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/).

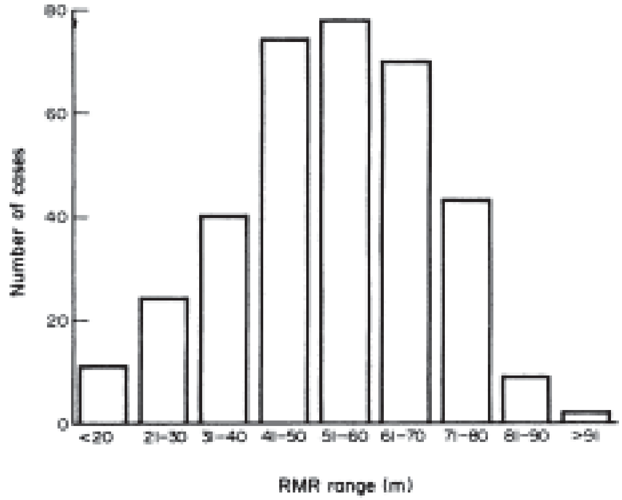

Figure 1-1. *Frequency distribution of the values of RMR in the case histories reported by /Bieniawski, 1989/.*

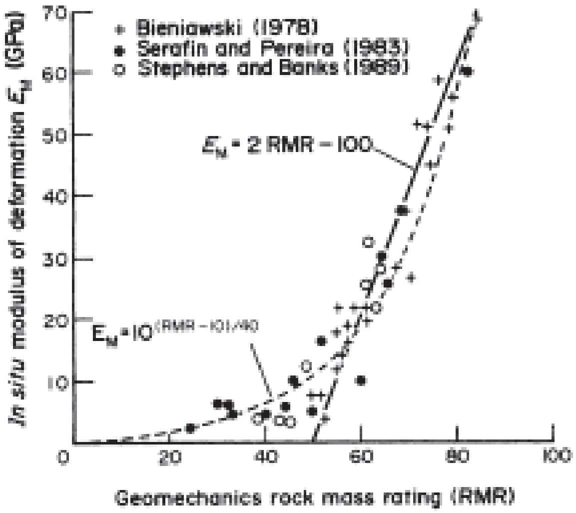

Figure 1-2. *Correlation between the in-situ deformation modulus of the rock mass and the Rock Mass Rating /Bieniawski, 1993/.*

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.

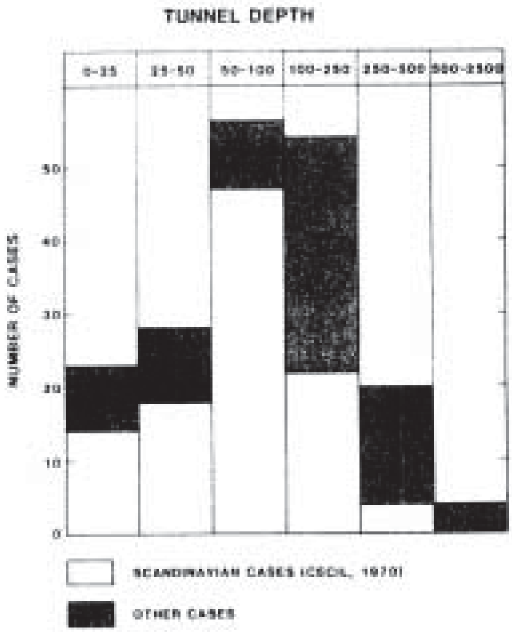

Figure 1-3. *Frequency distribution of the tunnel depth for the 212 cases in the Q-system database /Barton, 1988/.*

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;

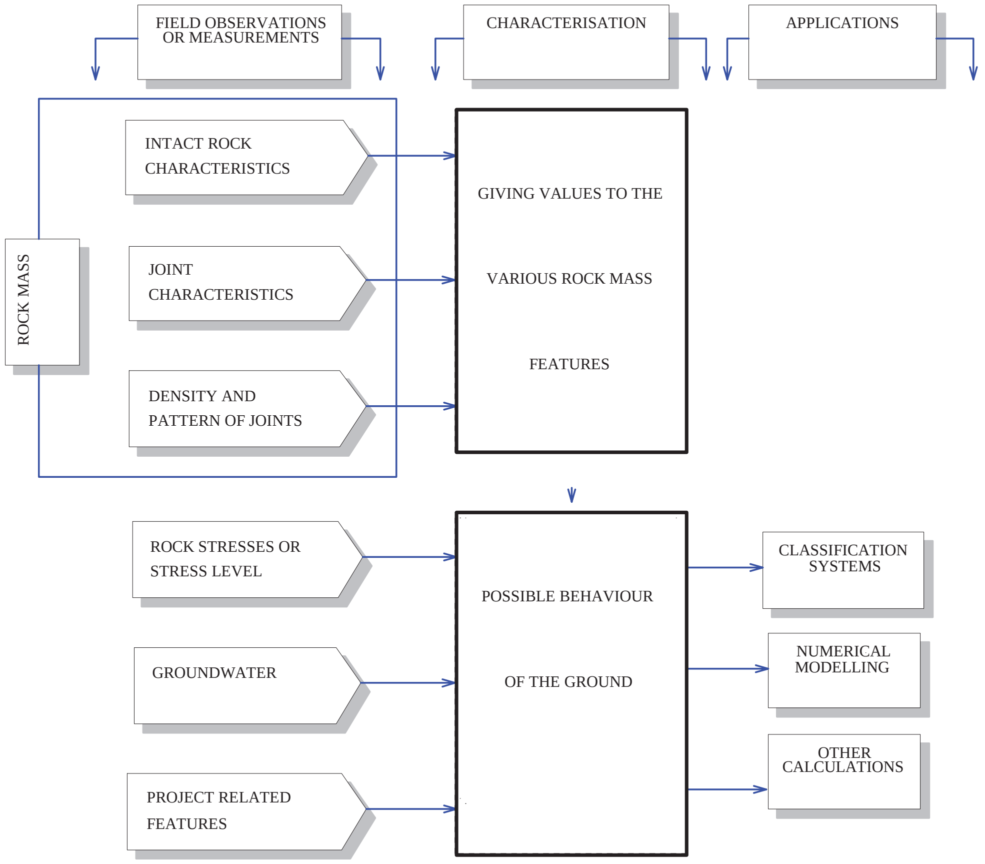

Figure 1-4. *Flow chart for rock mass characterisation and design /Palmström et al, 2001/.*

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.

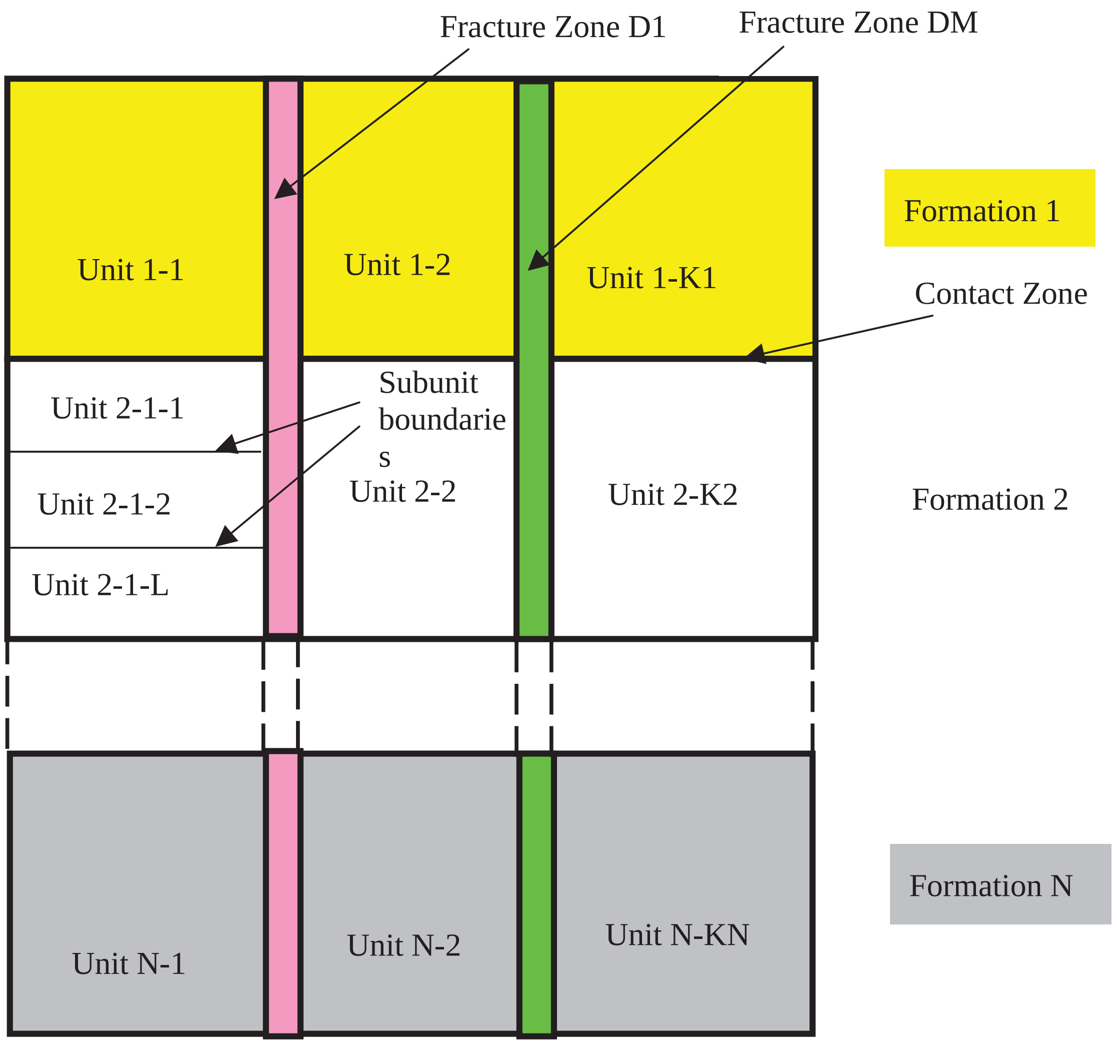

Figure 2-1. *The conceptual geological model of the site by Rock Unit System by dividing the rock mass into Rock Units (U1-1…UN-KN), Lithological Rock Formations (1…N), major Fracture Zones (D1…DM) each of them with rather homogeneous fracture properties (fracture set number, RQD, roughness, aperture,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

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (1)$$

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/:

$$Q' = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \qquad (2)$$

/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:

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

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;

$RMR_{\text{fracture infilling}}$ (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;

$RMR_{\text{fracture orientation}}$ (–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:

$$RCR = 8 \ln N + 30 \tag{10}$$

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/.

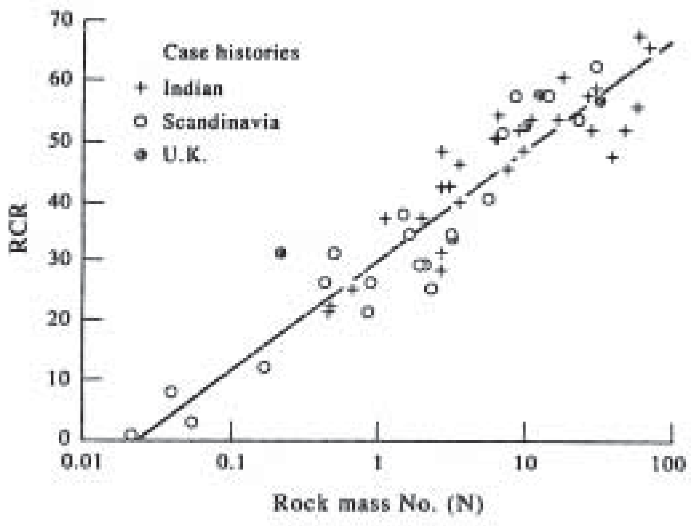

Figure 2-2. *Correlation between the reduced RMR (RCR) and Q (N) /Goel et al, 1995/.*

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:

$$GSI = RMR - 5 \text{ for } RMR > 23 \tag{11}$$

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.

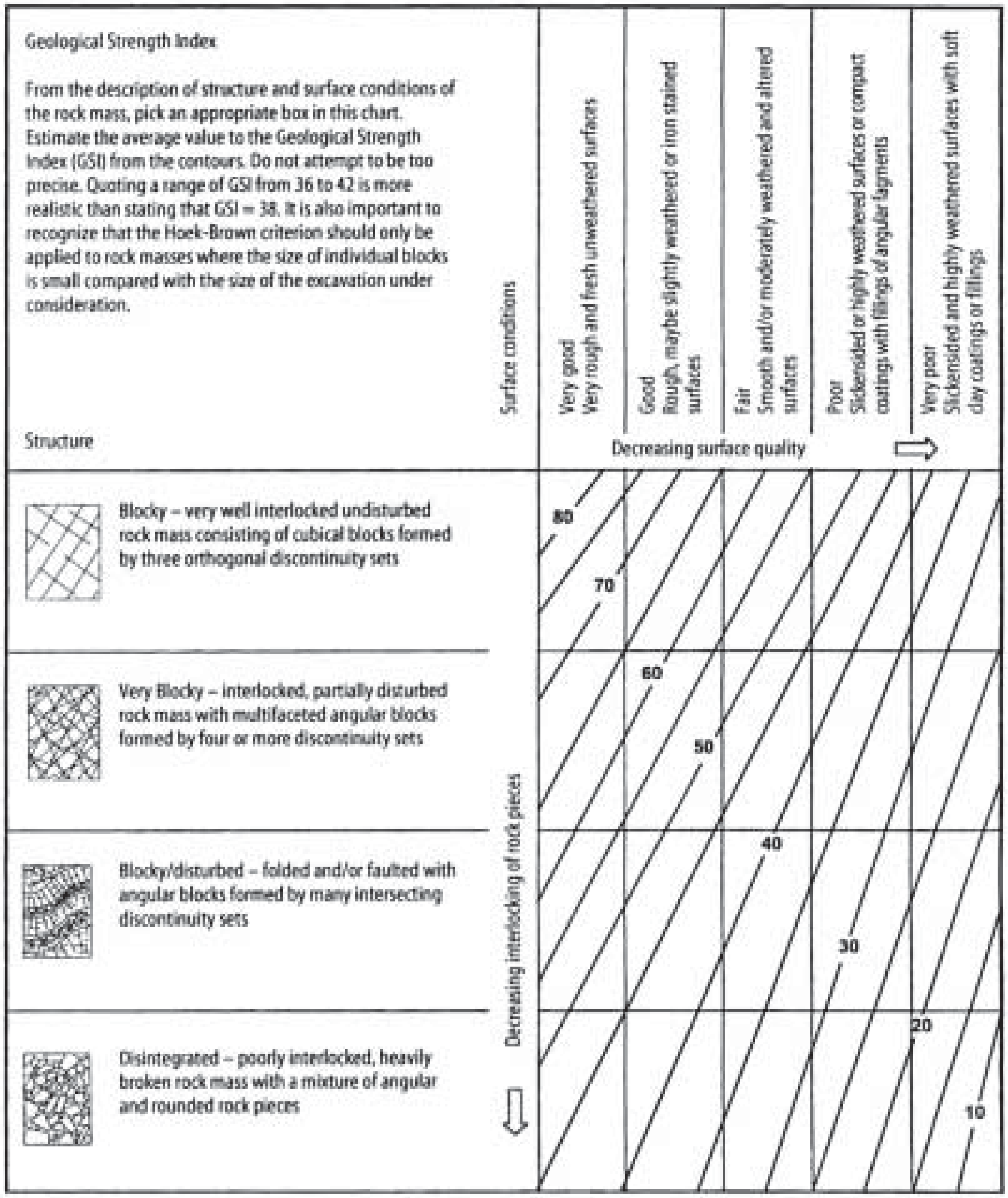

Figure 2-3. *Geological Strength Index (GSI): description of the rock mass quality based upon the interlocking of the rock block and the conditions of the fractures.*

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:

$$N = 3[V_0/V_p]^{V_0/2} \qquad (25)$$

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:

$$Q = 10^{\frac{V_p - 3500}{1000}} \qquad (26)$$

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

$$Q = (V_p - 3600)/50 \qquad (27)$$

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:

$$Q_c = Q \times \frac{\sigma_c}{100} \qquad (28)$$

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.

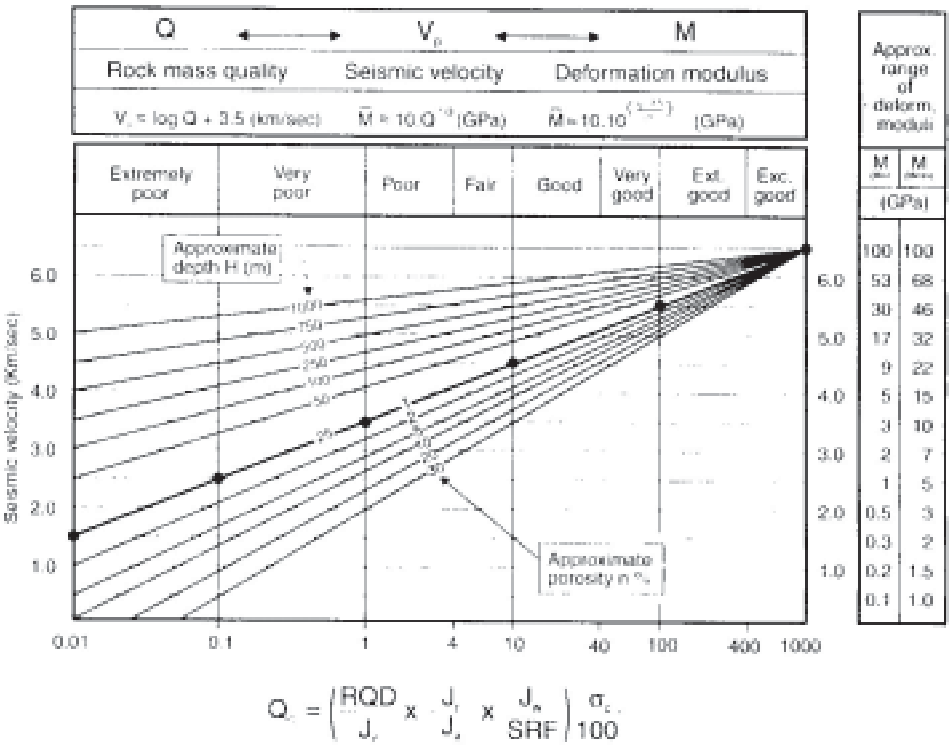

Figure 2-4. Correlation between the classical and modified Rock Mass Quality Q and Q_c, respectively, and seismic velocity and deformation modulus for design purposes /Barton, 1995/.

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:

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

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:

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

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:

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

and:

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

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

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

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:

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

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:

$$E_m = 5.6\ RMi^{0.375} \tag{51}$$