE1, respectively. No information is available for these two blocks, not even the specific rock types. These two blocks are left as blank blocks without rating at present stage.

Blocks L and N are normal large blocks where no mechanical testing and borehole data are available. However, surface-mapping data exists. It is assume that these two blocks have the same rock types, ratings and mechanical properties as those of Block I which is the nearest block supported by borehole and mechanical test data.

4. *Uncertain effect of rock types on block unit division*

   It was decided that the blocks A – N formed by the fracture zones as represented in the 550 m model should be taken as the unit system, and no further division of blocks according to different rock types, mechanical properties and fracture density should be considered. These blocks have therefore mixed major rock types. During processing of the borehole data, it was found out that distinct large zones of different rock types, sometimes over 100 m in one direction, exist with different mechanical and fracture characteristics, in the same blocks (see, e.g. Blocks H and J). The block system model defined for this project does not, therefore, represent properly the site geology, at least in view of rock lithology.

   It was also found out that in the same blocks with multiple boreholes, different fracture densities exist from different borehole logging data. Typically the number of fracture sets is 2 or 3, plus some random fractures. These two numbers will not only have large effects on Q ratings, but are also important indicators of rock mass stability conditions for any underground constructions. Therefore, a proper site investigation project should delineate the areas where the fracture set number is 0–2 and equal or larger than 3. The Q-system provides a very convenient tool for this purpose.

   These differences in rock types and fracture densities should be properly represented, at least in large scales qualitatively, in any geological model and numerical models, and should be considered in rock classification work, especially in view of the general aim of establishing rating methods and procedures. The effect of this ignorance on the final rating results is unknown and needs to be investigated.

5. *Uncertain greenstone formations*

There are an unknown number of lenses of greenstone in the model area, but their exact locations and extensions are largely unknown. A general estimation is that these greenstone lenses may occupy about 5% of the total volume of the model area. It is assumed that effect of the greenstone on the ratings could be ignored because of their small volume.

6. *Uncertain fracture surface conditions*

The surface condition of the fractures, such as roughness, weathering, in-filling, coating, flow, wall strength and aperture, are important indices for both Q and RMR rating systems. Among these parameters, the wall strength, roughness and coating are relatively more properly characterized by using Schmitt hammer tests and borehole observations, and the weathering, in-filling and aperture are largely unknown.

The fractures are treated as weathered or slightly weathered when their wall strength (JCS) is smaller than the strength of the intact rock σ_{ci}, whether the fractures are shallow or deep in rocks. This assumption is based on the water flow and washing out of gouge materials from natural fractures at different depth of the Äspö tunnels during the site visit, and the fact that weathering may be caused more by oxidized water at depth (cf. Figure 5-8).

Fracture coating minerals were observed in the core logging but in-filling thickness is not included. Infillings were not observed for the samples tested for shear by /Lanaro, 2002b/. Thus, it is assumed at this stage that no infilling exists in general for this project.

The fracture aperture is not properly measured in the borehole data, with only qualitative descriptions such as “zero” or “very small”. It is assumed that all fractures have very small apertures, typically 0.1–1 mm, or zero, for RMR ratings.

7. *Uncertain fracture orientation relative to tunnel orientation*

Since no tunnel is concerned, it is assumed that a general “fair” index for the fracture orientation should be adopted for all RMR ratings as a general condition. This will cause a reduction of total RMR rating by 5, and may be a relatively conservative estimation on average.

8. *Uncertain validity of the empirical coefficients used for determining mechanical properties using Q and RMR ratings*

The empirical coefficients, which are used for determining mechanical properties using Q and RMR values (cf. Eqs. (42), (43), (47) and (48)), are established over long periods of practices. However, they may still not be suitable for the site-specific conditions of the model area in this project. It is also impossible to investigate this issue further at this stage of the project.

The variability of the ratings, therefore also the derived properties, is mainly represented by the histograms of the individual parameters, rating values and the derived properties. This issue is described more in detail in the following sections about statistical treatment of variability.

5.8.2 Uncertainty quantification tables of rating parameters

Applying the procedure for quantification of the uncertainties of the rating parameters for RMR and Q systems in Sec. 4.2, the following tables were created for surface, shaft mapping and borehole data.

Table 5-4. Surface mapping.

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain			X	X	X	X
Probable	X	X				
Guesswork						

Table 5-5. Borehole KAS02.

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain	X	X*				
Probable			X	X		
Guesswork					X	X

* For the rock units with three non-parallel boreholes

Table 5-6. Borehole KA2511A.

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain	X	X*				
Probable			X	X		
Guesswork					X	X

* For the rock units with three non-parallel boreholes

Table 5-7. Borehole KA2598A.

Q	RQD	Jn	Jr	Ja	Jw	SRF
Certain	X	X*				
Probable			X	X		
Guesswork					X	X

* For the rock units with three non-parallel boreholes

Table 5-8. Surface mapping.

RMR	$R_{strength}$	R_{RQD}	Rs_{pacing}	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X			X		X	X	X	X	
Probable		X	X							
Guesswork					X					Constant

Table 5-9. Borehole KAS02.

RMR	$R_{strength}$	R_{RQD}	Rs_{pacing}	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X	X	X							
Probable				X*		X	X	X		
Guesswork					X*				X	Constant

* data from shaft mapping were available for these two columns.

Table 5-10. Borehole KA2598A.

RMR	$R_{strength}$	R_{RQD}	$R_{spacing}$	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X	X	X							
Probable					X	X	X	X		
Guesswork				X					X	Constant

Table 5-11. Borehole KA2511A.

RMR	$R_{strength}$	R_{RQD}	$R_{spacing}$	R_{length}	$R_{aperture}$	$R_{roughness}$	$R_{infilling}$	$R_{weathering}$	R_{water}	$R_{orientation}$
Certain	X	X	X							
Probable					X	X	X	X		
Guesswork				X					X	Constant

5.8.3 Treatment of stress and water effects

/Olsson et al, 1992/ recognised the need of discerning between mechanical properties of the rock mass and the effect of the loading conditions, especially when numerical tools are to be applied for predicting the behaviour of the excavations or structures by e.g. Finite Element Analysis or Discrete Element Method.

The stress and water are loading mechanisms that affect the rock mass. There are international opinions that these two factors should not be considered for rock characterization, but must be considered for design /see Palström et al, 2001/. It should also be noted that it is difficult for the empirical rating systems to consider effects of stress and water on mechanical properties of fractured rocks since constitutive laws are not part of the empirical rating systems. On the other hand, to ensure being reasonably conservative for underground construction design, water and stress loading effects must be considered in the rating systems.

Based on the above reasoning, two sets of rating calculations were performed: i) without considering the stress and water conditions by setting SFR = 1.0 and Jw = 1.0 in Q-system, and RMR_{water} = 15 in the RMR system, for rock characterization and ii) with values of SRF, Jw and RMR_{water} parameters determined by initial stress data and a hydrostatic water pressure assumption at corresponding depths, for design. The mechanical properties determined by i) should represent the rock mass properties under stress-free and dry conditions, and the mechanical properties derived by ii) can only be used for design of tunnel dimensioning and supporting, not for characterization of rock properties.

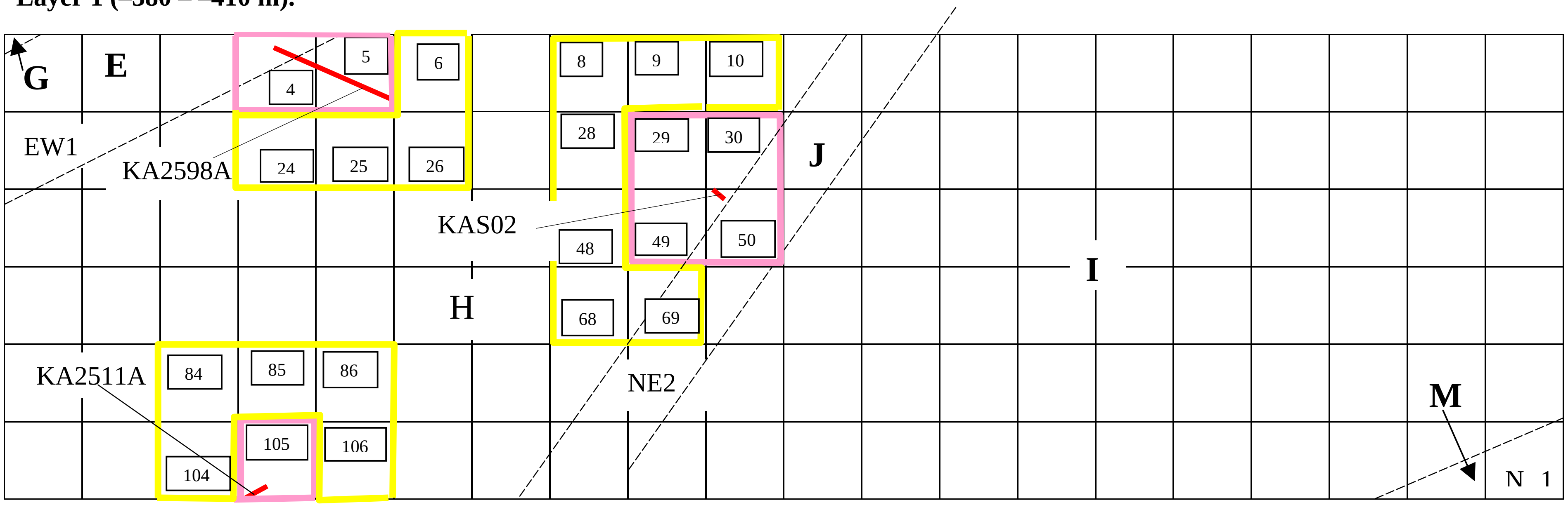
5.9 Summary of the results

According to the definition of the Test Case model, the deformation modulus and its standard deviation should be estimated for each cell, together with the Q and RMR ratings and a confidence level:

1. Ratings and properties are obtained by local data support;
2. Ratings and properties are obtained by interpolation/reasoning;
3. Ratings and properties are obtained through pure guesswork.

To define this confidence level, a rule of thumb is defined as below: i) The cells with one or more boreholes passing through will have confidence 1; ii) The cells will have confidence level 1 if they have nearly equal distances to a vertical borehole as a nearby cell that contains the vertical borehole. This applies to the borehole KAS02; iii) The immediate surrounding cells around the cells of confidence level 1, within the same unit/block, will have confidence level 2; iv) The rest of the cells have confidence 3. This rule basically interpolates the confidence level according to the distance of cells from boreholes. The cell structure of the layers and their confidence levels are shown in Figure 5-9 a to d.

Layer 1 (–380 – –410 m).

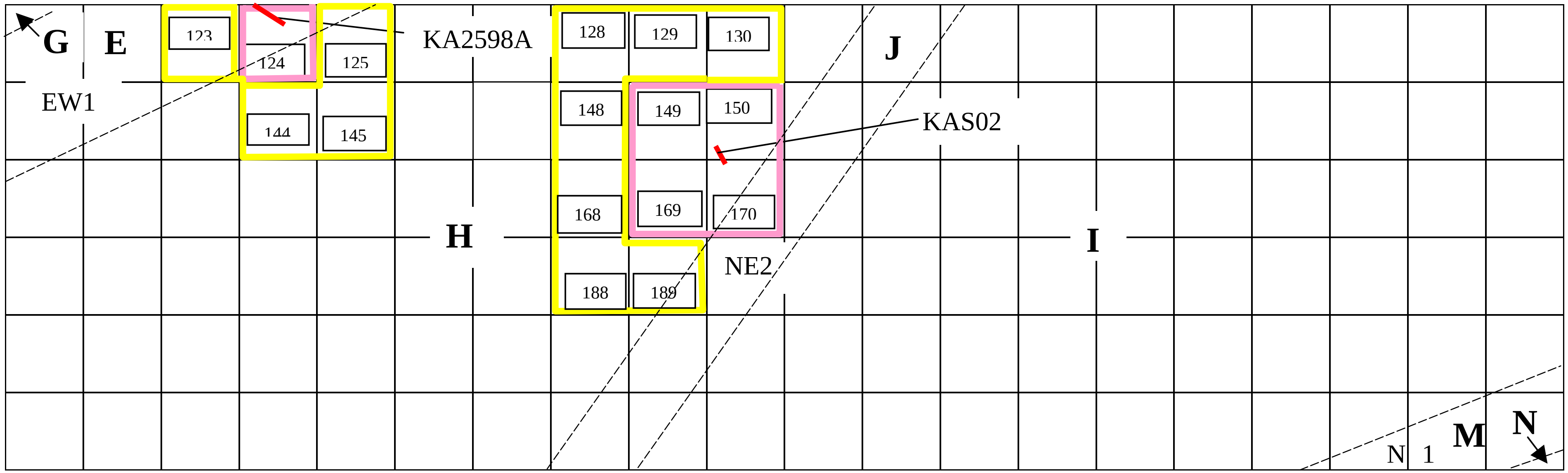


K – block (unit) number; - - - - - - - - - boundaries of blocks (fracture zones); 50 - cell number; Borehole sections in cells.

- cells with confidence level 1; - cells with confidence level 2; The rest of the cells have confidence level 3.

Figure 5-9a. *Confidence levels in cells – Layer 1.*

Layer 2 (–410 – –440 m).

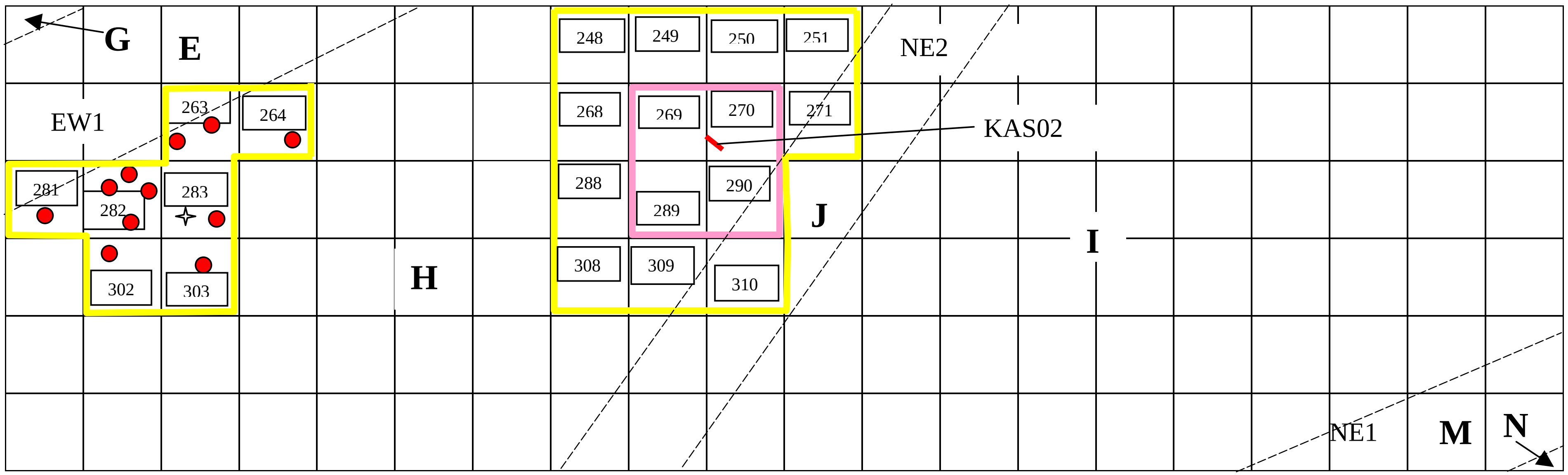


K – block(unit) number; - - - - - - - - - boundaries of blocks (fracture zones); 150 - cell number; Borehole sections in cells.

- cells with confidence level 1; - cells with confidence level 2; The rest of the cells have confidence level 3.

Figure 5-9b. *Confidence levels in cells – Layer 2.*

Layer 3 (–440 – –470 m).



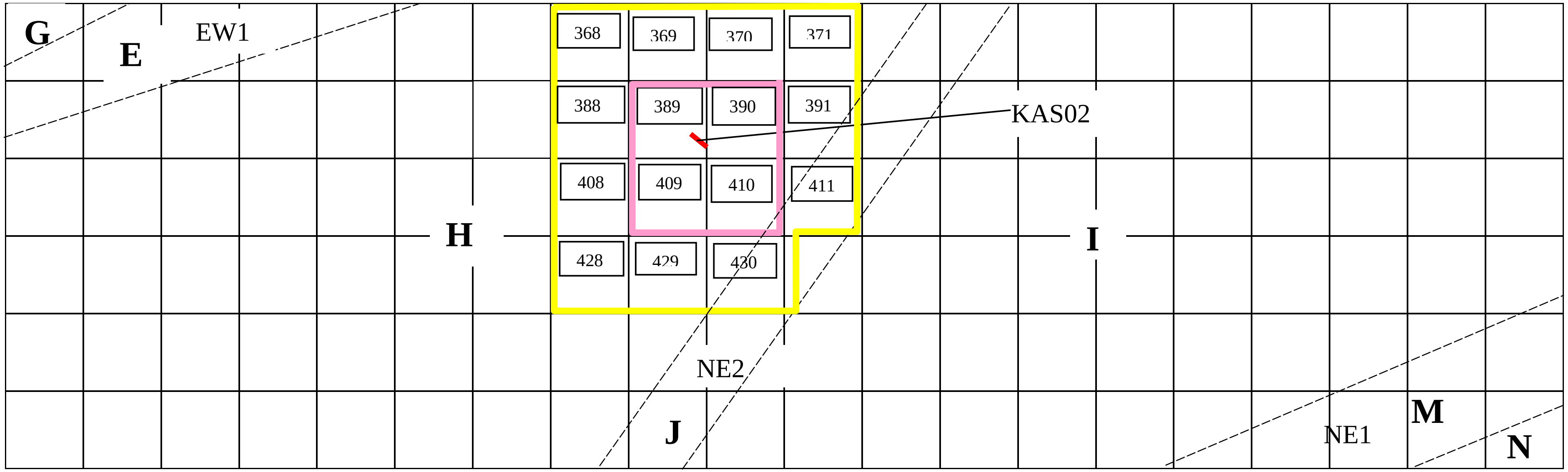
K – block (unit) number; --------- boundaries of blocks (fracture zones); 270 - cell number; Borehole sections in cells.

- cells with confidence level 1; - cells with confidence level 2; The rest of the cells have confidence level 3.

● - Locations where samples for shear testing of fractures were taken, /Lanaro, 2002a,b/; ✧ - Location where samples for uniaxial compressive strength were taken.

Figure 5-9c. *Confidence levels in cells – Layer 3.*

Layer 4 (–470 – –500 m).



K – block(unit) number; boundaries of blocks (fracture zones); 389 - cell number; Borehole sections in cells.

- cells with confidence level 1; - cells with confidence level 2; The rest of the cells have confidence level 3.

Figure 5-9d. *Confidence levels in cells – Layer 4.*

The classification systems are here applied for obtaining the ratings independently on the water pressure (J_w, RMR_{water}), stress state (SRF) and possible orientation of the excavations ($RMR_{orientation}$). Thus the following values were adopted:

<u>Q-system:</u> $J_w = 1$, SRF = 1.

<u>RMR-system:</u> $RMR_{water} = 15$, $RMR_{orientation} = 0$.

5.9.1 550 m Model

The project requires presenting the results in histogram forms as much as possible in order to represent the spatial variability of the parameters. For some rock units, these histograms are possible because enough data are available (Figure 5-10–Figure 5-12). For others, only ranges or single values can be produced.

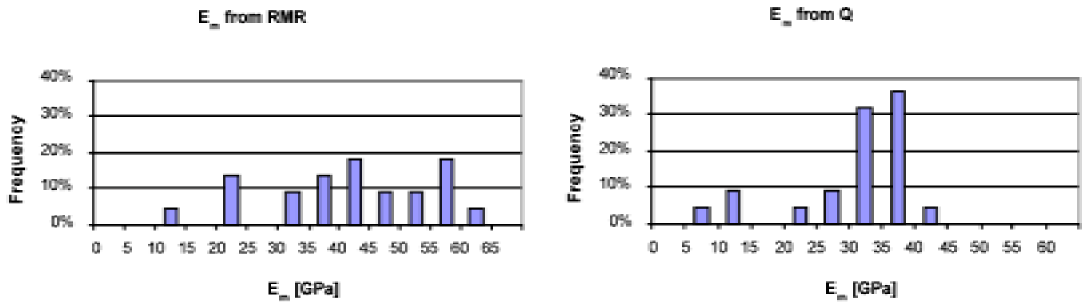


Figure 5-10. *Histograms of the deformation modulus for Block H.*

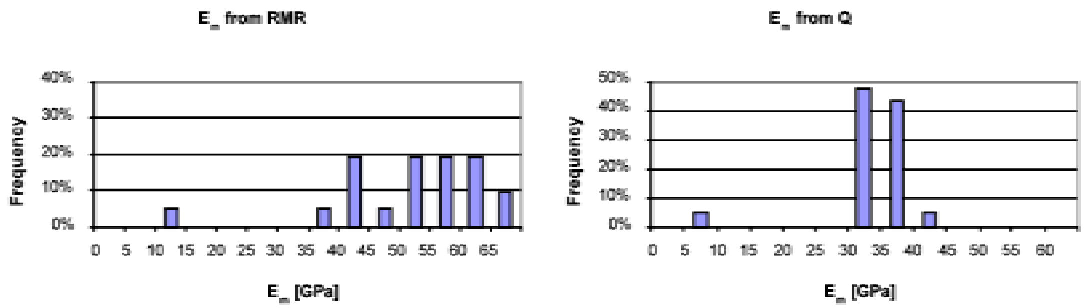


Figure 5-11. *Histograms of deformation modulus for Block I.*

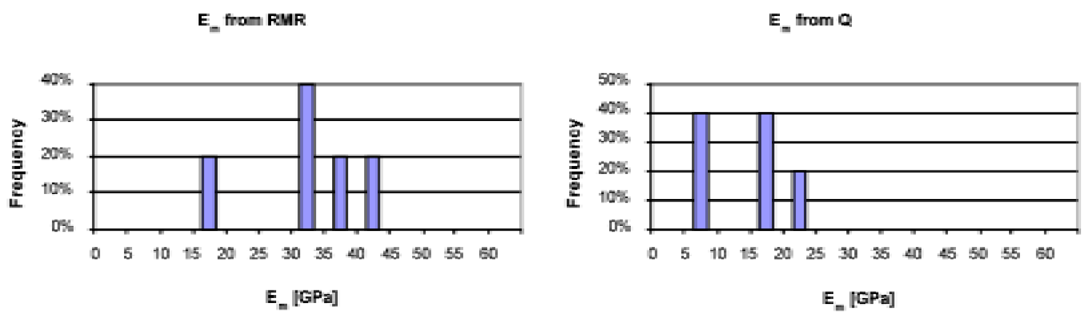


Figure 5-12. *Histograms of the deformation modulus for Block J.*

The classification results according to the Q- and RMR-system are presented in Table 5-12 and Table 5-13, respectively. In the tables, the following statistics of the ratings are reported:

Mean value: this is the average value obtained for the rock unit or the only obtained value from the characterisation (in case only one borehole section or only surface mapping were available);

Standard deviation: this is provided if there are more than three rating results. Where the number of the rating values is insufficient, no standard deviation is provided.

Maximum and minimum: the maximum and minimum rating is provided for the rock unit to give information about the spatial variability of the rating inside the rock unit or the target cell.

Table 5-12. Ratings and deformation modulus for characterisation of the rock unit using Q-system.

Rock Unit	Q				E_m (GPa)			
	Mean	Stdev	Max	Min	Mean	Stdev	Max	Min
A	11.1	–	–	–	26	–	42	10
B	0.5	–	–	–	8	–	–	–
C	0.5	–	–	–	8	–	–	–
D	0.5	–	–	–	8	–	–	–
E	4	–	4	4	15	0	25	6
F	0.5	–	–	–	8	–	–	–
G	33	–	–	–	38	–	61	15
H	13	7	33	0.1	25	9	61	2
I	15	5	33	1	29	6	61	1
J	2	1.5	4	0.5	9	6	15	2
K	1.5	–	–	–	4	–		
L	33	–	–	–	38	–	61	15
M	0.1	–	–	–	1	–		
N	33	–	–	–	38	–	61	15

(E_m is evaluated according to /Barton, 1983; Grimstad and Barton, 1993; Singh, 1997/)

Table 5-13. Ratings and deformation modulus for characterisation of the rock unit using RMR-system.

Rock Unit	RMR				E_m (GPa)			
	Mean	Stdev	Max	Min	Mean	Stdev	Max	Min
A	85	–	–	–	75	–	–	–
B	51	–	–	–	11	–	–	–
C	51	–	–	–	11	–	–	–
D	51	–	–	–	11	–	–	–
E	75	–	78	74	48	7	53	40
F	51	–	–	–	11	–	–	–
G	76	–	–	–	53	–	–	–
H	71	8	80	46	37	13	56	8
I	76	7	82	47	47	13	63	8
J	66	8	73	53	27	10	38	12
K	44	–	–	–	7	–	–	–
L	84	–	–	–	71	–	–	–
M	34	–	–	–	4	–	–	–
N	84	–	–	–	71	–	–	–

(E_m is evaluated according to /Serafim and Pereira, 1983/)

From the RMR ratings, the strength properties of the rock units are calculated according to /Hoek and Brown, 1997/ (Table 5-14). Two levels of the confining pressure are considered for the calculation. The higher confining pressure corresponds to the in-situ stress level at the depth of the Target Area, and the lower confining pressure level was used for highlighting non linearity of the Hoek and Brown curve, and corresponds to possible stress relief effects induced by excavation.

Table 5-14. Cohesion and friction angle by /Hoek and Brown, 1997/ of the rock units with two levels of the confining pressure.

Rock Unit	RMR	c 0–5 MPa [MPa]	c 10–20 MPa [MPa]	ϕ 0–5 MPa [deg]	ϕ 10–20 MPa [deg]	$\sigma_{cm(H-B)}$ [MPa]
A	85	8	20	61	50	60
B	51	2	7	43	26	5
C	51	2	7	43	26	5
D	51	2	7	43	26	5
E	77	6	16/2.6*	57/8*	44/7.3*	36/5.2*
F	51	2	7	43	26	5
G	76	5	18	62	49	38
H	71	5	16/2*	60/1*	47/2*	31/12*
I	76	6	17/2*	61/2*	47/3*	37/13*
J	66	3	15/3*	58/3*	38/13*	21/10*
K	44	1	6	44	23	3
L	84	8	20	62	51	59
M	34	1	5	35	18	1
N	84	8	20	62	51	59

* mean value/standard deviation

5.9.2 Target Area – 4–500 m Model

The Q and RMR ratings, and the deformation modulus calculated from them are produced for each cell and given in Table 5-15. For cells at the border between competent rocks and fracture zones, two sets of values are presented.

The ratings of the cells are produced according to the following rules of thumb:

1. For cells of confidence level 1, the Q and RMR ratings and their associated parameters are all calculated directly from local core sections and mechanical test results with samples taken from these cells, which may be therefore different from the mean values of the blocks/units;

2. For cells of confidence level 2, the Q and RMR ratings and their associated parameters are given as the same as that of the confidence 1 cells which they surround;

3. For the cells of confidence level 3, the Q and RMR ratings and their associated parameters are given as the mean values of the block/unit they belong to.

This technique was applied to determine the ratings and mechanical properties of all the 480 cells of the target Area. For the cells with confidence level 1, results are presented in Table 5-15 and Table 5-16.

Table 5-15. Q- and RMR-ratings for the cells the Target Area and the deformation modulus obtained by /Barton, 1983; Grimstad and Barton, 1993/, and /Singh, 1997/ from Q, and /Serafim and Pereira, 1983/ from RMR.

Cube ID	Rock Unit	Northing [m]	Easting [m]	Z depth [m]	Q Mean	RMR Mean	E_m Mean Q [GPa]	E_m StDev Q [GPa]	E_m Mean RMR [GPa]	E_m Stdev RMR [GPa]
4	E	7333.953	1933.815	–395	3.9	72	15	–	35	–
4	H				7.68	71	22	–	33	–
5	H	7335.785	1963.759	–395	6.45	67	20	3	28	13
50	H	7285.054	2117.142	–395	14.76	76	29	2	45	12
105	H	7186.065	1972.916	–395	15.92	75	30	–	42	–
124	E	7333.953	1933.815	–425	2.81	72	15	–	35	–
150	H	7314.998	2115.311	–425	16.2	78	30	–	50	–
170	H	7285.054	2117.142	–425	16.67	80	31	–	56	–
270	H	7314.998	2115.311	–455	15.83	77	30	0	47	4
389	H	7313.167	2085.367	–485	15.81	77	30	1	49	6

Table 5-16. Cohesion and friction angle by /Hoek and Brown, 1997/ for the target cells for two levels of confining pressure.

Cube ID	Rock Unit	c 0–5 MPa [MPa]	c 10–20 MPa [MPa]	ϕ 0–5 MPa [deg]	ϕ 10–20 MPa [deg]	$\sigma_{cm(H-B)}$ [MPa]	RMR char.
4	E	4	17	61	48	30	72
4	H	4	16	61	47	28	71
5	H	4/2*	15/3*	59/3*	44/4*	22/14*	67
50	H	5/2*	17/3*	60/2*	47/4*	35/14*	77
105	H	5	17	60	47	35	75
124	E	4	17	61	48	30	72
150	H	6	18	61	48	41	78
170	H	6	18	61	49	46	80
270	H	6/0*	17/0*	61/0*	48/0*	39/0*	77
389	H	6/1*	18/1*	61/0*	48/1*	40/5*	77

* mean value/standard deviation

5.9.3 Ramamurthy's approach

Ramamurthy's Criterion /Ramamurthy, 1995/ for determining the strength and deformation modulus is based on the equations presented in Section 2.3.7. Table 5-17 shows the results of this approach for some of the target cells. Some discussion about these results and their comparison with other approaches is also given in Section 5.9.6 and 5.9.7. Both deformation modulus and rock mass strength estimated according to Ramamurthy's Criterion appear to be higher than all the other approaches.

Table 5-17. Determination of the mechanical properties for the target cells according to /Ramamurthy, 1995/.

Cube ID	Rock Unit	σ_{cm} [MPa]	E_m best case [GPa]	E_m worst case [GPa]
4	E	146	60	17
4	H	167	64	19
5	H	88	69	23
50	H	144	60	23
105	H	170	66	27
124	E	137	67	14
150	H	–	66	31
170	H	172	65	39
270	H	164	65	25
389	H	151	64	25

5.9.4 RMi approach

The calculation is mainly based on the volumetric joint count, Jv, from the RQD value as presented for each cell in Table 5-18. The jC value (joint condition factor) is estimated from the rock characterization data processing forms used to determine Q- and RMR-values. It turned out that it was difficult to estimate the parameter for the joint length, jL. As the selection is dependent on the interpretation of the fracture persistence, this is difficult to be determined using borehole data. Therefore this rating was calculated by averaging the rating for both continuous and discontinuous joints to determine the jL value. As it can be observed in Table 5-18 the RMi-ratings fall in the range of 10–100 and implies "very strong rock mass" and the RMi value as "very high".

In Table 5-18, the deformation modulus and the rock mass strength were calculated according to Eqs.(12) and (51). RMi was calculated for some cells for the Äspö Test Case using two possible values of jC and jP. For each cell, the uniaxial compressive strength of the rock mass was then given as the average of these extreme values. This approach seems to give rock mass strength of the same order of magnitude as those obtained by the Hoek and Brown approach (cf. Figure 5-16). On the other hand, the deformation modulus is the lowest obtained from empirical relations (cf. Figure 5-15).

Table 5-18. Rock mass strength and deformation modulus calculated using RMi.

Cube ID	Jv	jC	jP	RMi	σ_{cm} Strength [MPa]	E_m [GPa]
50-H	5.3	0.7	0.15	28.2		
	5.3	1.3	0.20	37.6	329	21
105H	7.5	0.7	0.1	18.3		
	7.5	1.3	0.14	25.6	22	18
124-E	7.6	0.7	0.25	47		
	7.6	1.3	0.38	71.4	59.2	26
150-H	5.9	0.7	0.103	18.8		
	5.9	1.3	0.2	36.3	27.7	19
170-H	5	0.7	0.18	32.9		
	5	1.3	0.25	45.7	39.3	22
270-H	5.6	0.7	0.105	19.2		
	5.6	1.3	0.15	27.5	23.3	18
389-H	6.5	0.7	0.09	16.9		
	6.5	1.3	0.107	20.1	18.5	17

5.9.5 Relation between Q and RMR

The Q- and RMR-ratings were determined for two purposes: characterisation and design evaluation. This was achieved by taking into account (for design) or by disregarding (for characterisation) the effect of water pressure, stresses, direction of excavation and the orientation of the fracture sets. For this reason, the ratings obtained for design and characterisation do not necessarily coincide.

The results obtained by the two independent classification systems are compatible with the results published in the literature. The relation between Q and RMR derived for the Äspö Test Case, and for design purposes, closely resembles the published ones (Figure 5-13).

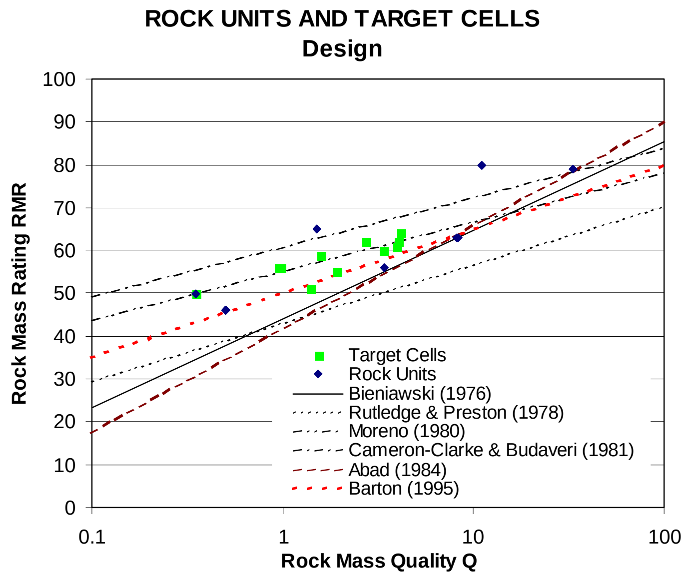


Figure 5-13. *Design: Q-rating versus RMR-rating for the rock units and the target cells.*

On the other hand, the same relations seem to underestimate the RMR determined for the Äspö Test Case as a function of Q, when the results concerning the characterisation of the site are considered (Figure 5-14). This is in part due to the fact that the effect of the water, of the stress level and of the supposed orientation of the excavation are neglected. However, as it was shown by /Goel et al, 1995/, those relations applies only for the "design" configuration considered in the present study, and different equations should be developed for the "characterisation" configuration.

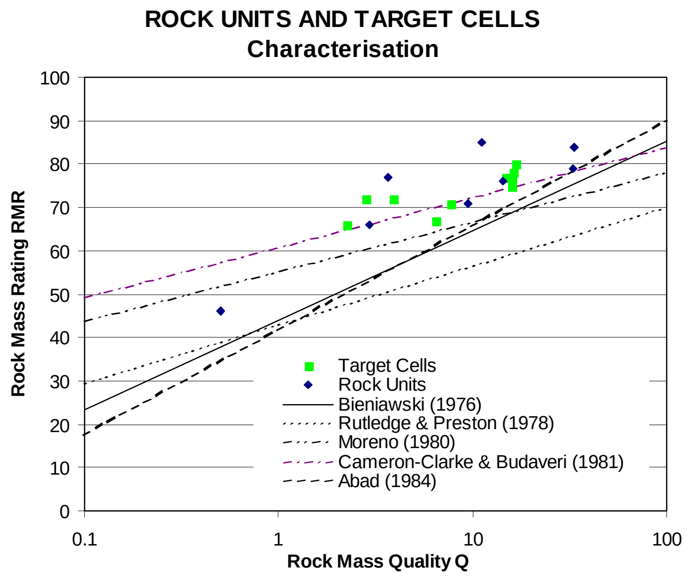


Figure 5-14. *Characterisation: Q-rating versus RMR-rating for the rock units and the target cells.*

5.9.6 Characterisation ratings and rock mass deformation modulus

The deformation modulus of the rock mass obtained according to /Bieniawski, 1978; Serafim and Pereira, 1983; Grimstad and Barton, 1993; Singh, 1997; Ramamurthy, 1993, 1995; Palmström, 1995/ are plotted for each rock unit and target cell against the RMR for characterisation of the cell or rock unit. The choice of RMR for rating the rock mass in the rock unit or target cell is completely arbitrary since not all of the relations for determining the deformation modulus are function of RMR. However, this comparison is suitable for observing the range of variation of the deformation modulus depending on the relation used for the calculation.

In Figure 5-15, the four relations are compared for the target cells. It immediately appears that the spreading of the value is quite wide. Furthermore, in consequence of the choice of RMR as classification parameter for the plot, all methods based on the strength of the rock mass (RMi and Ramamurthy's) don't show any well defined behaviour, as it happens for Bieniawski's and Serafim and Pereira's methods. Q is correlated with RMR through a logarithmic equation, thus the modulus determined by Q should plot according to a logarithmic curve against RMR. More important conclusions can be made about the range of the values of the modulus. Three groups of values can be recognised in the Test Case result:

1. High deformation modulus (50–60 GPa): these are mainly the values obtained by Ramamurthy's method ("best case"). These results do not seem to mirror any increase when increasing the RMR of the target cells; on the contrary, it appears that most of the target cells with low RMR correspond to the highest values of the deformation modulus. Because the method is new and the Empirical group has not enough familiarity with the method, this is considered only for comparison.

2. Medium deformation modulus (25–50 GPa): these values are obtained by Bieniawski's and Serafim and Pereira's classical equations. These values are directly related to RMR, thus the graphs show nice smooth curves. There is a neat increase of the modulus with increasing RMR. The two approaches give the same results for the better classes of the rock mass (towards 80), while Bieniawski's formula gives higher values for lower RMR compared to Serafim and Pereira's.

3. Low deformation modulus (15–25 GPa): these are obtained by the relation provided by Grimstad and Barton, Ramamurthy ("worst case") and Palmström. The moduli in this study are obtained from the characterisation parameters (Q'). The equation provided by Grimstad and Barton is instead obtained to relate Q and the deformation modulus. This means they are not completely compatible. Moreover, as it is shown in Figure 5-15, while for design the relation between RMR and Q is the same as the one observed by several authors, for characterisation there seems to be a shift of Q towards lower values for the same RMR. In turn, when using the Grimstad and Barton's formula for the mean value, the deformation modulus is lower than for the other methods. The relation by Palmström is quite new to the Empirical group, thus it is difficult to judge its validity. Equation (51) relates the strength of the rock mass with the modulus of deformation.

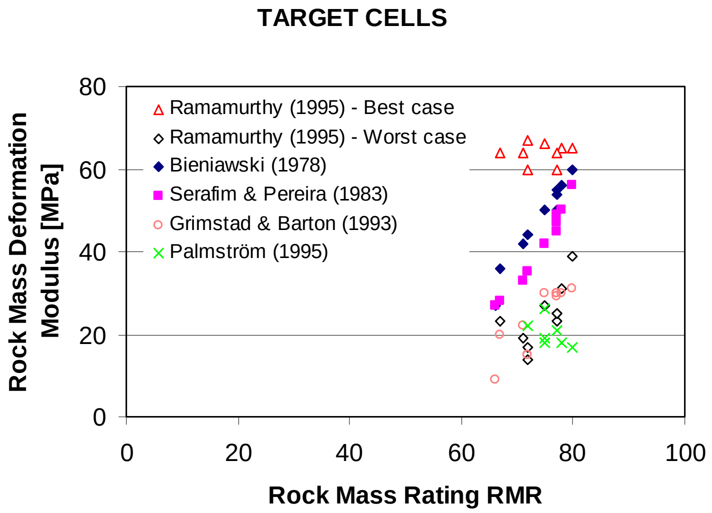


***Figure 5-15.** Comparison of the deformation modulus obtained with different relations for the Target Cells.*

Table 5-19. Resume of the results of the rock mass deformation modulus from the five different methods for the target cells.

Cube ID	Rock Unit	E_m /Bienawski, 1978/ [GPa]	E_m /Serafim and Pereira, 1983/ [GPa]	E_m /Grimstad and Barton, 1993/ [GPa]	E_m /Palmström, 1995/ [GPa]	E_m Best case /Ramamurthy, 1995/ [GPa]	E_m Worst case /Ramamurthy, 1995/ [GPa]
4	E	44	35	15		60	17
4	H	42	33	22		64	19
5	H	36	28	20		64	23
50	H	50	45	29	21	60	23
105	H	50	42	30	18	66	27
124	E	44	35	15	26	67	14
150	H	56	50	30	19	66	31
170	H	60	56	31	22	65	39
270	H	54	47	30	18	65	25
389	H	55	49	30	17	64	25

As observed before, results form Bieniawski's and Serafim and Pereira's methods are very similar to one another, and they range in between the extreme minimum values provided by Palmström, and the extreme maximum values provided by Ramamurthy. For this reason, the equation by Serafim and Pereira correlating RMR and deformation modulus of the rock mass will be used for characterising the target cells in the result Table 5-19.

5.9.7 Characterisation ratings and rock mass strength

The three relations between ratings and rock mass strength investigated in this study are compared in Figure 5-16, where the values of the rock mass strength are plotted against the RMR value of the respective target cell. It can be observed that results by Hoek and Brown's and Palmström's equations are very similar (it should be kept in mind that Hoek and Brown's equation uses GSI, which is directly obtained from RMR for characterisation). It can be observed that the range of variation of the two results is very consistent and between the range 18–60 MPa. Concerning Ramamurthy's Criterion, the values of the rock mass strength are very high, almost of the same order of magnitude as the uniaxial compressive strength of the intact rock in the cells. Considering the consistence between Hoek and Brown's and Palmström's results, they can be considered almost equivalent.

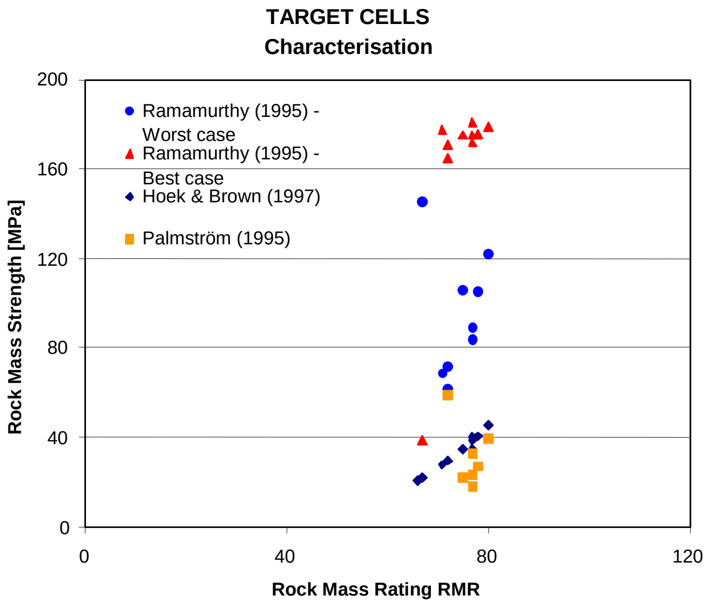


__Figure 5-16.__ Comparison between the rock mass uniaxial compressive strength obtained with different methods: /Hoek and Brown, 1997; Ramamurthy, 1995; Palmström, 1995/.

Table 5-20. Resume table of the rock mass strength obtained by three methods for the target cells.

Cube ID	Rock Unit	σ_{cm} /Hoek and Brown, 1997/ [MPa]	σ_{cm} /Palmström, 1995/ [MPa]	σ_{cm} /Ramamurthy, 1995/ [MPa]
4	E	30	–	146
4	H	28	–	167
5	H	22	–	88
50	H	35	33	144
105	H	35	22	170
124	E	30	60	137
150	H	41	28	–
170	H	46	39	172
270	H	39	23	164
389	H	40	18	151

Because RMR method and Hoek and Brown's Criterion are the mostly used in the practice, and because they appear to give values of the rock mass strength in between the values provided by the other methods, it is here chosen to assign Hoek and Brown's parameters to the target cells. Consequently, also the Hoek and Brown's parameters for characterising the shear strength of the rock mass are chosen. The values of the equivalent cohesion c and friction angle ϕ for two stress levels required by the Project (0–5 MPa and 10–20 MPa) are reported in Table 5-21.

Table 5-21. Cohesion and friction angle by /Hoek and Brown, 1997/ for two levels of the confining pressure for the target cells.

Cube ID	Rock Unit	c 0–5 MPa [MPa]	c 10–20 MPa [MPa]	ϕ 0–5 MPa [deg]	ϕ 10–20 MPa [deg]
4	E	4	17	61	48
4	H	4	16	61	47
5	H	4/2*	15/3*	59/3*	44/4*
50	H	5/2*	17/3*	60/2*	47/4*
105	H	5	17	60	47
124	E	4	17	61	48
150	H	6	18	61	48
170	H	6	18	61	49
270	H	6/0*	17/0*	61/0*	48/0*
389	H	6/1*	18/1*	61/0*	48/1*

* mean value/standard deviation

5.9.8 Characterisation results by using geophysical methods

There are very few geophysical data that can be used for determination of the rock mass ratings within the test area. Only three seismic profiles were available. The Q values obtained from the seismic velocity analysis, vary between 1 and 5.This agrees well with the ratings obtained from the rock mass classification for blocks A, B, C, D, E, I and L. The calculated Q-ratings for the rock units and fracture zones presented in Table 5.22 are obtained according to Eqs. (26) and (27). The mean deformation modulus was then calculated according to Eqs. (42) and (43). The Q-rating and deformation modulus produced using seismic velocity data appear to be higher that those obtained using borehole data (cf. Table 5-12).

Table 5.22. Q-ratings and deformation modulus derived from geophysical methods.

Block	Q-rating	E_m, Deformation modulus [GPa]
A	37 mean 0.1–5* zone	39
B	38** mean 1.6 zone	39 5
E	36** 2.5 zone	39 10
I	33 mean 0.1–4*zone	38 0–15
J	–	–
K	–	–
L	35 mean 0.1–1*zone	37 –
M	–	–

*= weakness zones inside the rock unit

**= Value at the margin of the RVS fracture zone

5.10 Comparison of the results with core logging and DFN fracture model

5.10.1 Comparison with the “reference estimation”-model

The Norwegian Geotechnical Institute (NGI) has independently performed rock mass characterization for this Project, using Q and RMR for the three boreholes KAS02 (Figure 5-17 and Figure 5-18), KA2511A and KA2598A /Makurat et al, 2001/[1]. In this section we compare their rating values with those presented by BBK and the differences are discussed. The comparison is made with ratings calculated with SRF = 1 and Jw =1, and RMR_{water} =15, $RMR_{orientation}$ =0, for Q- and RMR-systems respectively.

[1] The difference is that all available data from the test were at disposal for the rating.

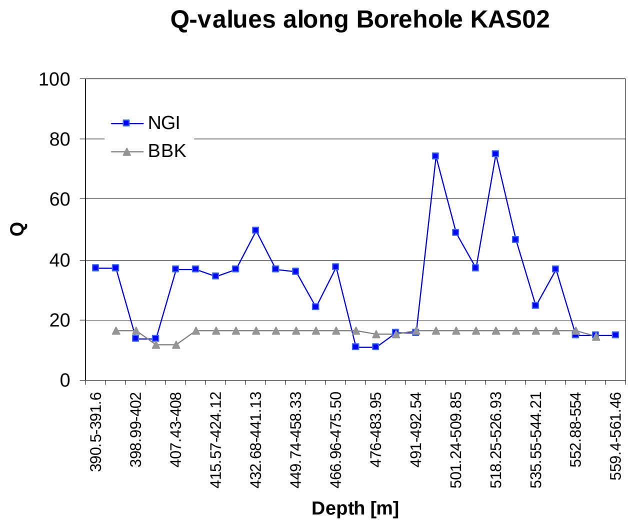


Figure 5-17. *Comparison of the Q-rating calculated by BBK and NGI for characterisation of the rock mass along borehole KAS02 (SRF = 1 and Jw =1).*

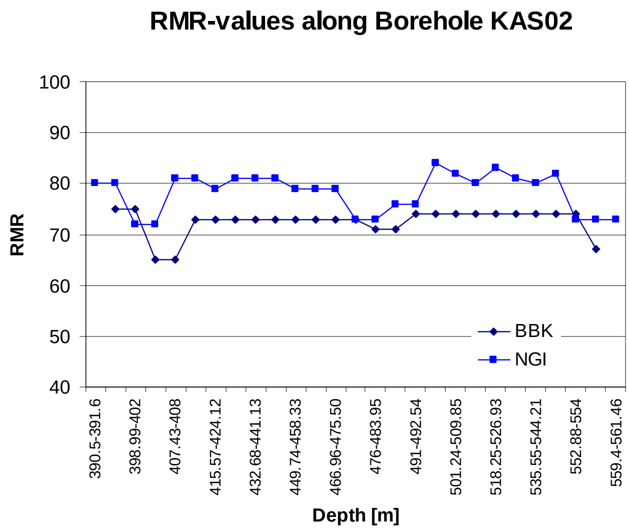


Figure 5-18. *Comparison of the RMR-values calculated by BBK and NGI for characterisation of the rock mass along borehole KAS02 (RMR_{water}=15; $RMR_{orientation}$=0).*

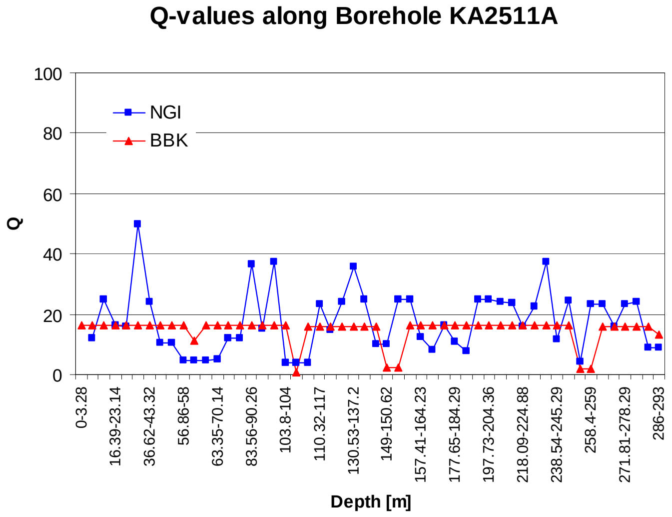


Figure 5-19. *Comparison of the Q-rating calculated by BBK and NGI for characterisation of the rock mass along borehole KA2511A (SRF = 1 and Jw =1).*

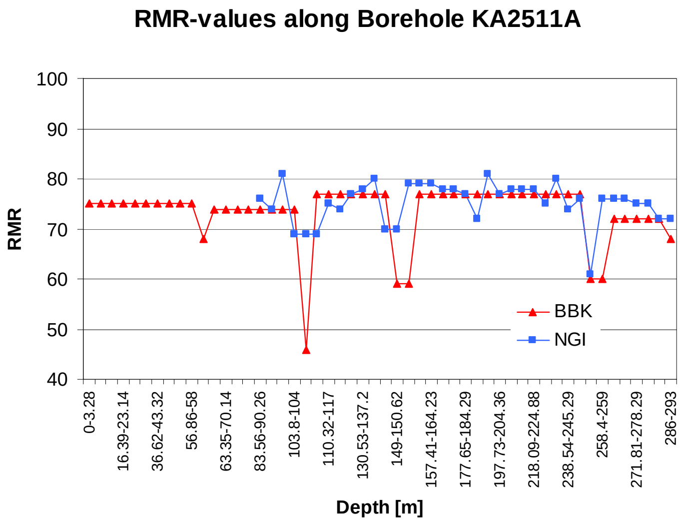


Figure 5-20. *Comparison of the RMR-values calculated by BBK and NGI for characterisation of the rock mass along borehole KA2511A (RMR_{water}=15; $RMR_{orientation}$=0).*

In general it is found that the Q-values by NGI are higher than those by BBK. This is especially appreciable for KAS02 and KA2598A, while for borehole KA2511A matching is good (Figure 5-19 and Figure 5-20). However, the ratings are in the same range of Q, between 10 and 40. For KAS02 and KA2598A the NGI-values are higher than BBK values. It should be noted that the Q-values by NGI have a larger spread than that of BBK. This is the effect of the different lengths for each domain of core section on which they are calculated. NGI has in most cases used the length of the core box as domain length (about 8 m), while BBK has based the division in domain on sections with almost constant RQD. As the rock is rather homogenous with respect to the fracture frequency, the BBK-domain length is usually much larger than that presented by NGI (Figure 5-21).

The reason for larger Q-values from NGI seems mainly to depend on two parameters, namely Jn and Jr. BBK has for most of the domains used Jn = 6, based on the number of fracture sets observed on the stereographic projection of the data from the core logging (e.g. two main fracture sets plus some random fractures). NGI has used a Jn-value which varies with the logging but in general the values are lower than 6 (Figure 5-22). The Jr-parameter has for BBK a value normally around 2 for undulating and planar fractures. The selection of that value is based on SICADA data where most fractures have a value of JRC around 8. Also inspection on the ground surface and tunnel walls indicates that the fractures are mostly undulating and planar. NGI, instead, has in general used a value of 3 for Jr representing rough surfaces. The other parameters are in good agreement between the two team's results.

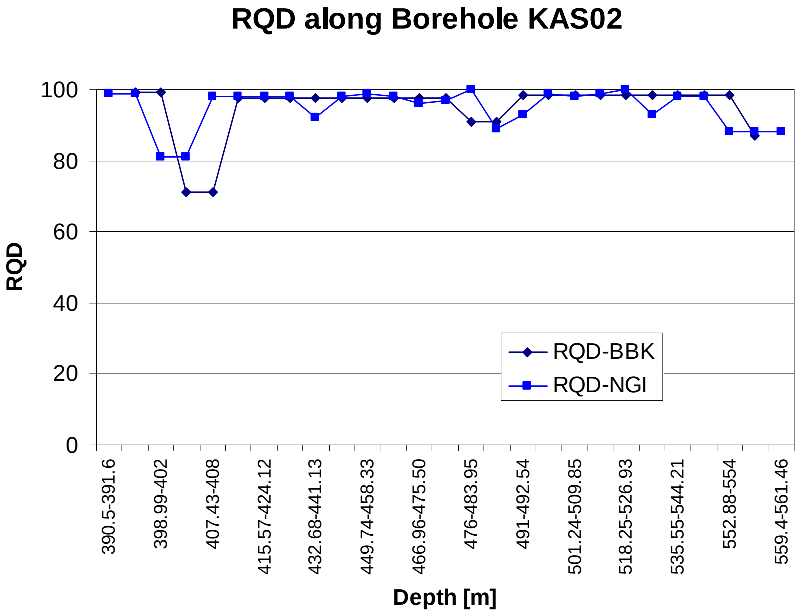


Figure 5-21. *Comparison of the RQD calculated by BBK and NGI for borehole KAS02.*

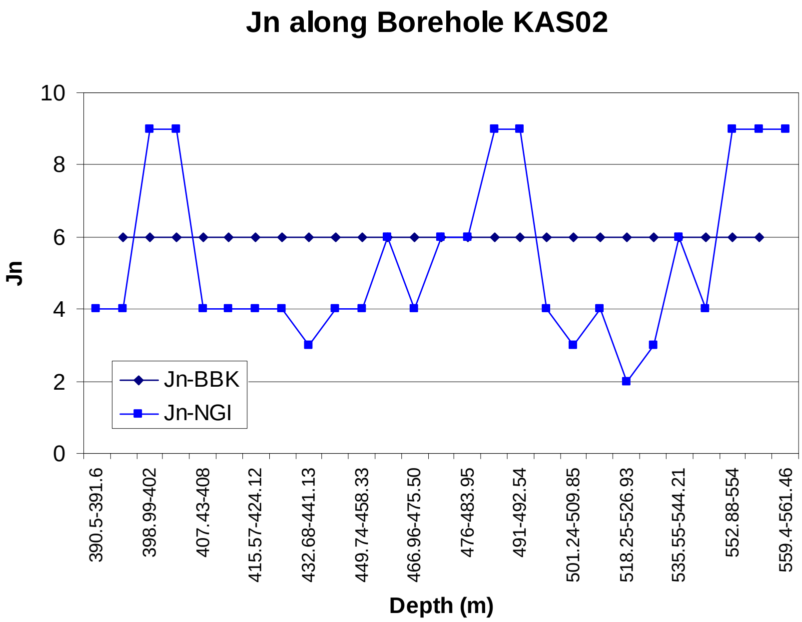


Figure 5-22. *Comparison between Jn calculated by BBK and NGI for borehole KAS02.*

Between the two sets of results, the RMR-ratings have a difference smaller than 10 points, where the BBK rating is usually lower. Comparing the single parameters, there is only one difference between the BBK and NGI results, namely in the $RMR_{fracture\ length}$. NGI has in general used the value of $RMR_{fracture\ length}$ =1, indicating that the length of the fractures is 10–20 m, while BBK has normally a value of $RMR_{fracture\ length}$ = 4, representing fracture length of 1–3m. Please note that the DFN model has an average length of the long fractures ≤ 4 m, more in favour of the BBK values. Also surface-mapping results showed a mean fracture length of 1 m.

5.10.2 Characterisation result with data from the DFN fracture model

The theoretical approach has used the DFN-model for the geometrical presentation of the fractures in the target area /Hermansson et al, 1999/. In order to compare with the approach used for the empirical model the ratings and properties in some of the cells in the Target Area have been recalculated using the fracture data from the DFN model. Results are presented in Table 5-23 and Table 5-24.

Table 5-23. Comparison between Q- and RMR-ratings for characterisation by using borehole data and DFN data for description of the fracture sets for the rock units.

Rock Unit	Q charact. (borehole)	Q charact. (DFN)	RMR charact. (borehole)	RMR charact. (DFN)
A	6	8	85	83
B	4	3	79	77
C	4	3	79	77
D	4	3	79	77
E	3.5	4.5	77	75
F	4	3	79	77
G	33	11	79	74
H	10	8	71	70
I	14	10	76	76
J	3	3	66	63
K	–	–	–	–
L	33	22	84	82
M	–	–	–	–
N	33	22	84	82

Table 5-24. Comparison between Q- and RMR-ratings for characterisation by using borehole data and DFN data for description of the fracture sets for the target cells.

Cube ID	Rock Unit	Q charact. (borehole)	Q' charact. (DFN)	RMR charact. (borehole)	RMR charact. (DFN)
4	E	4	5	72	70
	H	8	5	71	69
5	H	6.5	4	67	65
50	H	15	10	77	75
105	H	16	10.5	75	73
124	E	3	5	72	70
150	H	16	11	78	78
170	H	17	11	80	80
269	H	16	11	77	76
389	H	16	10.5	77	77

The DFN data provides the fracture set number and ranges of trace lengths of each set of fractures, from ZEDEX tunnel mapping results. It was not used as a first-hand data in the empirical approach as described above for the following reasons:

1. The DFN data is valid mainly for the ZEDEX tunnel area, without being conditioned from surface mapping results that is valid for the large site of 550 m model. The number of sets in the DFN data is 3 and the trace length is 2–4 meters, with lower cut-off limit of 0.5 m during mapping on tunnel walls. From the surface mapping, the trace length is about 1 m or less. From the borehole logging data, the fracture set number varies between 2 and 3, plus random fractures. Therefore the DFN data is not compatible with surface and borehole logging data, although 3 sets of fractures may be taken as in the same range of set numbers from borehole data. The rating systems do not require complete fracture system realizations.

2. There are only two parameters in the Q and RMR systems that need fractures set number and trace length: the Jn in Q and $RMR_{\text{trace length}}$ in RMR. For current work, Jn is obtained from the borehole logging data directly along borehole depth/length and it is needed for estimating statistical variations. Using a constant mean value of Jn=9 valid only for the ZEDEX tunnel area will have a negative effect on statistical treatment of rating parameters as required. For trace length, the data from surface mapping over larger area is more reliable than that from any tunnel mapping over much smaller exposure areas on the tunnel walls, beside the fact that a 0.5 m cut-off limit of the mean trace length in the ZEDEX mapping is quite large, considering only 1 m mean trace length from surface mapping. Therefore the mean trace length from surface mapping was taken as the basis in the current work of the empirical approach.

3. Because the aim of the current stage of work is to establish methodology, and the common classification system practice is to use original data from boreholes and surface mapping before any tunnel mapping being available for characterization, direct use of local DFN data from underground tunnel mapping at the very start is not appropriate for establishing methodology and recommendation at the next stage of the project.

4. DFN model's main limitation is the largely unknown fracture size and shape inside the rocks, not due to the model itself but to the limitations of mapping, especially when only tunnel mapping is used. Using the DFN trace length parameter having the most severe limitation for rating calculations is not a proper way for use of the DFN models that are a very useful tool for other applications.

Despite the differences in the fracture data and approaches, the use of DFN fracture data and borehole fracture data produces very similar rating results, as shown in Table 5-23 and Table 5-24. For Q-ratings, the differences are caused mainly by the different fracture set numbers (2 or 3 sets plus random fractures in BBK work and fixed 3 sets in the DFN model). For RMR-ratings, the differences are caused by the mean fracture trace length (about 1–3 m in BBK work and 4 m in the DFN model).

In theory, the distribution of RQD along a line inside the rock mass could be simulated by the DFN model. However, this technique does not guarantee that scale effect on fracture spacing and orientation are captured. Thus, since RQD data are always available from borehole drilling, it should be recommended to use the collected data instead of DFN determination of RQD for the purpose of empirical rock mass characterisation.

6 Discussion

A summary table of the major parameters for the characterisation and classification systems in this study is contained in Figure 6-1. The available geometrical, geological and mechanical information is here divided according to the quantity and quality into three groups: i) for pre-investigations; ii) for characterisation; iii) for design.

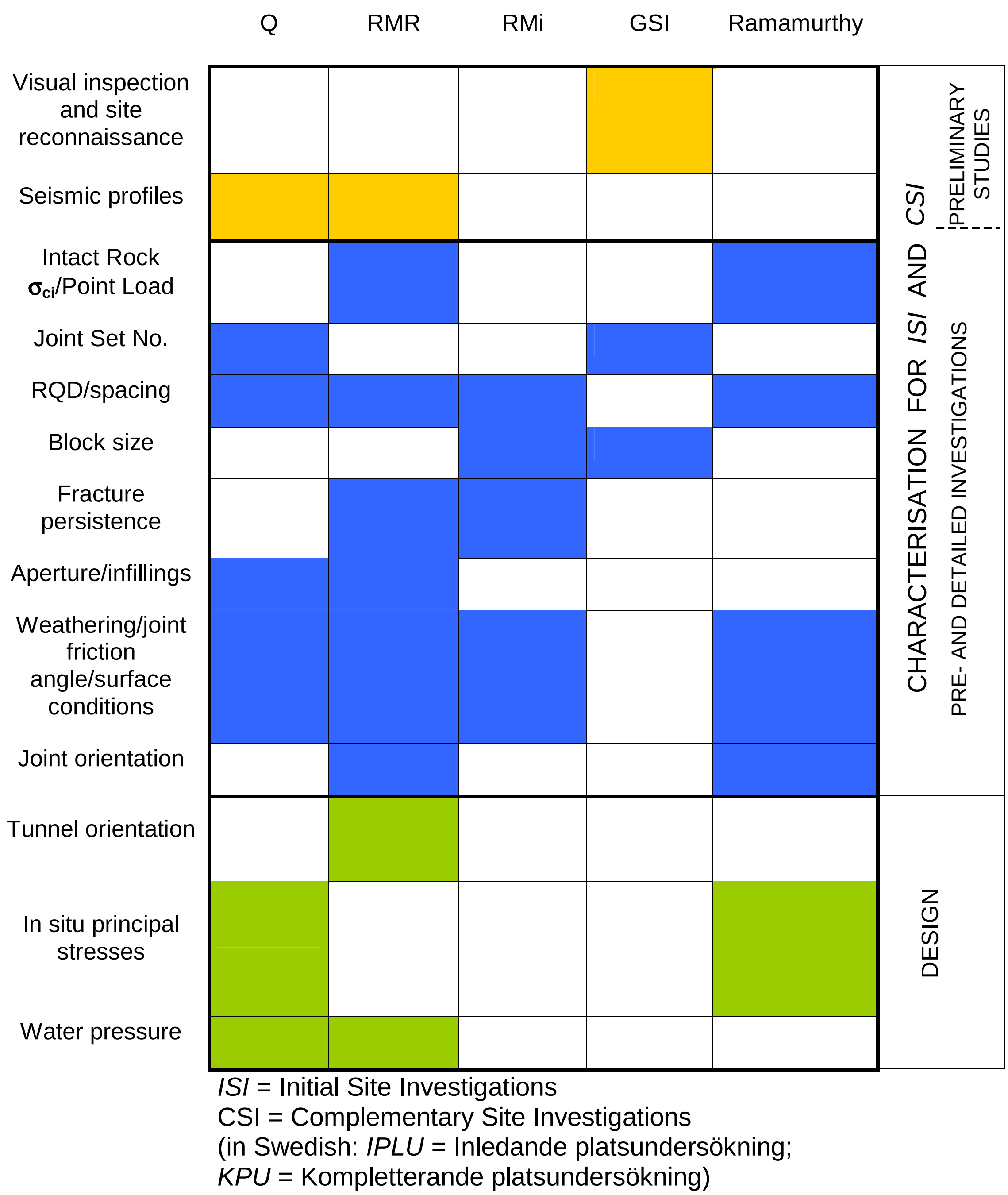


Figure 6-1. *Comparison of the input parameters for the major classification systems: Q, RMR, RMi, GSI and Ramamurthy's system.*

1. *Preliminary feasibility studies*

   There are two methods that can be applied for a preliminary evaluation of the rock mass quality based on information obtained by observation of fracture systems (e.g. GSI) and by seismic profiling (e.g. Q).

   Caution should be taken about the reliability of such preliminary evaluations of the rock quality and characterization because of the low confidence in the input data.

2. *Pre- and detailed investigations*

   Most of the characterization and classification systems use parameters recorded from surface and tunnel mappings, and borehole core logging, laboratory testing on rock fractures and intact rock. Characterization is considered as a basis for deciding the suitability of a certain site for the construction of a potential repository.

3. *Design*

   When the characterization results are evaluated in combination with the geometrical features of the excavation, and with the in-situ measurements of rock stresses and water pressure, the design of a potential repository can be performed. Many of the systems used for characterization were actually invented for classifying the rock mass according to design requirements, each of them related to a certain dimensioning and support solution. Traces of this kind of approach still remain in the structure of the characterization systems. This exigency of distinguishing between characterization and classification (for design) of the rock mass is a recent achievement.

6.1 General observations about the rock mass characterisation

6.1.1 Geological/geometrical model and available information

As explained before, the uncertainty on the result of the rock mass characterisation directly depends on the quantity and quality of the available input data. Moreover, the larger the uncertainty, the lower the reliability of the rock mass characterisation obtained. In general, the illustrated methodology for the rock mass characterisation and the derived mechanical properties provide satisfying information about rock mass quality where geological data available are sufficient. This was the case for some rock units of the Äspö Test Case on which the methodology was applied (e.g. Block H and I). A demonstration of this is given by the good agreement between the characterisation results presented in this Report and the results by NGI, those been based on a much wider geological database. However, for some deformation zones, the basic geological information provided was reputed insufficient or completely absent for providing a satisfying estimation of the properties in those volumes (e.g. Block K and L,Äspö Test Case). An attempt of characterisation was made on the basis of unclear site geology and engineering judgment. These results have naturally a quite low degree of reliability. In practice, this would imply a need for collecting more information for improvement.

Some of the blocks of the Äspö Test Case extend from the surface to 550 m depth. Because the geological information is not equally accurate at different depth, it results in that the geology/geometry of the blocks might not reflect the reality at certain locations. This is the reason why some blocks belonging to the same geological unit exhibit quite different rock quality. This is particularly the case of the blocks that belong to the deformation zones EW1a and EW1b (Block B, C, and D, on one side, and Block E). The knowledge about the two deformation zones is not equally accurate, and so is their geometrical definition at depth.

The mechanical properties are evaluated as average values for each rock unit for the 550 model. This means that, especially for the deformation zones, the properties can be very different in different locations within the rock unit, and this can be overlooked by the adopted averaging technique. It was also observed that between volumes of good rock quality there are minor weakness zones that can extend for some meters in width. The quality of those sections is comparable with the quality of the major deformation zones but their extent is limited. This kind of features, even if occurring in volumes of relatively intact rock should be highlighted for its importance for design. With the averaged mechanical parameters of the rock mass, also the maximum and minimum properties of a certain volume of the rock mass should be given. This will give a measure of the rock mass heterogeneity and point the attention to the weakness zones on one side.

6.1.2 Main characterisation systems and their peculiarities

Q-SYSTEM: The system is based on a product/division of parameters, therefore a small variation of an individual parameter can cause large changes of Q-values of even one order of magnitude. The comparison between the NGI's and BBK's results indicate that the Q-system is very sensitive to the selection of the parameters for joint set number Jn, roughness number Jr, Stress Reduction Factor (SRF) and joint water factor Jw. The SRF is of great importance for design of tunnels and rock mass classification (i.e. definition of support classes), but is not very suitable for the characterisation of the rock mass. The reason is that stress-dependence of rock properties can only be properly considered by using well defined and validated constitutive models, which are not included in the classification and characterisation systems. Therefore, one should exclude the influence of the *boundary conditions*, loading factors and support techniques in characterisation /Palmström et al, 2001/. In this way, there is no needs for adjusting the SRF factor for *characterisation* of the rock mass. In this project, this was easily done by setting SRF equal to 1 and the same applies to Jw for water pressure effect /see Almén, 2002/.

The Q-system provides also equations that relate the ratings with the seismic velocity in the rock mass. This technique can be used as a quick technique for evaluating rock quality during pre-investigations from the surface information. Its accuracy and correlation with the rock mass rating systems needs further validations for site-specific conditions. In addition, the correlation with depth depends on both the measuring techniques and stress and water conditions.

RMR-SYSTEM: This classification method is based on a summation of rating parameters, thus a small variation of a single parameter usually does not have a large effect on the total rating result. Moreover, the large number of parameters involved requires quite detailed descriptions of many of the geological features of the rock mass and fractures. The classes of rock mass are divided according to a linear scale, where each class has an equally wide rating interval. This makes the method not particularly sensitive when it comes to weaker rocks and deformation zones in hard rocks.

Different from the Q-system that was developed mainly based on experiences in hard and good quality rocks, the RMR-system was derived from experiences in sedimentary, metamorphic and hard rocks. Thus, the definition of "poor" rock masses can be slightly different when using Q- or RMR-systems: what is relatively poor in hard rock masses can be seen as fair rock in sedimentary and metamorphic rock masses where the weakest part can be sometimes classed as soil.

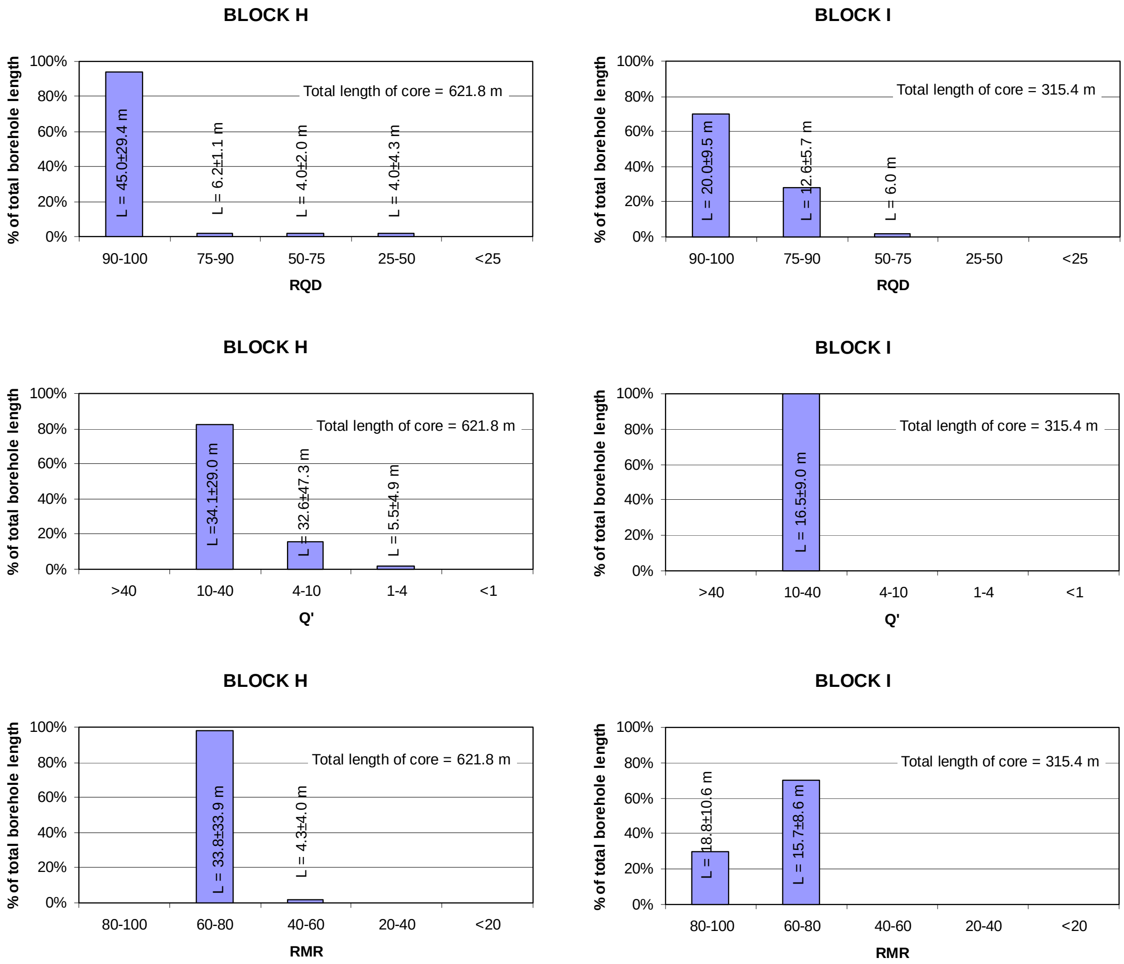
For characterisation purposes, the water rating parameter should be set to 15. The tunnel orientation can be considered for design, but it should be neglected for characterisation by setting the rating parameter to either 0 or –5, "favourable" orientation or "fair" orientation with respect to fracture orientation, respectively /see Almén, 2002/.

6.1.3 Extrapolation of the properties outside the investigated volume

The characterisation of several hundred meters of borehole cores makes it possible to recognise some patterns that are useful for the extrapolation of the data to larger volumes of rock of similar geological settings. One possibility is to analyse the length of homogeneous sections of rock mass in the borehole. For this purpose, RQD, Q and RMR value ranges are subdivided in classes of quality as in Table 2-1, Table 2-2 and Table 2-3.

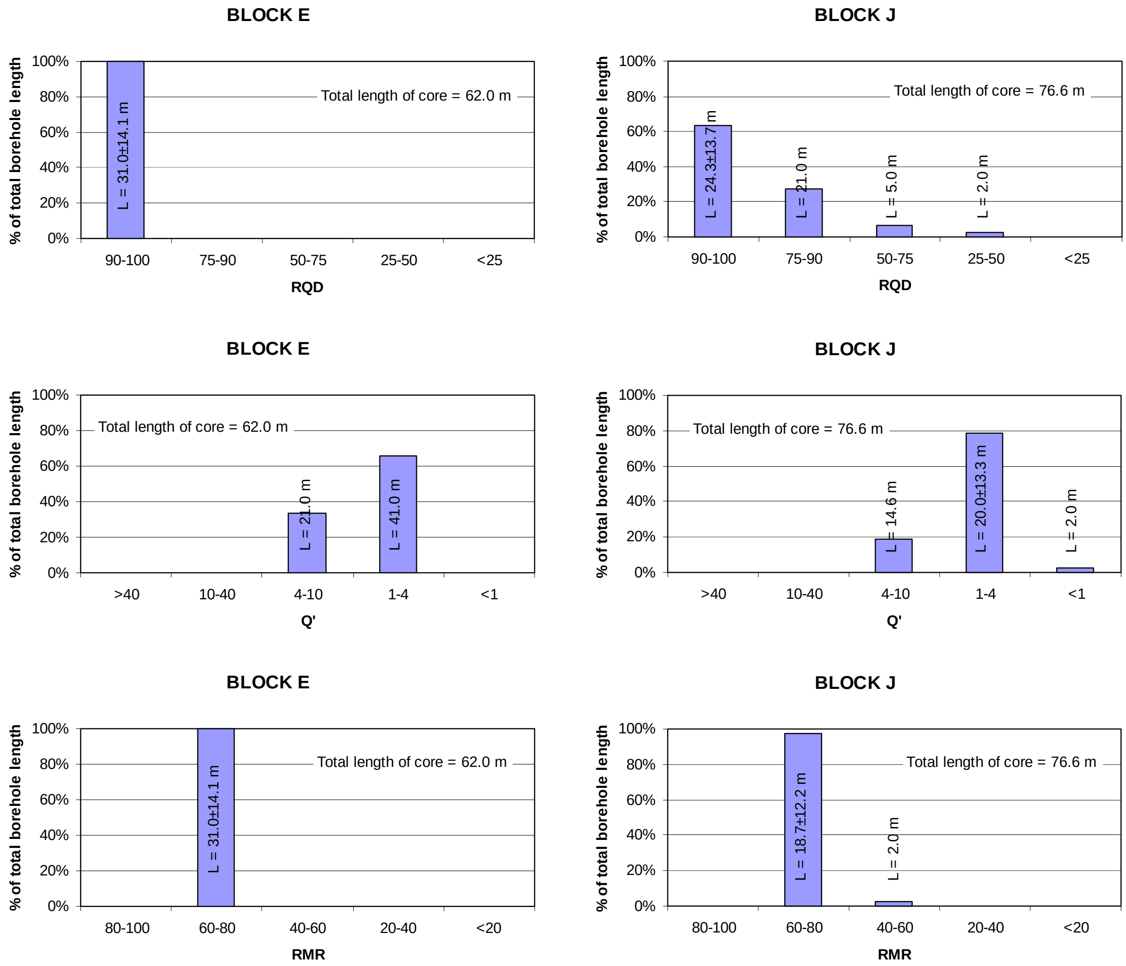
Interesting information can be obtained from the analysis of the length of borehole sections with rock that belongs to the same rock class. It can be observed that the three parameters do not give exactly the same partitioning of the rock mass along the boreholes. The fact that a borehole section has the highest RQD does not imply that the rock mass quality from RMR and Q are the highest, as it is shown in Figure 6-2 for competent rock. The diagrams also show the average length of borehole within the same rock class. This can be of help when trying to extrapolate information to rock volumes with no geological information. One can infer the rock mass quality of adjacent points with a reasonable confidence depending on the average volume (length of borehole) of rock with the same properties recorded at the site. However, the classification according to RQD does not coincide with that of the other rock classification systems due to all the other geological parameters involved in the characterisation.

Figure 6-3 shows a typical example where the statistics of the length of borehole within the same rock class for a deformation zone. This can appear quite differently depending on the accuracy of the geological/geometrical model. As observed before, Block J belongs to NE-2, while Block E to EW1b. The first deformation zone is known in detail since there are a series of tunnels intersecting it. On the other hand, the second deformation zone EW1b is very poorly known, especially at depth. This is the reason why the rock along borehole KA2598A can appear very good despite the fact that it should have crossed the deformation zone. The confidence on the geometrical/ geological model has to be carefully considered in particular when extrapolating rock mass information to unknown volumes of rock.



(Data from borehole KAS02, KA2511A and KA2598A.)

Figure 6-2. *Competent rock: Block H and I.*



(For Block E data from KA2598A; for Block J data from KAS02.)

Figure 6-3. *Deformation zones: Block E and J.*

An attempt of determining the pattern in the variation of RQD, Q and RMR inside rather homogeneous blocks like Block H and I was done by means of variograms. The variogram for the three parameters was determined. The variation of RQD, Q and RMR inside the blocks seems to be too small to be identified by this method (Figure 6-4 for Block H).

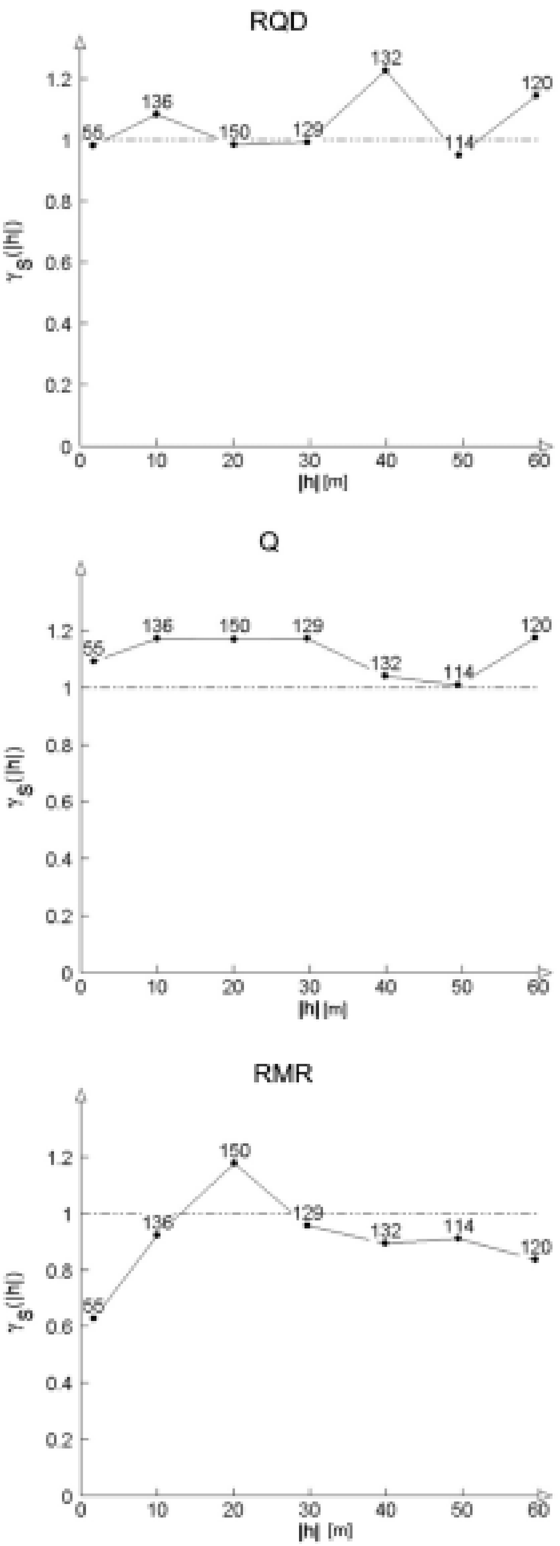


Figure 6-4. Standard semi-variogram of RQD, Q and RMR along borehole KAS02 for Block H.

6.1.4 Partitioning of the borehole according to RQD

During analysis of the logging of the boreholes for the characterisation, sections with approximately the same RQD were isolated as homogeneous domains for parameterisation. Although most of the rock exhibited a RQD higher than 90 (”very good" rock according to the RQD classification), the result of the characterisation indicated that the classes of rock at the Äspö Test Case were more numerous due to the influence of other important parameters (e.g. aperture, spacing, persistence, joint properties and weathering). This indicates that partitioning the borehole according to RQD for rating parameterisation does not prejudice the result of the characterisation by overlooking the other geological parameters. In quite homogeneous crystalline rock masses like in Äspö, this partitioning method can then be used as general technique for characterizing the rock along boreholes.

In general, RQD alone might not be the only parameter for identifying sub-domains for parameterisation inside the rock units. RQD is sometimes not sensitive enough to univocally identify the variations of quality in the rock mass. Some other parameters, especially fracture frequency, fracture set number and fracture size, are equally significant, and their use in parameterisation domain division should be looked into in the future. However, those parameters were already considered in the geological model when subdividing the rock mass into homogeneous rock units.

Another important aspect is the size of the rock domains to be characterised. As it is shown by the comparison between NGI and BBK results, small domains (e.g. short length of core) produce more scattered results than large domains. This is due to the fact that a large size of the domain tends to average the properties of the rock mass so that heterogeneities are overlooked. Thus, it appears important to decide the scale of the volume of rock to be characterised. Depending on the kind of analysis tools for which the rock mass parameters are derived, a suitable domain size should be selected. In fact, on the basis of experimental results, it seems reasonable that for a volume of rock large enough, some relevant properties should become scale-free. This volume is usually referred to as Representative Elementary Volume (REV) /Cunha, 1990/.

6.2 Mechanical parameters as outcome of the characterisation process

Some general consideration can be made about the rock mass mechanical properties derived from the available empirical relations.

1. The choice of a relevant set of mechanical parameters to be used for characterising the rock mass should be done very carefully. This choice has to be done on the basis of widely accepted *definitions* of the parameters and it should focus on parameters useful for design, safety assessment and prediction of the mechanical behaviour of the rock masses. This implies that the choice of these parameters also depends on the kinds of analysis to be used in the calculations (e.g. qualitative or quantitative analyses; analytical or empirical versus numerical methods; continuous versus discontinuous modelling, etc).

2. The properties obtained from the characterisation should be interpreted as initial values without *stress and water pressure dependence*. However, there are certain techniques available to estimate more realistic stress and water pressure dependence using analytical, numerical and experimental methods (cf. Figure 3-5).

6.3 Issues of special importance and difficulty

Some questions still remain unsolved due to the complexity and size of the geological object, due to the lack of scientific tools to handle a particular problem or due to economic reasons. The most important questions are listed below:

1. *What to do when the provided geological/geometrical model of the site, which may be constructed based on a simplified overall geological approach, does not agree with some of the detailed geological/geometrical information available, e.g. in boreholes?*

   It can happen that borehole information inside a volume of rock described as deformation zone does not show information for fracture frequency, infilling and degradation typical of a deformation zone. This can be caused by low confidence in the geological/geometrical model in some particular locations. The level of confidence depends on the amount of geological data available, but also on the degree of simplification applied for constructing the geological/geometrical model. The user of the geological/geometrical model of the site should therefore be provided with the indication about the reliability and confidence of the model at each location. This is particularly true when the geological/geometrical model inferred on the surface is extrapolated several hundred meters deep in the rock mass.

2. *How to evaluate scale effects in rating systems, e.g. effects of using different division of core-section lengths for parameterisation?*

   The rock mass characterisation is sensitive to the technique adopted for isolating homogeneous sections of borehole on which the characterisation is performed. In fact, depending on the geological parameter used for identifying homogeneous sections, the length of the sections can vary markedly. Moreover, the choice of the ranges of variation of each parameter that identifies homogeneous sections has great effects on the length. In principle, long borehole sections tend to smoothen out the local variations, so that the characterisation gives averaging results. On the other hand, some parameters are scale dependent (e.g. fracture density) so that the results have different statistics when changing the length of the analysed sections. Sensitivity studies on the issue by varying core section lengths for parameterisation are needed for evaluating its impact. For example, the minimum core length for determining the rock mass mechanical properties needed for the design of a tunnel should be investigated.

3. *How to evaluate and quantify spatial variations with limited data population compared to site volume?*

   Information about the rock mass is often obtained along linear or planar features, rather than in volumes (e.g. boreholes and surface mapping). Thus, information has to be extrapolated outside boreholes or in depth from the surface. This imply that the patterns of variation of the geological and mechanical features of the rock mass have to be investigated more completely in order to forecast properties outside the known areas. This can be done in a statistical fashion, by extrapolating the statistics of the parameters (classification ratings, mechanical properties) to the recognised homogeneous geological units. This methodology, however, cannot be applied to pinpoint properties at specific points. Another possibility is applying geostatistical methods. By means of these methods, the spatial pattern of variation is approximated in mathematical terms so that an evaluation of a parameter in a particular position in space is possible and is related to the values of that parameter in its neighbours.

4. *How to validate the empirical approach and its outcome?*

   The empirical methods were developed based on experiences on rock engineering case histories. This makes the results of the classification much dependent on the excavation techniques, support solutions, different local and national geological and working environments, safety measures, excavation geometries, success/failure ratios, etc. On the other hand, many of the successful cases of numerical modelling for design, stability and safety evaluation are based on back analysis of the rock mass behaviour so that the model parameters are adjusted to fit in-situ measurements. In turn, the values of each property empirically obtained from the rock mass characterisation do not necessarily coincide with those inputted in the numerical models, which depend on the constitutive laws and numerical method adopted for the calculation.

5. *Can geophysical data be used for rock mass classification more extensively?*

   It should be investigated in a systematic way if geophysical data could be used more extensively for correlation with rock mass characterisation. In this project only the correlation between the Q-system and seismic velocity was used, but this correlation must be investigated more in detail for two purposes: the first is for validating empirical correlations and the second is for investigating site specific effects. (There also exists correlation between RMR and the seismic velocities, but it was not explored in this project).

   It would be of great advantage to use geophysical results for this purpose in order to increase the confidence of the characterisation inside large volumes of rock and also between boreholes. It seems that the correlation with quality of the rock mass is site specific, which implies that a study should be carried out for each candidate site for enhancing the reliability of the geophysical relations.

6. *Do we have knowledge of the depth dependency of geological parameters?*

   A number of parameters are used in the classification systems to describe the rock mass. However, there is very little information/knowledge about the stress dependency or their variation with depth. As the repository will be placed quite deep, the knowledge of the stress dependency is very important. For improving the understanding of the stress and depth dependence of parameters, it is essential that the variation of the input data for parameterisation of the rating systems with depth should be investigated.

7. *How can the characterisation be improved with combined empirical approaches and numerical homogenisation/up-scaling models in 3-D?*

   The empirical approaches always imply a certain degree of homogenisation of the properties of the rock mass because this is treated as an equivalent continuum. On the other hand, rock block size and fracture parameters might change with the size of the analysed rock volume and with orientation, so that the process of homogenisation is implicitly scale-dependent. Under the same boundary conditions, two volumes of rock of different size in the same rock mass can exhibit different equivalent mechanical and hydraulic properties. This discrepancy usually remains till a certain volume size is reached, and for volumes larger than this the equivalent properties do not vary. This volume is called the Representative Elementary Volume (REV). The attention focuses therefore on two aspects: a) the characterisation can give different results depending on the size of the homogeneous volume of rock on which it is performed; b) the characterisation can give different results depending on the direction along which the property is investigated /Long et al, 1982/.

Analytical and numerical modelling, preferably in 3-D, can greatly improve the methods for characterisation of the rock mass, making it possible to derive the scale-dependency of the rock mass properties and to establish at what scale the hydro-mechanical properties become constant. The issue of the scale dependency is relevant when evaluating tunnel-size and site conditions. For this purpose, several case histories at different scales should be characterised with empirical methods and analytical models.

8. *Can we determine the hydro-mechanical properties of the rock mass from the results of the characterisation?*

   As discussed before, characterisation should be kept separated from boundary conditions. Thus the coupled hydro-mechanical response of the rock mass should not be of concern at this stage. However, the modern modelling techniques could help improving the characterisation methods by enabling them to derive correlations for predicting the hydro-mechanical properties of the rock mass. Depending on the sophistication level of those techniques, it could become possible to describe the coupled hydro-mechanical behaviour of the rock mass with the empirical methods. Considering the hydraulic conditions, /La Pointe et al, 1996/ have proved that the continuum approaches have several shortcomings in modelling the fluid flow through the rock mass. Network theory, statistical methods and numerical simulations of discontinuous media seem to be more suitable for modelling the flow. /Liu et al, 1999/ have proposed that the effective hydraulic properties of fractured rock masses can be derived from RQD and RMR. Since mechanical and hydraulic properties always affect each other, the empirical approaches for characterising the rock mass properties with water effects should be more carefully investigated.

9. *How do we improve the quantification of variability and uncertainty of input data and derived rock mass properties?*

   Spatial variability of the input data and properties derived from the rock mass characterisation can be quantified based on scientific evaluation tools. Besides the traditional statistical methods, geostatistical tools such as variograms and kriging are available and both describing and predicting the occurrence of certain parameter values at certain locations.

   On the other hand, uncertainties due to the data gathering technique are more difficult to estimate. In fact, these are derived from the theoretical assumptions or empirical relations assumed for obtaining the geological/rock mechanical data based on the experimental observations.

   Additional uncertainties come from the techniques for processing the experimental data, and the nature of the classification systems. Since the classification systems are based on empirical relations, the quantification of the uncertainty in the determination of the rating values and rock mass properties can only be improved by: i) validating the parameters obtained against experimental results and case histories at site-specific conditions; ii) comparing results obtained from different independent classification systems; iii) building a theoretical model that carefully describes the rock mass behaviour and allows for a sensitivity study of all geological/geometrical/mechanical parameters involved in the characterisation and obtained from it.

7 Conclusions

7.1 Rock mass characterisation in a site investigation process

The aim of the rock mass characterisation is the determination of the quality of the rock mass and of its mechanical properties. These properties are to be used in further studies about stability, structure design and alternative solution studies. Thus, at some stage, the parameters obtained from the characterisation should be directly useful for the design of the excavation. When stability analyses are concerned, stress and water conditions have to be regarded so that the rock volume can be considered in its geometrical and geological context. The effect of stress and water pressure could then be considered as external boundary conditions. An additional set of parameters could then be provided for describing the influence of stress and water pressure on the strength and deformability of the rock mass. This approach suits very well the rock unit system described in Sec. 2.2. In fact, for characterising a rock unit, that usually spans several hundred meters in depth and width, it would only be necessary giving the average properties and the variation of the average with depth, instead of specifying them point-by-point or stepwise. This is also compatible with the format of the RVS input data /Markström, 2001/ that sometimes requires gradient information.

By keeping rock mass properties separated from environmental and boundary conditions, no safety factors for failure mechanism (rock burst or slabbing, etc), tunnel shape or depth would affect the determination of the rock quality and mechanical properties. This is not the case for the traditional classification systems. It would be only when inferring the stability of the excavation that these factors should be considered. Apparently, this approach is a modification of the traditional classification systems for rock mass characterisation: Q should be preferably calculated as Q', where SRF and Jw are set to one /Hoek et al, 1995/, and RMR calculated using zero for the orientation rating and 15 for the water rating /Hoek and Brown, 1997/. The philosophy of keeping rock mass quality and boundary conditions separated for the purpose of rock mass characterisation is illustrated in the flowchart in Figure 7-1, together with the following phases of rock mass classification and structure design. The outcome of the characterisation (e.g. rock mass quality and derived mechanical properties) is usually used as input parameter for designing of the structures either by using classification systems or numerical modelling.

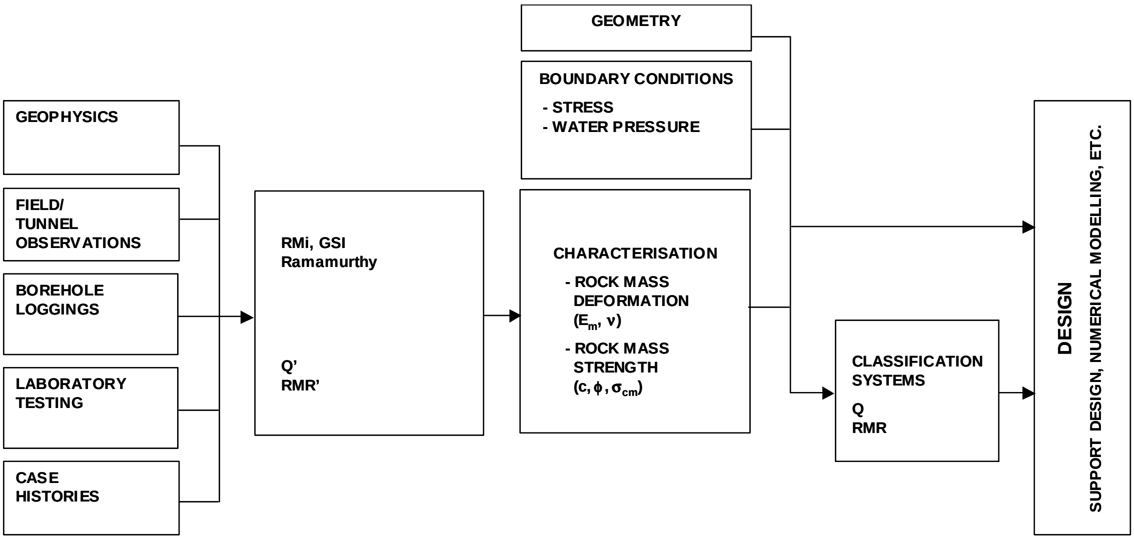


Figure 7-1. *Suggested flowchart for rock mass characterisation, classification and design.*

7.2 Important aspects for rock mass characterisation

Based on the experiences gathered in this project, some important aspects of the rock mass characterisation are listed here. Some of them are general and some might be site specific and therefore might not be valid outside Äspö.

The geological information available for characterisation is quantitatively different at the different stages of the site investigation. At the pre-investigation stage, the rock classification systems are perhaps the only tool for an initial estimation of rock mass properties using surface mapping and geophysical data. On the other hand, the lack of input data at this stage can cause the characterisation to be uncertain. The degree of uncertainty should diminish as soon as not-redundant geological information is collected.

Among the classification systems reviewed in this Report, Q and RMR were given more attention because: i) they are based on very broad databases of case histories; ii) they are worldwide largely used; and because iii) they are provided with a variety of empirical relations with rock mass strength and deformation parameters. However, other characterisation and classification methods, of which some have been reviewed in this report, are also useful for determining the quality of the rock mass, and can provide the extremes of variation of the rock mass properties.

Different scenarios can be identified for rock mass characterisation depending on the quantity and quality of the available geological data and the rock mass quality:

1. *Scarce geological data:*

   This is particularly the case of site preliminary feasibility study. Often only a qualitative description of the site is available (e.g. rock types but no details on formation boundaries at depth, major surface structural features, pictures, geological maps, etc). In this case, very experienced judgement for assigning RMR, Q and GSI might be used for rock mass characterisation so that even in a very early stage of the investigations, some indications of the deformation modulus and strength of the rock mass can be estimated through a proper sensitivity analysis of the parameters in the empirical relations. However the data are uncertain, may be sparsely distributed and only available at the surface.

Similarly, when seismic profiles or sonic logging in boreholes are measured at the site, the rock mass can be preliminarily characterised with the available relations between the seismic velocities and the Q-rating. Indications about the rock mass deformability (dynamic deformation modulus and Poisson's ratio) can also be determined.

2. *Detailed geological data:*

   It refers here to geological data obtained from borehole logging, tunnel and surface mapping and in situ and laboratory testing on rock material and fractures. In this case, the Q and RMR-systems are suggested because the procedure associated with them is quite standardized.

3. *Competent rock masses*:

   In this case, as illustrated by the Äspö Test Case /Hudson, 2002/, RMR seems to be more robust to give an objective evaluation of the rock mass quality. Also the RMR-results independently obtained by BBK and NGI in Sec. 5.10.1 were almost exactly coincident for the same boreholes. This system takes into account most of the information directly provided by the geological surveying and borehole core logging. It also explicitly considers the strength of the intact rock. Relations are available for deducing the deformation modulus and the strength of the rock mass. However, when applied to the Äspö Test Case, this method appears to have a higher sensitivity to the quality variations in the rock mass toward the better rock mass classes (very good and good rock) than Q-system (cf. Figure 6-2).

4. *Rock mass of poor quality*:

   In contrast to what observed for volumes of competent rock at the Äspö Test Case, the Q-system seems to be more adequate for describing deformation zones and fractured zones in hard rock. In these conditions, Q-values produce a larger variation of rock classes than the RMR-system. This makes the description of the rock mass more detailed and sensitive to fracture zones (cf. Figure 6-3)

These considerations suggest that at least two classification systems should be applied for each site in a completely independent way /Bieniawski, 1988/. In /Palmström et al, 2001/, it is also recommended that different rating systems should not be mixed. As illustrated before, different systems are more or less suitable to certain rock mass conditions. However, each classical classification systems could be developed into some simplified site-related classification/characterisation system particularly suited to a certain environment (e.g Swedish crystalline rocks).

The rating systems occur to be site specific. This will have consequences when several sites are to be compared. Thus, it is crucial that comparable results are obtained at the end of the process. For guaranteeing that this comparison is possible, the characterisation must be a robust process where different operators will obtain the same result for the same site. For this reason, there is an exigency of unifying the definitions and codes for representing the geological parameters so that no ambiguities appear when comparing the parameters obtained with different techniques, by different operators and at different times and places. It would be very useful to accompany the geological data by sheets with the definitions and the methods used for collecting the data.

For the exigency of comparison and for the use of those parameters for further design calculations, there must be consistency in the definitions and agreement on the choice of the significant mechanical parameters. It is here suggested to refer to the definitions of the rock mass strength parameters and deformation modulus in Sec. 2.5.1.

The construction of the structural geological/geometrical model should consider the needs of rock mechanics characterisation. Some aspects of marginal interest from the geological point of view are sometimes of major relevance for the rock mechanics characterization, on which the design of the underground construction is based. Thus, one should be careful with the different concept of homogeneity in geology and in rock mechanics. An example is that changing fracture density from 2 sets to 3 sets may not affect the geological homogeneity of a rock unit, but will have significant effects on rock mechanics properties, therefore on the rock mechanics definition of homogeneous units. Moreover, some indications of the confidence in the spatial position of the boundaries between the rock units should also be provided for rock mechanics characterization. This has consequences in the determination of the properties for each rock unit.

The characterisation of the weakness zones should be better defined and improved compared to the technique used for the Äspö Test Case. For large weakness zones defined in the geological model, a more detailed characterisation technique is needed. For small weakness zones within a homogeneous rock unit, special attention should be taken during the characterisation process.

In consequence of the difference between homogeneous volumes of rock from a geological and a rock mechanics point of view, it is necessary to obtain the fracture sets information for each rock unit, without over-simplifying, extrapolating and averaging over the whole site volume. Rock fractures exhibit patterns and fracture set number varying with depth and when approaching deformation zones. These features cannot be ignored when characterizing in detail the rock mass for design purposes.

Furthermore, a parametric analysis should be performed to study the effect of the characterisation-domain size on the ratings and derived properties. In fact according to the Representative Element Volume concept, also the rock quality ratings would be scale-dependent. The size of the reference rock volume on which characterisation is performed is sometimes defined for different reasons: e.g. characterisation cell size (as for the Äspö Test Case), modelling mesh cell size, etc. It is then important to evaluate the sensitivity of such volume size on the characterisation result for a certain site.

As pointed out before, the empirical relations relating the mechanical properties with the classification systems are site-dependent. It is thus recommended to validate the outcomes of the empirical methods against in situ measurements. This is possible for instance by determining the in situ deformation modulus of the rock mass by for example Goodman-jack testing, hydraulic jacking or pressiometer, even at an earlier stage of the investigations. It is sometimes rather difficult to avoid the influence of the excavation on the determined deformation modulus, if testing is carried out close to pre-existent excavations. The deformation modulus of the rock mass can also be back-calculated from in situ measurements of displacements around the excavation, using numerical simulations with cautions for influences of material models, geometry and boundary conditions.

For very early estimations, parameter values for rock mass strength and deformability might be taken from similar projects in the same geographical area only if they have the same similar geological environments.

Dynamic properties of rock masses may also need to be considered in rock mass characterization. For example, the wireline seismic methods measure the seismic velocity (Vertical Seismic Profiling, VSP) in the rock immediately close to the borehole. From this kind of measurements, the dynamic in-situ deformation modulus of the rock mass can be determined along the borehole axis, when both the shear and primary wave velocity are obtained.

The validation of the rock mass strength against in-situ tests is a more difficult task, and it is not feasible at the site-investigation stage. Usually, the rock mass strength cannot be calculated directly from the deformation of the rock mass. It is thus quite complicated, and not very often done in tunnelling engineering, to set up a destructive test at the site. Some hints about the rock mass strength can be obtained from the parts of an excavation where the particularly difficult conditions have caused stability problems. However, this is often the case of sections of poorer rock masses, and the strength of good rock masses would seldom be assessed in this way. The influence of the size of the investigated volume must be considered due to the scale effects on the rock mass strength.

Spatial variability can occur as variation of the geological parameters around their mean values and/or as variation of the mean parameters according to existent trends. For example, fracture frequency can either randomly vary in space by keeping the same mean value, or increase by approaching a deformation zone. If the parameters randomly vary but present a spatial correlation, eventually their statistics will vary by changing the volume of rock mass on which they are calculated, thus they are said to be scale dependent. Because of this spatial variability, also the characterisation results can vary along a chosen direction of for increasing volumes of rock. Thus, spatial variability of the geological/fracture parameters should be checked and if possible highlighted.

Uncertainties are often due to lack of information. Missing geological/geometrical input data imply uncertainty in the determination of the characterisation rating parameters. Due to the way the empirical systems are structured, uncertainties do affect the final ratings (e.g Q or RMR) with different extent. In consequence: i) not all the ratings are equally sensitive to uncertainty; ii) great uncertainty does not necessarily results in large variation of the expected characterisation results and vice-versa.

7.3 Requirements for gathering input data for rock mechanics characterisation

Since the geological input data are the base of the characterisation systems, the attention naturally focuses on them. In general, a lack of information affects the results and quality of the site characterisation. When practically and economically possible, more input data are desirable even thought that some data can never be assessed during site investigation. Based on the experience of rock mass characterisation at Äspö, some important recommendations, requirements and improvements to gathering of the input data for rock mechanics characterisation are listed here:

1. The characterisation and classification must be performed systematically with all available information at the site by skilled engineering geologists or rock mechanics professionals. Only in this case, the result of the rock mass characterisation/classification can be considered robust enough.

2. The kinds and definitions of the observed/measured parameters during site investigation should reflect the needs of the rock mechanics characterization and classification systems.

3. The data should be collected according to international standards and very well defined rules used at the site, if site-specific. This is particularly true for the available records of roughness, aperture and fracture surface conditions.

4. For the characterisation, at least two independent systems should be used and preferable Q and RMR.

5. More attention should be paid to the partitioning of the rock mass into rock units and to the homogenisation techniques for giving properties to the units. In particular, some refinements of the geological/rock mechanics model should be considered after the rock mass characterisation has been performed. In this way, more detailed domains with homogeneous rock properties could be isolated to be useful in the design process. Two situations can be considered:

    - The subdivision of the rock mass into homogeneous domains/rock mechanical units for characterisation should, besides RQD, also use other geological parameters, as fracture frequency, fracture set number and fracture size.

    - A subdivision of the rock units based on a statistical methods would probably not be representative especially for smaller weakness zones that in that way could be overlooked.

6. Improved geological surface investigation and borehole logging considering rock mechanics characterization needs: i) definite lithology boundaries; ii) identification of the fracture set/density distribution; iii) true fracture orientation; iv) quantitative indication of the persistence of the fracture sets; v) rock mechanics description of fracture surface conditions especially quantitative description of roughness and aperture; vi) precise indication of the fracture weathering; vii) water condition of the fractures (dry, wet, etc). Particular attention should be reserved to fracture orientation. This parameter is important not only for fracture set determination, but also for block stability analyses that are based on the true orientation of the rock fractures and the geometry of the excavations.

7. Standard rating forms should be designed for the characterisation according to each empirical system. From the compilation of these forms it would be immediately clear what data are missing guiding further investigations for completing the input of the characterisation. Qualitative descriptions should be provided for supporting the choice of parameters and ratings for the characterisation, even if this is the result of a guesswork.

8. A systematic and improved mechanical testing of intact rock/fracture samples with recorded sample locations and testing techniques is recommended. The number of test samples should be adequate for statistical analysis of the results and from a number of different locations, so that also the spatial variability of the rock mass can be explored. For example, the uniaxial compressive strength of the intact rock and the shear strength of the fractures can be determined by means of:

    - Point load tests. are particularly recommended as low-budget completion of the testing performed on intact from rock core samples /ISRM, 1985/.
    - Schmidt hammer tests: are warmly recommended on fracture surfaces and fresh rock surfaces for determining the weathering degree of the fracture with respect to the rest of the rock mass /Deere et al, 1966/.
    - Tilt tests /Barton et al, 1985/ and shear tests /ISRM, 1981/ on core samples: can be used for characterising the rock fractures at different locations.

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