AFTES

GUIDELINES FOR

CARACTERISATION OF ROCK MASSES USEFUL FOR THE DESIGN AND THE CONSTRUCTION OF UNDERGROUND STRUCTURES

A.F.T.E.S. welcomes comments on this paper

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PREFACE

In 1978, AFTES issued its first Recommendations on the description of rock masses, using the following approach:

- Describe in detail all factors potentially influencing the stability of underground structures
- Classify field conditions with respect to each individual factor separately, without attempting to link them together.

This new version retains this basic principle, but with important additions.

Firstly, we make a clear distinction between the characterisation of (i) the rock matrix, (ii) discontinuities, and (iii) the rock mass taken in its entirety, dealt with in the three main chapters forming the backbone of the Recommendations.

We have placed the description of the influencing factors within the general context of the geotechnical survey. This description is dependent on a geological model. This model is made up of 'homogeneous sub-units' displaying the relevant characteristics. We were also concerned about the transition from instrumental and field data to data values used in the design analyses.

Lastly, we took the decision – and risk – of presenting general rock classification systems, not in order to advocate their systematic use but to point out their limitations. Despite their apparent convenience, these classification systems (and more importantly, the correlations drawn from them) simplify to an outrageous degree a reality which is always complex. They can never be a substitute for abundant observation, measurement and testing, and one must always bear in mind the value of the parameters on which they are based throughout the whole design and construction process.

The descriptive approach recommended here by AFTES applies not only to structural stability analysis but equally to the selection of location, cross section and construction method. It is not confined to tunnels only, and these Recommendations may be thought useful for other types of rock engineering.

Jean Piraud
Chairman, AFTES Technical Committee

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1 - INTRODUCTION

1.1. – PURPOSE OF ROCK MASS CHARACTERISATION

The most important goal in the characterisation of rock masses is to provide the engineer with qualitative and quantitative data to describe their structure and assess their mechanical and hydraulic properties at a scale commensurate with the volume of rock affected by the structures. The overlying materials (sand, scree, moraine, etc.) are ignored in these Recommendations.

It is essential to have precise knowledge of this data for project design, selection of construction methods, support and lining thickness. The cost of the works is directly dependent on these points.

Whereas the mechanical properties of the rock matrix can be determined from laboratory tests on small specimens, those of a rock mass measuring several thousand cubic metres in size which may contain within itself many discontinuities and heterogeneities cannot be determined directly.

In situ field tests, whose number is inevitably limited by their high cost, come somewhere between laboratory tests and the full size structure in terms of scale. They are instructive but still imperfect for fully ascertaining the mechanical properties of the rock mass at the relevant scale.

Not being amenable to direct measurement, the mechanical and hydraulic properties of the rock mass must necessarily be approached by indirect methods:

• either by trying to construct a model of the rock mass relevant to the size of the structure under consideration, using test data obtained at a smaller scale and the characteristics of the discontinuities,

• or by resorting to current rock classification systems and the mechanical characteristics which can be derived empirically from them, based on the back-analysis of full scale structures, as operated by various authors.

In both cases, it is vital to arrive at the most methodical and comprehensive characterisation of the rock mass as possible.

1.2 – GEOLOGICAL MODEL

1.2.1 – Geological survey

Before embarking on the stage of a rock mass characterisation properly so-called, as defined in these Recommendations, the design process normally begins with a geological survey to situate the project area within the general geological setting.

The survey rests essentially on field work by engineering geologists using the full armoury of tools and methods available to the them:

• Desk study to collect published material, maps and data

• General mapping of the project area, detailed mapping of outcrops and indicators, collection of hydrogeological data

• Photography at various scales (satellite imagery and aerial photographs)

• Geophysical methods: high resolution seismic reflection and refraction, resistivity and electromagnetic tests, thermographics, ground penetrating radar, etc.

• Exploratory boreholes, shafts and adits

• Information available from any nearby structures, etc.

The geological survey locates the major geological units and their relations, the main discontinuities, the tectonic history, etc.

1.2.2 – Geological model

The first step in the characterisation of rock masses requires constructing a tentative geological model showing the geological structure of the rock mass complete with its constituent units, boundaries, major features, heterogeneities and uncertainties.

It will ideally be a three-dimensional conceptual model yielding cross sections to be used for understanding the structures and identifying singularities and indeterminate points.

The geological model is the indispensable basis for proceeding with the characterisation of the rock behaviour parameters. With this model, the rock can be subdivided into homogeneous sub-units whose mechanical and hydraulic properties can subsequently be determined at project scale.

What governs the size of a sub-unit within the rock mass is its uniformity of its geotechnical properties, producing uniformity of response to the structure to be built. A sub-unit may thus occupy part of a geological stage, the whole stage or several stage. It may be homogeneous in terms of its lithology, jointing, rock stresses, etc.

Of course, even small features that are only local singularities in the wider rock mass must be treated as individual sub-units and as such, be the subject of geotechnical characterisation.

Subsequently, as the design studies proceed, the latest results from the geological survey will be incorporated from time to time into the model.

1.3 – GEOTECHNICAL CHARACTERISATION OF SUB-UNITS

Characterisation of a homogeneous sub-unit in a rock[2] mass always involves determining the parameters of the rock matrix and of the discontinuities; with discontinuities, the geological survey must make it possible to choose the most relevant scale at which they must be analysed and characterised with reference to the scale of the project under design.

Some homogeneous sub-units as defined in para. 1.2.2. may consist of more or less regularly alternating rock layers, each with highly contrasting geotechnical properties (for example, marl limestone, flysch, etc.) which must be analysed separately before proceeding with the characterisation of the whole sub-unit.

[1] It must be noted that some homogeneous sub-units thus defined are liable to include random heterogeneities (karst cavities for example) unidentifiable by the exploratory works. It is the engineer's job to decide what steps are to be taken in respect of this risk, after assessing its probability.

[2] Rock must be understood in a very general sense. It may be a mass of soft rock or rock crushed by tectonic action to the point where it becomes like a soil, as well as a highly competent rock mass.

Characteristic parameters for the rock matrix and discontinuities appear at the top of Table 1. Those for the rock mass, some of which derive from the former, are listed in the bottom half of the Table.

2 – MATRIX CHARACTERISTICS

PRELIMINARY REMARK. Most laboratory tests used to characterise rock matrix parameters are inexpensive compared to field exploratory works (drilling) and even more so, the cost of building the structure. It is always advisable to perform enough testing in order to obtain data that can be manipulated by statistical methods and

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Table 1 – Characteristic parameters of rock matrix, discontinuities and rock mass
(Numbers refer to paragraphs in these Guidelines)

reveal the homogeneity or scatter in the measurs. Meaningful laboratory test results are essential to draw full benefit from the information obtained by drilling.

2.1 – IDENTIFICATION PARAMETERS

2.1.1 – Common names

Rock names are based on chemical and mineralogical composition, texture and the way their were formed. There are three main classes of rock: igneous, metamorphic and sedimentary rock (Appendix 1).

Igneous rock is solidified magma. Solidification at depth produces plutonic rock which solidified slowly and permitted crystals to grow large enough to be seen with the naked eye; the most common example is **granite**. Extrusive rock is formed from magmas emerging directly at the Earth's surface; few crystals can be seen by eye because of the rapid cooling of the material. The most widespread extrusive rock is **basalt**.

Sedimentary rock forms at the surface, on land or under water, by deposition of originally near-horizontal beds. Sedimentary rock subdivides into:

• Detrital rocks, resulting from the deposition of debris from pre-existing rocks resulting from erosion and transport processes (running water, glaciers, wind); the most widespread representatives are **sandstone** and the **argillaceous rocks.**

• Physical/chemical and/or biogenic rocks formed by precipitation of ions in solution and/or living matter; the commonest are **carbonate rocks** and **saline rocks,** still called evaporites.

Metamorphic rocks are the result of profound transformation in the solid state of pre-existing sedimentary or igneous rocks by elevated temperatures and/or pressures. They often exhibit schistosity or foliation accompanied by lineation. The commonest are **schist** and **gneiss** in which the minerals are strongly oriented. Marble and quartzite are massive, completely recrystallised rocks in which the orientation of the minerals (calcite or quartz) is hardly visible to the naked eye.

It is important to bear in mind possible variations in the facies of rocks belonging to the same geological stage and the fact that some common names deriving from the geological map do not always tally with the lithology of the rock concerned. Reference should be made to the full description in the description accompanying the map.

It is preferable to use the terms in Appendix 1 and avoid employing unusual complex names.

2.1.2 – Petrography and mineralogy

A petrographic description covers the following observations with the naked eye or magnifying glass or (preferably) by inspection of thin sections under the microscope:

• Identification of minerals present

• Size and arrangement of the minerals (texture)

• Proportions of the different constituents

• Voids and discontinuities (pores and fissures).

Mineralogical analysis of the constituents establishes the mineral composition of the rock and yields information on its properties such as weathering potential, swelling potential, ability to stick, abrasion potential, etc.

The mineralogical analysis is usually performed by X ray diffraction on a small powdered specimen. It allows the identification of the minerals present and, after interpretation, yields the semi-quantified composition. Special preparation is needed if it is suspected that swelling clay minerals might be present.

Additional quantitative determinations of $CaCO_3$, silica, sulphates, organic matter, etc. refine the identification.

The clay fraction, if present, must be characterised from the Atterberg limits. A methylene blue absorption test will estimate the activity of the clay fraction (French standard NF P 94-068).

2.1.3 – Alteration of the minerals in the rock matrix

Alteration of the matrix is the result of physical/chemical changes in the constituent rock minerals. It is usually associated with major changes in the physical and mechanical properties of the rock. Some minerals are subject to dissolution (e.g. calcite, gypsum), other to decompositione (e.g. biotite, plagioclase). As a general rule, the rock loses cohesion. The process is usually subdivided into

• Hydrothermal alteration, usually confined to the walls of major discontinuities in which fluids from deep-lying sources circulate. It frequently causes major mineralogical changes (appearance of special minerals such as chlorite, etc.), usually accompanied by significant changes in mechanical properties.

• Weathering working down from the surface to sometimes considerable depth. Processes include solution of gypsum, calcite, etc., mechanical disruption (increased microcracking) and mineralogical changes producing clay minerals.

The intensity of alteration and weathering of the matrix can be quantified by mineralogical analysis and indirectly by tests such as the methylene blue test and measurement of ultrasound wave velocity.

A clear distinction must be made between the degree of alteration of rock as taken from a borehole or at the tunnel face and its potential of alterability when exposed to the atmosphere.

Photography I– Hydrothermal alteration – Granite (Ghangzou China)

2.1.4 – Densities (French standard P 94-410-1/2/3)

Different densities (M.L.$^{-3}$ dimensions) are applicable according to the condition of the material.

• **Natural density** $\rho = m/v$

Ratio between dry mass md of the oven-dried sample and the volume V of the sample including any air it contains.

• **Dry density** $\rho_d = m_d/v$

Ratio between dry mass m_d of the oven-dried sample and the volume V of the sample including any air it contains.

• **Density of solid particles** $\rho_s = m_s/v_s$

Ratio between the dry mass of solid particles ms in a powdered specimen and the volume v_s occupied by the particles (measured in a pycnometer). This characteristic of the solid phase of the rock material is directly dependent on the mineral composition of the rock. Appendix 1 lists values for the more common minerals.

2.1.5 – Volume weights

Corresponding unit weights ($M.L^{-2}.T^{2}$ *dimensions*) γ, γ_d and γ_s to he above unit masses are obtained by multiplying the unit masses by the acceleration due to gravity g = 9.81 m/s²:

$$\gamma = \rho \times g$$

2.1.6 – Moisture content (French standard P 94-410-1)

The moisture content by weight w is the ratio, expressed as percentage of the mass of water mw to the mass of the dry material m_d:

$$w(\%) = (m_w/m_d) \times 100$$

2.1.7. Porosity (French standard P 94-410-3)

The porosity n is the ratio, expressed as percentage of the volume of voids v_v to the total volume of the rock sample v:

$$n\ (\%) = (v_v/v) \times 100$$

The porosity is governed mainly by the presence of globular voids (pores) and much less by fissures (flat, thin voids). Some voids may be inaccessible to saturation water (occluded voids).

Values for porosity classes are listed in Table 2.

CLASS	POROSITY N VALUES	DESCRIPTION
P 1	0 % < n < 1 %	Very low porosity
P 2	1 % < n < 5 %	low porosity
P 3	5 % < n < 15 %	Moderate porosity
P 4	15 % < n < 30 %	High moderate porosity
P 5	n > 30 %	Very high porosity

Table 2 – Porosity classes for rock matrix

In the majority of sedimentary rocks, a dry unit mass ρ_d of the order of 2.7 t/m³ is a good indicator of the porosity.

2.1.8 – Degree of saturation

The degree of saturation with water S_r is the ratio, expressed as percentage of the volume of water in the sample v_w to the volume of voids v_v; it is the percentage of the pore space filled with water.

$$Sr = (v_w/v_v) \times 100$$

Rock is described as 'dry' when S_r = 0. It is 'saturated' when S_r = 100%.

2.1.9 – Permeability

The permeability k of a rock sample is described by a coefficient relating the flow Q passing across a surface S under a hydraulic head gradient i (Darcy's law).

$$Q/S = k \times i$$

The dimension of k is a velocity ($L.T^{1}$).

The permeability of the rock matrix is strongly influenced by microcracking (interconnected voids) and therefore varies with the state of stress. The proper choice of representative samples and their pre-test condition is particularly important. Laboratory tests are done with special longitudinal, radial, etc. permeameters or triaxial apparatus. If permeability is found to be anisotropic, tests should be made in several directions.

Knowledge of the matrix permeability is essential only for some underground projects (mined storage, waste disposal by containment, etc.).

2.1.9 – Ultrasound wave velocity (French standard p 94-411) – Continuity index

Ultrasound wave velocity yields information on alteration and weathering and/or fissuration and porosity.

Measuring wave velocity in several different directions may reveal anisotropy due to preferential orientation of microcracks or rock structure.

There are two types of wave:

- Compression (or longitudinal or P) waves noted Vp
- Shear (or transverse or S) waves, not routinely measured, noted Vs.

The continuity index IC of rock is defined as the ratio between Vp measured on samples and the theoretical Vp* derived from the mineral composition of the sample:

$$IC\ (\%) = 100 \times (Vp/Vp^*)$$

Vp* is the harmonic mean of the Vpi wave velocities in the constituent rock minerals (Appendix 2) multiplied by their proportion by volume c_i:

$$1/Vp^* = \sum c_i/Vp_i$$

An approximation is usually possible. It consists of estimating the theoretical wave velocity Vp* from the table in Appendix 3 showing maximum theoretical wave velocities for some rocks, assumed to be sound without pores and fissures.

The continuity index for the same petrographic composition falls as pore porosity increases; the trend is even more marked as microcrack porosity increases.

Continuity classes based on IC value are listed in Table 3.

CLASS	CONTINUITY INDEX IC	DESCRIPTION
IC 1	IC > 90 %	Very high continuity
IC 2	75 %< IC < 90 %	High continuity
IC 3	50 %< IC < 75 %	Moderate continuity
IC 4	25 %< IC < 50 %	Low continuity
IC 5	IC < 25 %	Very low continuity

Table 3 – Rock matrix continuity classes

2.2 – MECHANICAL PARAMETERS

The mechanical parameters relevant to geotechnical classification and to the choice and optimisation of tunnelling techniques and plant are determined in the laboratory from representative samples.

Attention must be given to any **anisotropy** in measured properties. Parameters describing mechanical behaviour very often differ according to the orientation of the sample with respect to bedding (in sedimentary rocks) or foliation (in metamorphic rocks, see example in figure 1). **With some rocks, the anisotropy ratio, defined as the ratio between maximum and minimum values of a parameter, measured at different orientations, may be in excess of 5.**

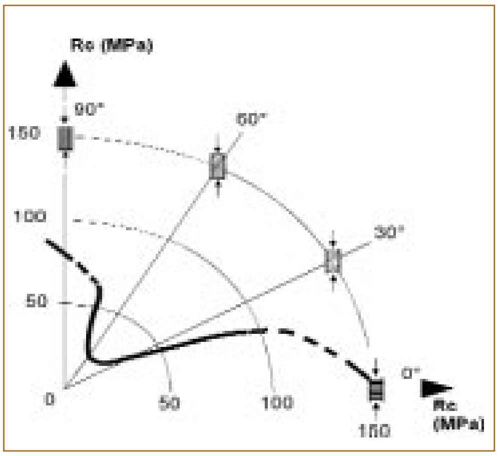


Figure 1 – Example of anisotropic uniaxial compressive strength (Lias shales, Grand'Maison, France, 1980)

Mechanical tests should be made in several directions; this must always be done with metamorphic rocks.

2.2.1 – Deformability: instantaneous behaviour

2.2.1.1 – Young's modulus (French standard P 94-425)

In a uniaxial compression test, the Young's modulus of elasticity E is defined as the slope of a loading/unloading cycle on the axial stress/strain curve $\sigma_1 = f(\varepsilon_1)$ at half the failure stress of the sample σ_c.

Classes based on stiffness values (the reciprocal of deformability) are listed in Table 4.

CLASSES	YOUNG'S MODULUS	DESCRIPTION
DE 1	E > 50 GPa	Extremely stiff matrix
DE 2	20 GPa < E < 50 GPa	Very stiff matrix
DE 3	5 GPa < E < 20 GPa	Stiff matrix
DE 4	1 GPa < E < 5 GPa	Fairly stiff matrix
DE 5	0,1 GPa < E < 1 GPa	Low stiff matrix
DE 6	E < 0,1 GPa	Very low stiff matrix

Table 4 – Rock matrix stiffness classes (reciprocal of deformability)

2.2.1.2 – Poisson's ratio (French standard P 94-425)

In the uniaxial compression test, the Poisson's ratio n is defined as the ratio between the slopes of the straight-line limbs of the $\sigma_1 = f(\varepsilon_3)$ and $\sigma_1 = f(\varepsilon_1)$ curves.

$$\nu = d\varepsilon_3 / d\varepsilon_1$$

ε_1 = axial strain

ε_3 = transverse strain.

Values of the Poisson's ratio for different rocks generally fall between 0.15 and 0.40.

2.2.2 – Deformability: time-dependent behaviour related to creep

2.2.2.1 – Definition

Time-dependent effects take the form of changes in strain and/or stress over time. They have three causes:

1. They may be related to an intrinsic rheological property of the rock whose deformation under constant load increase over time. It is mainly encountered in evaporites (rock salt, potash, gypsum, etc.), argillaceous rocks (marl, claystone) and some carbonate rocks such as chalk.

2. They may be the consequence of damage. In any rock, time-dependent effects more or less appear when the microcracking threshold is exceeded.

3. They may be due to changes over time in the pressure of fluids in pores and fissures, due to changes in boundary conditions of flow patterns caused by construction of the works (effect of consolidation and fluid flow pressures).

Only time-dependent effects associated with creep mechanisms 1 and 2 are considered here. They produce viscoelastic (reversible) or viscoplastic (irreversible) responses in the rock matrix. They have nothing in common with responses due to fluid flow (mechanism 3).

Time-dependent behaviour is more marked as higher temperatures reduce viscosity. This is particularly noticeable in evaporites. High stresses have the same effect, especially deviator stresses.

The long-term performance of underground structures may be affected by changing stresses and strains, and progressive loss of strength. The magnitude of these processes is frequently different *in situ* from in the laboratory, although laboratory tests do yield a significant approximation of the importance, pattern and order of magnitude of time-dependent behaviour to be expected *in situ.*

2.2.2.2 – Laboratory characterisation of time-dependent behaviour: creep test

In studying time-dependent rock behaviour in the laboratory, the commonest test is the **creep test** which is a stress-controlled test to measure strain under constant stress. The **relaxation test,** a strain-controlled test which measures stress changes under constant strain, is not often used.

The creep test is usually preferred. It offers a reasonable simulation of conditions obtaining around underground openings; after the transient tunnelling period, the state of stress around the opening thereafter remains substantially unchanged over time.

If laboratory tests characterising time-dependent or long-term rock behaviour are to have maximum significance, test conditions (temperature, stress range, degree of saturation, drained or undrained boundary condition, etc.) must reflect actual in situ conditions of the structure and the host materials.

The creep test may be performed under uniaxial or triaxial pressure and drained or undrained conditions, preferably at several different deviator stresses, to determine the following parameters (figure 2):

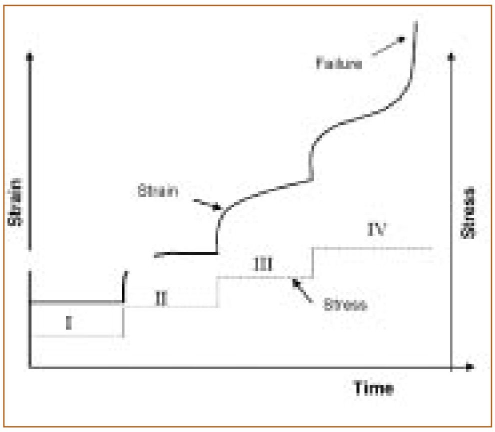


Figure 2 – IIncremental creep test

I No creep: applied stress lower than creep threshold
II Transient (primary) creep decreasing over time: applied stress slightly greater than creep threshold
III Creep slowing down over time: applied stress lower than long-term strength
IV Three-phase primary, secondary (stationary) and tertiary (culminating in failure) creep: applied stress greater that long-term strength

• **Creep threshold,** the state of stress below which creep is negligible (the threshold may be near-zero for some evaporites).

• **Creep rate,** which generally increases with deviator stress.

• **Creep acceleration threshold** beyond which **"tertiary creep"** occurs and culminates in failure; it characterises the long term strength of the rock. In rocks exhibiting no (or little) creep under normal civil engineering conditions (granites, etc.), long term strength is of the order of 90% of the strength measured in quick tests (such as the standard uniaxial compressive strength test); in chalks, it is of the order of 50%; it falls to 30% or less for evaporites.

Creep rates cover a very extended range, depending on the stress ranges concerned:

• The fastest rates ($>10^{-5}s^{-1}$) cover the range of instantaneous strain as measured in the laboratory.

• The slowest rates ($<10^{-11}s^{-1}$) are scarcely measurable but represent deformation over geological time.

Creep rates relevant to actual tunnels, chambers, storage cavities, etc. will lie somewhere between the two extremes. Feedback from real projects is still scarce and possible rates cover such a wide range (being governed by the characteristics of the rock mass and the underground structure) as to make any forecast of time-dependent creep around a real tunnel highly problematical (see para. 4.2.1.5 below).

2.2.3 – Time-dependent behaviour related to swelling

2.2.3.1 – Swelling potential

Rock **swelling** is due to the rock increasing in volume over time, simultaneously with increasing moisture content (wetting under saturated or unsaturated conditions). Changes in the state of stress is a promoting factor. When rock expansion is constrained, high stresses may result.

Rock swelling has two causes:

1. Water may be taken up by hydrophilic minerals, mainly swelling clay minerals such as **smectite**, some hydroxides and sulphates.

2. Change from **anhydrite** ($CaSO_4$) to gypsum ($CaSO_4$,$2H_2O$).

Smectite-type minerals are relatively commonplace and might be present whenever the rock contains clay minerals: clay, marl, molasse, fault gouge, karst infill, weathered igneous and metamorphic rock.

Anhydrite and gypsum occur either as large masses in sedimentary beds or intruded into large tectonic discontinuities, or again, finely scattered within marl or other material.

Three conditions must be fulfilled for swelling to occur:

1. Potentially swelling minerals in the rock.

2. Available water.

3. A state of stress that permits volume increase.

2.2.3.2 – Identification of swelling potential

IIt is strongly recommended to identify the risk of swelling at a very early stage, as follows:

• Qualitative observation of behaviour by immersing specimens in water and seeing how fast they break up.

• Methylene blue tests to determine the specific surface and clay character of the material.

• Total or semi-quantitative (R.X.) mineralogical analyses for the proportion of swelling clay mineral and/or anhydrite and gypsum present.

• If anhydrite is present, microcracking should be investigated, since water take-up is directly linked to the exchange surface area available.

Additional information can be obtained with the electronic microscope to gain a better insight into the distribution of the clay flakes and any other special features liable to favour or oppose swelling (such as smectite being encapsulated in a calcite envelope for example).

2.2.3.3 – Quantification of swelling potential

To confirm the risk of swelling, mineralogical analysis must be supplemented with laboratory tests.

The International Society for Rock Mechanics ISRM recommends three laboratory tests to characterise rock swelling potential (Int. J. of Rock Mech. and Min. Sci., 1999, vol. 36, 291-306):

1. Measurement of axial swelling pressure at constant volume (determination of swelling pressure σ_g).

2. Determination of unrestrained axial and radial swelling strain.

3. Measurement of axial pressure versus axial strain (to determine swelling index C_g).

The third test derives directly from the Huder-Amberg test developed in the early seventies to characterise swelling in soils (cf. Appendix 4).

The swelling test for soils described in French standard P 94-091 uses the oedometer to measure swelling on four specimens assumed to be identical. The specimens are wetted and subjected to four different levels of axial stress. The test consists of measuring the thickness increase $\Delta h/h$ of each of the four specimens and correlating thickness change with the corresponding axial stress levels by fitting a straight line on a semi-log plot ($\Delta h/h - \log \sigma$).

The Huder-Amberg test uses the same type of interpretation but has the advantage of requiring only one test specimen, and this limits the ever-present problem of scatter inherent in irregularities in material type and condition, when using multiple specimens.

In practical terms, the first step should be to measure the axial swelling pressure at constant volume. Next, axial pressure is measured versus axial strain (Huder-Amberg type), the specimen being wetted for the first time with axial pressure substantially equivalent to the previously determined swelling pressure.

Axial swelling pressures may vary greatly in different materials, from near-negligible (less than 0.1 MPa) to several MPa in some marls. There is frequently severe scatter in the swelling pressures measured on any given material. This is why it is advisable to perform several tests, regardless of the type of test used to quantify swelling potential.

2.2.3.4 – Comments

Materials containing clay minerals (clay, marl, molasses) frequently exhibit considerable anisotropy, due to the way they were formed. This **anisotropy** may lead to very different swelling potentials when tested in different directions, especially the directions parallel and perpendicular to the plane in which the clay particles were deposited. Tests must be made in these two directions for a proper characterisation of swelling.

The composition of the **water** used in the test may exert a strong influence on the development of swelling. Certain chemicals may promote or inhibit swelling. The test water should therefore be clearly identified. Swelling may develop very differently *in situ*, depending on whether the water comes from the surrounding rock or from some external source (from inside the tunnel).

2.2.4 – Mechanical strength

2.2.4.1 – – Unconfined compressive strength σ_c or uniaxial compression test (French standard P 94-420

The failure stress in uniaxial compression is defined as

$$\sigma_c = F_{max}/A$$

F_{max} = maximum axial force reached in the test

A = area of the pre-test circular cross section of the test specimen.

Rock strength classes used in the present Recommendations[3] and ISRM appear in Table 5.

[3] They differ from the classes used in the AFTES 1978 Recommendations

Photo 2 - Uniaxial compression test

UNIAXIAL COMPRESSIVE STRENGTH σ_c	CLASS	DESCRIPTION
$\sigma c > 200$ MPa	RC1	Extremely strong matrix
100 MPa < σc < 200 MPa	RC2	Very strong matrix
50 MPa < σc < 100 MPa	RC3	Strong matrix
25 MPa < σc < 50 MPa	RC4	Moderately strong
5 MPa < σc < 25 MPa	RC5	Low strong matrix
1 MPa < σc < 5 MPa	RC6	Very low strong matrix
σc< 1 MPA	RC7	Extremely low strong matrix

Table 5 – Uniaxial compressive strength classes

Soft rock and stiff soils often classify as Class RC6 and RC7.

Rock tensile strength σ_{tb} is determined by the indirect Brazilian test (figure 3) using the procedure described in French standard P 94-422.

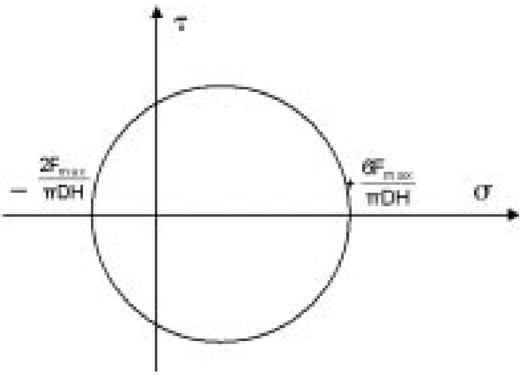


Figure 3 – Stress state at failure at specimen centre in compression test

Photo 3 – Specimen after failure in triaxial test (French standard p 94-422)

The test breaks a cylinder of diameter D and height H by applying a compressive load F at diametrically opposed sides of the cylinder. The stress at failure of the Brazilian test specimen σ_{tb} is given by the equation

$$\sigma_{tb} = 2\ F_{max}/\pi \times D \times H$$

F_{max} = load at failure

H = cylinder height

D = cylinder diameter.

2.2.4.2 – Brittleness index FR

The brittleness index FR is defined as the ratio σ_c/σ_{tb}.

It is useful as characterising the drillability and failure type for hard rocks (σ_c > 25 MPa).

FR usually ranges from 5 to 30. Brittleness classes are listed in Table 6.

CLASS	BRITTLENESS INDEX FR VALUES	DESCRIPTION
FR 1	FR >25	Very brittle matrix
FR 2	15 < FR < 25	Brittle matrix
FR 3	10 < FR < 15	Moderately brittle matrix
FR 4	FR < 10	Low brittle matrix

Table 6 – Rock matrix brittleness classes

2.2.4.3 – Point load compression test, so called– Franklin test (French standard P 94-429

The point load compression tests involves breaking rock fragments of indeterminate shape or core pieces between two cones with spherical tips (dimensions standardised). Specimen thickness between points may range from 25cm to 100cm. Test specimens are usually pieces of rock taken from 50mm dia. cores, although charts are available to correct results from other core sizes (Broch & Franklin, Int. J. of Rock Mech. and Min. Sc., 1972, vol 9, 669-697).

Test results are expressed as a strength index I_s (in MPa):

$$I_s = F/D^2$$

F = load at failure

D = specimen diameter or distance between points.

The index obtained with a 50mm diameter is written I_{s50}.

The point load compression test can be performed with very lightweight apparatus on site. Correlations yield an estimate of rock unconfined compressive strength:

$$20\ I_{s50} < \sigma_c < 27\ I_{s50}$$

However, the point load compression test can never be considered as a substitute for the uniaxial compression test.

2.2.5 - Triaxial test and failure criteria

2.2.5.I – Triaxial test (French standard P 94-423)

The triaxial test is an axial compression test in which a constant confining stress σ_3 is applied to the specimen. The axial compressive stress is continued to failure.

Determination of the failure criterion for a rock involves performing several triaxial tests with increasing confining stresses. There must be at least four test runs, including a uniaxial compression test in which $\sigma_3 = 0$.

The full $\sigma_1 - \sigma_3 = f(\varepsilon_1)$ curve is plotted for each test at a different confining stress to find the peak value of the deviator failure stress $\sigma_1 - \sigma_3$, plus, possibly, the residual (constant) stress at which the specimen shears along the failure plane.

Measurement of axial and transverse strain in the course of the test leads to values for the Young's modulus and Poisson's ratio under confining stress conditions.

This is the standard soils mechanics criterion written as

$$\tau = c + \sigma_n \times \tan \varphi$$

- σ_n is the normal stress and and τ is the shear stress on the failure surface
- c is cohesion
- φ is the internal friction angle.

In terms of the principal stresses, with $\sigma_1 > \sigma_3$, the Mohr-Coulomb criterion is written

$$\sigma_1 = [\sigma_3(1 + \sin\phi) + 2c(\cos\phi)] / (1 - \sin\phi)$$

This limit strength criterion can apply to the elastic limits, peaks or plateaus on the stress strain curves from triaxial tests (total or effective stresses, drained or undrained tests).

The Mohr-Coulomb criterion may suit the mechanical behaviour of certain rocks in the moderate confining stress range. More generally, it may be acceptable for representing the behaviour of a given rock **within a certain limited range of confining stresses** (linearisation of a parabolic criterion).

2.2.5.2 – Hoek & Brown criterion

This parabolic criterion is well suited to the mechanical behaviour of rock and is usually used for these materials.

The criterion for intact rock is written

$$\sigma_1 = \sigma_3 + \sigma_{ci} \times [(m_i\,\sigma_{ci}) + 1]^{1/2}$$

in which

- σ_{ci} is the uniaxial compressive strength of the intact rock material
- m_i is a constant dependent on rock type.

From the standard expression of a parabolic criterion expressed in terms of uniaxial compressive σ_{ci} and σ_{ti} tensile failure stresses (Appendix 5), it can be seen that parameter m_i is very close to the brittleness index FR:

$$m_i \approx \sigma_{ci}/\sigma_{ti} = FR$$

Photo 4 – Triaxial test apparatus

2.2.6 – Parameters for resistance to excavation

The parameters examined in this section concern the response of rock to various means of breaking it up (its decohesion). They are useful for assessing rock in respect of excavating and crushing techniques, in order to refine forecasts as to

- mechanised excavation, through hardness and drillability tests,
- excavating tool wear and consumption, through abrasion tests,
- crushing performance, through fragmentation tests.

Test results are interpreted with the aid of multi-criteria correlation models which also refer to other parameters (mineralogy, mechanical properties, discontinuities), investigated in another location.

2.2.6.1 – Hardness and "drillability"

These tests can be classified under three headings based on the techniques used.

1. Tests assessing the penetration of drill bits and cutter tools, such as the CERCHAR, Siever and other tests.

2. Static indentation tests which assess the mark left by a point applied to the rock, such as the punch test, Vickers, Knoops and Schreiner tests, micro-indentation test, etc.

3. Rebound tests, in which the rebound of a known mass hitting the rock is measured (Schmidt hammer).

The commonest test is the penetration test derived from the test first described by French research institute CERCHAR: **"Drill bit penetration resistance index" – French standard P 94-412.**

It characterises rock **hardness** via its resistance to penetration by a drill bit under standard test conditions. The hardness test is suited to fine grained rocks of moderate to low strength.

Rock hardness classes based on the CERCHAR-INERIS rules are listed in Table 7.

2.2.6.2 – Abrasiveness

CLASS	HARDNESS VALUES	DESCRIPTION
DU 1	> 120	Extremely hard matrix
DU 2	80 - 120	Very hard matrix
DU 3	40 - 80	Hard matrix
DU 4	20 - 40	Moderately hard matrix
DU 5	5 - 20	Soft matrix
DU 6	< 5	Very soft matrix

Table 7 – Rock matrix hardness classes based on CERCHAR-INERIS test results

The abrasion potential of a rock is determined by its mineralogical composition, especially the percentage of **quartz** it contains, and its intergranular cohesion and grain size.

Abrasiveness can be characterised by two conventional, standardised indexes; it must be recognised that they bear no relationship to each other.

2.2.6.2.1 – Point scratch test – French standard P 94-430-1

The first index, AIN[4] comes from the CERCHAR-INERIS point scratch test. Test results are expressed as an abrasiveness index characterising the ability of a rock to cause wear of the cutting tool.

Rock abrasiveness classes derived from this index are listed in Table 8.

CLASS	ABRASIVENESS INDEX A_{IN} VALUES	DESCRIPTION
A_{IN} 1	>4.0 *	Extremely abrasive matrix
A_{IN} 2	2.0 – 4.0	Very abrasive matrix
A_{IN} 3	1.0 – 2.0	Abrasive matrix
A_{IN} 4	0.5 – 1.0	Low abrasiveness matrix
A_{IN} 5	< 0.5	Very low abrasive matrix

*Quartz and gemstones register higher than 6

Table 8 – Rock matrix abrasiveness classes based on results of CERCHAR-INERIS test

2.2.6.2.2 – Rotating drilling bit test (French standard P 94-430-2)

The other index is derived from the LCPC abrasiveness test. This test is suitable for rocks whose tensile strength is greater than

[4] CAI (CERCHAR Abrasiveness Index) in English-speaking countries

1 MPa. The result is expressed by the abrasiveness index ABR calculated from wear of a drill bit rotated in a body of 4/6.3 aggregate obtained from the rock under test.

Rock abrasiveness classes based on the LCPC test are listed in Table 9.

CLASS	ABRASIVENESS INDEX A_{BR} VALUES	DESCRIPTION
A_{BR} 1	$A_{BR} > 2000$	Very highly abrasive
A_{BR} 2	$1500 < A_{BR} < 2000$	Highly abrasive
A_{BR} 3	$1000 < A_{BR} < 1500$	Moderately abrasive
A_{BR} 4	$500 < A_{BR} < 1000$	Low abrasiveness
A_{BR} 5	$0 < A_{BR} < 500$	Very low abrasiveness

Table 9 – Rock matrix abrasiveness test based on results of LCPC test

2.2.6.3 – Norwegian tests: DRI index

The Norwegian Drilling Rate Index defined by the University of Trondheim (Movinkel & Johannessen 1986) combines the results of an S20 fragmentation test and an SJ drill bit penetration test. The DRI chart and a description of the S20 and SJ tests appear in Appendix 6.

2.2.7 – Other tests

2.2.7.1 – – Fragmentability (French standard P 94-066) – Degradability (P 94-067)

These standard tests characterise the susceptibility of the rock to fragment and degrade under the action of drilling and excavating machines. They are mainly used when rock cuttings are to be used as above-ground building material, but they are undoubtedly useful in tunnelling work to characterise spoil muck.

2.2.7.2 – Los Angeles test (French standard P 18-573) – Micro-Deval test in presence of water (P 18-572

It will be remembered that the Los Angeles test characterises the resistance of rock to fragmentation while the micro-Deval (MDE) test characterises the rock's resistance to wear in the presence of water. These are essential parameters when dealing with hard rocks, for characterising the possibility, when applicable, of using muck as something more elaborate than plain fill (ballast, road-making and concrete aggregate, road foundation courses, etc.).

3 – CHARACTERISTICS OF DISCONTINUITIES

The system of discontinuities in the rock mass must be investigated in detail **at prototype scale of the project,** mainly through statistical analysis in order to consider the natural variability in their geometrical and mechanical parameters. Different data acquisition methods are appropriate to different scales of observation:

• aerial photography and surface geophysics at regional scale

• outcrop and borehole surveys at local scale.

When core drilling, it is always recommended to accompany the geologist's logs (geological drill logs which include records from the various borehole loggings and joint mapping) with colour photographs of all the core boxes; photographs should include a scale, colour chart and legible identification markings. Photographic records may indeed provide the engineer with abundant valuable information, especially about discontinuities. However, they could not replace the direct inspection of the cores which is essential , even if it is often difficult.

3.1 – JOINT IDENTIFICATION PARAMETERS

3.1.1 – Types and origins of the discontinuities

The term discontinuity is used in rock mechanics in a very general sense, to designate any interuption of the continuity in the rock material with its attendant mechanical, hydraulic and thermal properties. The surface can be treated, over a certain distance, as a plane displaying zero or low tensile strength.

Discontinuities represent a wide variety of types of surfaces whose geological identification conveys important information on some of their geometrical and mechanical features (parameters). Instead of the conventional terminology of joints, fractures or cracks, preference is given to the following terms:

• **Bedding planes** are surfaces separating sedimentary rock strata. They are very persistent and may contain clay material resulting in very low shear strength.

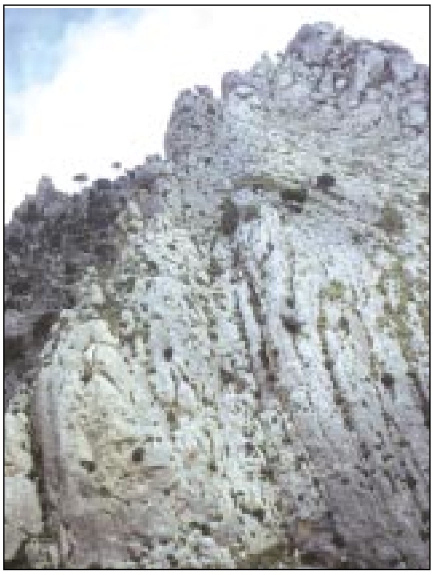


Photo 5 – Rock mass with complex tectonic history. Tibi region, Spain

• **Joints** properly so called, are discontinuities between two rock compartments without any apparent relative movement between them. The joint may be tight or open with generally planar, relatively smooth walls, and may extend over a distance measured in decimetres to decametres. They often occur as joint sets running in two or three directions.

• **Faults** are the result of relative movement between the two rock compartments between which they lie, caused by the stress field during the relevant tectonic episode (normal, reverse and strike-slip faults). Persistence is very variable (metres to several kilometres long) and they frequently contain infilling material (gouge) with poor mechanical properties.

• **Schistosity planes** cause the rock to break into thin parallel layers under the action of tectonic stresses. Persistence is variable and small unitary schistosity planes may form surfaces persisting for very long distances through localised failure of rock bridges between closely-spaced parallel or stepped discontinuities.

• **Fracture zones** are assemblages of discontinuities of variable persistence and orientation, organised in a more or less planar pattern.

• **Lithological contact surfaces** between host rock and veins often form virtual discontinuities, although they may be real surfaces where differential weathering has occurred.

Note : If in French, discontinuity is the generic term, in English, "joint" is the term currently used for discontinuity in this recommendation. For example, joint set means set of discontinuities.

3.1.2 –Description of discontinuities

Discontinuities within a homogeneous zone are systematically mapped in order to characterise, statistically, a joint system usually consisting of several joint sets with different strikes and possibly a few singular units which may be important from the rock mechanics viewpoint (faults, altered veins, etc.).

These observations are made along a scanline marked on natural outcrops, tunnel walls or the sides of excavations, or from boreholes, whether or not cores have been recovered (down-the-hole viewing techniques).

In order to ensure these measures are representative, surveys must be plotted in several spatial directions and must cover a sufficiently large volume, commensurate with the average joint density. Lastly, it is important that the method used to collect the data be clearly described.

The full procedure for the systematic analysis of joint systems is based on characterising each discontinuity with the following eight parameters:

1. **Strike:** describes the position of the discontinuity plane in space with respect to North. Two conventions are used (figure 4):

• The first, based on the direction of the horizontal line of the plane α (0-180°), the dip angle β (0-90°) and the direction of the dip vector, is normally used in field work.

• The second, which uses the azimuth of the dip vector α_p (0-360°) and dip angle β (0-90°), is the method recommended by AFTES for plotting results.

Strike is the basic parameter for establishing the initial identification of joint sets. It also determines the shapes of the individual blocks comprising the rock mass, and thereby, the anisotropy which will govern hydraulic and mechanical behaviour.

Photo 6
Sangatte grey chalk

2. **Spacing**: is the perpendicular distance (modal or average) between adjacent discontinuities in a set. In fact, the distance between two successive intersections of the joint trace with the survey line can be measured in the field. This measurement is frequently biased because it depends on the joint persistence (for any given number of traces on a surface, the longer ones have more chance of being cut by the line of the survey and therefore appear to be more closely spaced) and the direction of the survey line.

3. **Persistence**: Joint persistence or size concerns the total area of the discontinuity in all directions. It is an important parameter because, along with joint spacing, it governs the connectivity of the discontinuity network and therefore the permeability of the rock mass and the volume of the intact rock blocks. It cannot be measured directly, but can be quantify by observing the discontinuity trace lengths (intersection line of the discontinuity and the surface of exposure. It cannot be measured in boreholes.

4. **Joint wall roughness and waviness**: measured respectively in millimetres or centimetres, and in decimetres to metres. These are crucial parameters because they govern the process of dilatancy and therefore the joint shear strength. Although difficult to measure, every effort must be made to estimate them (cf. para. 3.3.2 Shear Strength Parameters).

5. **Joint wall weathering**: This is an important parameter especially when the discontinuity walls are in direct rock to rock intact, because it governs the deformability and the possibility of dilatancy affecting the shear strength. The degree of alteration can be assessed directly in the field from the description of the weathering products, their thickness and the Schmidt hammer test (Appendix 9).

6. **Joint width** or **aperture**: Size of the gap between the joint walls measured perpendicularly to the joint plane.

7. **Infill**: The nature of the material filling the discontinuity or coating the walls must be characterised, along with its thickness and its mechanical properties.

8. **Presence of water**: Presence of damp spots or water flow.

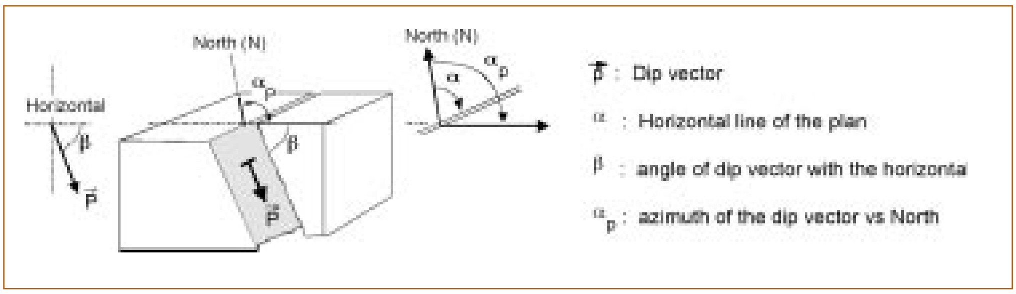


Figure 4 – Attitude of a disontinuty in space

3.2 – CHARACTERISATION OF JOINT SYSTEMS

3.2.1 – Directional joint set patterns

Discontinuities are not arbitrarily oriented in rock, they frequently occur as sets, in numbers determined by the geological and mechanical processes occurring at the time of rock formation and the tectonic history of the rock body. The distribution of a population of discontinuities into directional sets is investigated from the **azimuth** and **dip** parameters but other geometric parameters such as aperture and persistence may also be relevant in identifying **joint sets.**

The very concept of **joint sets** however is a simplification of reality which may sometimes be excessive or unjustified. This type of analysis must therefore be conducted with prudence, making use of the geologist's experience. In all cases, it is necessary to work according to joint type, distinguishing for example between bedding planes, foliation or schistosity, joints proper and faults.

The most frequently used analytical approach is the stereographic projection (see Appendix 7 for more details). On the basis of field observations and stereographic plots, the pattern of joint sets can be described with the help of Table 10.

3.2.2 – Statistical analysis of geometrical parameters for each joint set

Once the directional joint sets (if any) have been identified, statistical analysis of each set can proceed, by means of distribution histograms of geometrical parameters such as strike, persistence and spacing. One can then calculate the means and standard deviations for each parameter, and a distribution function may be fitted.

3.2.2.1 – Joint set orientation

The orientation of each joint set must be considered in relation to the direction of tunnel driving. The dip angle β and the angle δ between the axis of heading A and the direction of the dip vector ap for each joint set dictate the tunnelling method, as described in Table 11. The stereogram in figure 5 is a graphical representation of these classes, with explicit examples in figure 6.

CLASS		ORIENTATION OF DISCONTINUITIES		DESCRIPTION OF TUNNELLING CONDITIONS	
		Angle δ between direction of dip vector α_p and axis of heading	Dip β		
OR 1		Indeterminate	0° to 20°	With subhorizontal beds	
OR 2	a	0° to 30°	20° to 90°	Cross-bed	(a) with dip
	b				(b) against dip
OR 3		30° to 65°	20° to 90°	Intermédiate conditions	
OR 4	a	65° to 90°	20° to 60°	With strike	(a) moderate dip
	b		60° to 90°		(b) steep dip

Table 11 – Joint orientation classes with respect to tunnel axis

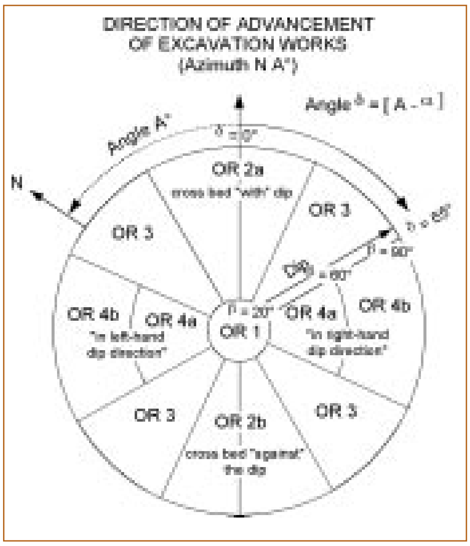


Figure 5 – Joint orientation. Stereogram and polar plot for one joint set

Sectors in the stereogram correspond to the geometrical locus of the poles of joint planes oriented according to OR classes from Table 11. See also sketches in figure 6

3.2.2.2 – Joint spacing in each joint set

The histogram of joint spacing ei in a joint set can be readily obtained from the distances di between discontinuities in the same set intersecting the survey lines, with reference to the angle θ between the survey line and the normal to the mean plane of the joint set ($e_i = d_i.\cos\theta$). The next step is to calculate the mean value ES and standard deviation of the spacing, and various modal values if they emerge clearly from the histogram.

In bedded rock masses, spacing is equal to bed thickness.

Joint spacing classes are listed in Table 12.

3.2.2.3 – Joint set persistence

Joint persistence must be analysed with caution. It can only be estimated from

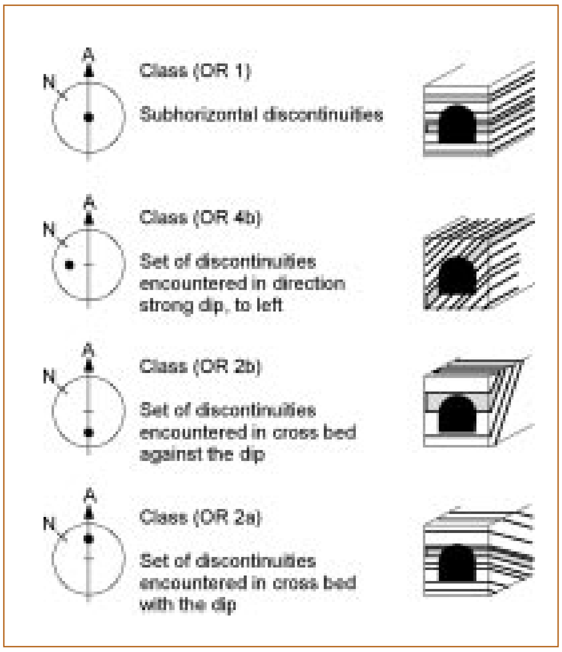


Figure 6 – Sketches of selected OR orientation classes from Table 11 and stereogram data from figure 5
Stereogram and plot of plane pole for the joint set considered (upper hemisphere) at left
Explanatory block diagram at right

CLASSE		DESCRIPTION OF JOINT SET NUMBERS
N 1		Few discontinuities or occasional random discontinuities
N 2	a	One main set
	b	One main set plus random discontinuities
N 3	a	Two main sets
	b	Two main sets plus random discontinuities
N 4	a	Three or more main sets
	b	Three or more main sets and scattered discontinuities
N 5		Abundant discontinuities in no discernible pattern

Table 10 – Classes and descriptions for joint set numbers

CLASS	SPACING (cm)	DESCRIPTION
ES 1	> 200	Very widely spaced discontinuities
ES 2	60 à 200	Widely spaced discontinuities
ES 3	20 à 60	Moderately spaced discontinuities
ES 4	6 à 20	Closely spaced discontinuities
ES 5	< 6	Very closely spaced discontinuities

Table 12 – Joint spacing classes within same joint set

measurements of 2D trace lengths on surfaces (outcrops, adit walls), but these data are often biased. Furthermore, 2D trace data can only be extended to actual 3D joint persistence with the aid of a model.

Appendix 8 reviews recent developments in these two areas.

3.2.3 – Overall joint density indexes

The spatial density of a population of discontinuities is defined as the mean joint area per unit volume of rock. This definition encompasses both joint spacing and joint persistence and cannot be determined directly. It is usually only possible to estimate joint density by counting lengths of massive rock blocks between discontinuities encountered along a survey line in boreholes, outcrops or adit walls.

Joint data (without any morphogenetic breakdown) is reduced for interpretation by means of various indexes and statistical manipulations.

3.2.3.1 – Rock Quality Designation

RQD was defined by D. Deere (1963) as the cumulative length of intact core pieces longer than 4 inches divided by the total length of the core run not more than 1.50m long, expressed as a percentage. Core size must be least NX ($2^1/_8$ inches or 54,7 mm), drilled with a double-tube core barrel, with substantially 100% recovery.

In order not to penalise ground not yielding exactly 100% recovery and systematise RQD records for the purposes of comparison, AFTES recommends calculating RQD from **1-metre core runs**:

RQD, expressed as a percentage, is the total length of pieces of intact core longer than 10 cm divided by core run of 1-metre length.

In order to keep RQD meaningful, the following coring conditions are critical:

• Core diameter greater than 50mm in massive and poorly jointed rock mass; in rock mass of a kind which is inherently jointed or incorporates planes of weakness (schists, mudstones, marls, bedded limestones, etc.), larger borehole sizes are recommended, such as 85mm.

• Length of intact core pieces measured along the core centreline.

• Core recovery index R (percentage of recovered core lengths per core run legth) to be 90-100%; the RQD index for R values lower than 90% is not considered meaningful.

• Only natural discontinuities must be considered; breakage caused by drilling, core handling and placing cores in the boxes should be ignored (this is particularly important in inclined boreholes which are more susceptible to core breakage); doubtful cases should be considered as natural joints and included in the RQD calculation as a conservative measure.

• Discontinuities substantially parallel to the core centreline to be ignored in the count.

• RQD must be determined promptly after the core is recovered in order to forestall any subsequent changes in the material due to swelling, stress release or drying; the recommendation at the beginning of section 3 above to systematically photograph all cores is critically important here. Photographs taken at the time of the RQD count allow any changes to the cores over time to be identified.

RQD classes and assessments of the overall quality of the rock (see para. 4.1.1) that can be derived from the D. Deere approach are presented in Table 13.

3.2.3.2 – Interval between discontinuities (ID index)

The ID index is defined as the mean of the intact rock length between successive discontinuities along a survey line whose length and orientation must be recorded. The counts must be made in several directions, carefully chosen with reference to the characteristic joint strikes and tunnel orientation (this is also valid for the RQD approach).

The joint density classes obtained from the ID index are listed in Table 14

CLASS	RQD %	DESCRIPTION (After D. Deere)
RQD 1	90 to 100	Excellent
RQD 2	75 to 90	Good
RQD 3	50 to 75	Fair
RQD 4	25 to 50	Poor
RQD 5	0 to 25	Very poor

Table 13 – Overall rock quality classes by the RQD method
** *after D. Deere*

CLASS	ID INDEX (CM)	DESCRIPTION
ID 1	> 200	Very low density
ID 2	60-200	Low density
ID 3	20-60	Moderate density
ID 4	6-20	High density
ID 5	<6	Very high density

Table 14 – Joint density classes as determined along survey line

CLASS	ID INDEX (CM)	DESCRIPTION
FD 1	FD < 1	Very low density
FD 2	1 < FD < 2	Low density
FD 3	2 < FD < 5	Moderate density
FD 4	5 < FD < 15	High density
FD 5	FD > 15	Very high density

Table 15 – Joint frequency classes as determined along survey line

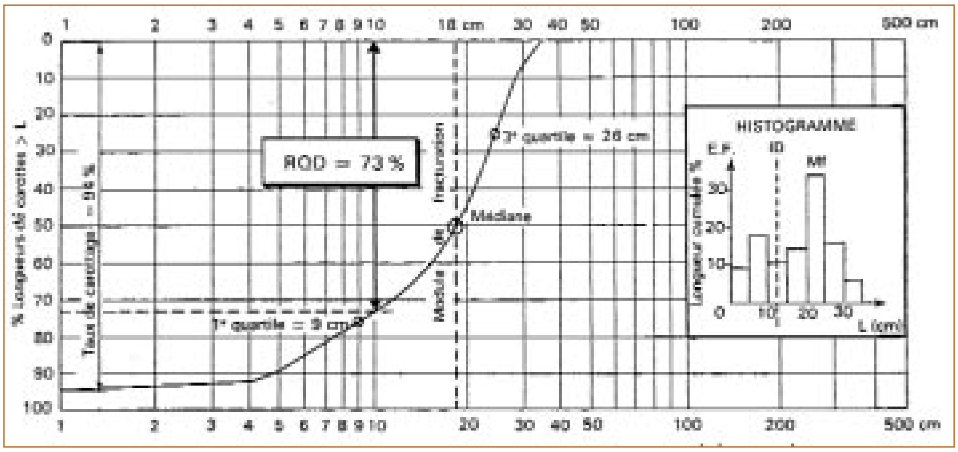


Figure 7 – Histogram and cumulative frequency curve for lengths off pieces of core (after C. Louis 1974)

It is also recommended (figure 7)

- To draw the histograms of lengths for each survey line and calculate standard deviation S and coefficient of variation CV

$$CV = S/ID$$

- To plot the cumulative frequency curve of core lengths (equivalent to a grain size distribution curve) in order to have a complete image of the joint density. This curve can be used to quantify the mean joint index represented by **the median, and dispersion represented for example by the 25% and 75% quartiles.**

The joint frequencies FD can also be calculated (Table 15); it is the number of joints per metre of drilled core, the inverse value of the ID index.

3.2.3.3 – Remarks on use of indexes

It must not be forgotten that the determination of the above indexes from borehole cores requires the coring conditions stipulated in para. 3.2.3.1 always to be adhered to.

Attention is also drawn to the fact that RQD, the most commonly measured parameter, is a quality index for the rock mass but yields very little information on joint density. For example, an RQD of 100 may result from a core run not encountering a discontinuity or exhibiting a discontinuity every 11cm. **AFTES therefore recommends using the ID index to characterise joint density.**

It is also strongly recommended to represent the variations of these indexes along the survey line in the form of diagrams. For borehole measurements, **it is essential to accompany the complete core diagram showing core piece lengths against depth with the FD diagram (expressed in m-1) and the RQD diagram** (figure 8). This arrangement facilitates analysis of the rock mass discontinuities, and the different methods become mutually complementary. Homogeneous zones can be identified, or contrasts between zones with more or less high density of discontinuities. The discontinuity data can also be compared with data from downhole logging performed in the same borehole.

It might also be useful to quantify these variables by a moving mean process (e.g. RQD and FD calculated in one-metre steps in a 4m window moved down the borehole

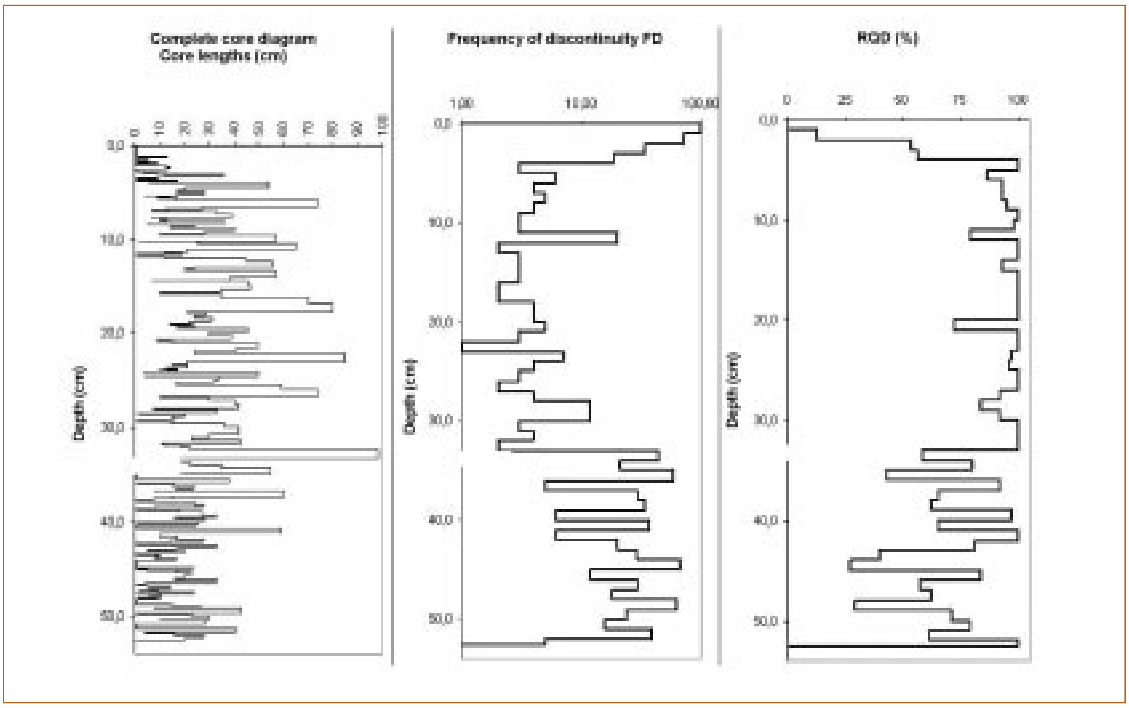


Figure 8 – Characterisation of the discontinuity network in a limestone rock mass from vertical cored borehole (Serratrice & Durville 1997)

in 2m increments) or modify the values of the parameters used to calculate indexes in specific zones in the borehole such as crushed zones which can conventionally be considered as consisting of 1cm fragments.

3.3 – MECHANICAL PARAMETERS OF DISCONTINUITIES

We are interested here in discontinuities without infilling material, otherwise mechanical behaviour would be governed by the behaviour of the infilling material, which should then be studied in itself.

The mechanical characteristics of discontinuities are usually obtained from laboratory tests; in situ tests are much less common because they are more difficult to perform and are more costly. Uniaxial compression tests and shear tests with normal load (French standard NF P94-424) are performed to characterise the behaviour of the discontinuities.

The following leading parameters characterising the mechanical behaviour of discontinuities can be obtained by analysing these laboratory test results :

- deformation parameters: **normal and tangential stiffness,**
- shear strength defined by **peak and residual friction angles and apparent cohesion,**
- a geometric parameter, **dilatancy**, a measure of the deformation in the normal direction accompanying tangential deformation during shear.

3.3.1 – Deformation parameters

3.3.1.1 – Normal stiffness

Uniaxial compression tests on joints oriented perpendicular to the direction of load application always trace a hyperbolic curve of normal stress σ_n versus normal displacement U_n with an asymptote representing the maximum limit of joint closure U_{max}. Total joint opening can be obtained for a non zero tensile stress a, if there are rock bridges across, or filling material in, the joint (Figure 9).

The slope of this curve gives the normal stiffness K_n, defined as

$$K_n = \delta\sigma_n/\delta U_n$$

The value of K_n is dependent on the normal stress and can be expressed in terms of parameters α, U_{max} and K_{ni} (initial normal stiffness) characterising the mechanical behaviour of the joint subjected to uniaxial compressive load, determined by fitting a curve on the test results, although not commonly done.

3.3.1.2 – Tangential stiffness

Similarly, the shear test is used to define the tangential stiffness K_s as the slope of the curve of tangential stress τ vs tangential displacement U_s before failure (figure 9):

$$K_s = \delta\tau/\delta U_s$$

3.3.2 – Shear strength parameters

The behaviour of a discontinuity during a shear test (French standard XP P 94-424) is governed by the nature of the joint walls but more importantly, by their surface conditions. In particular, joint wall roughness, interlocking and weathering play a primordial role.

In the ideal case of a planar and smooth discontinuity, i.e. with no asperities, shear behaviour is entirely controlled by wall friction. Shear strength is usually expressed by the Coulomb criterion:

$$\tau = \sigma_n \times \tan \phi_b$$

in which ϕ_b is the friction angle for a planar joint or basic friction angle, chiefly dependent on the petrographic composition and degree of weathering of the joint walls.

Natural discontinuities generally have walls which are very irregular with abundant asperities of varied shape and size, representing different scales of roughness. Their shear behaviour reveals three fundamental parameters (figure 10):

- **Peak shear strength,** defined by the maximum shear stress τ_p, at which the asperities shear.
- **Residual shear strength** τ_r, characteristic of the friction of the joint walls which come into contact with each other after the asperities have sheared.

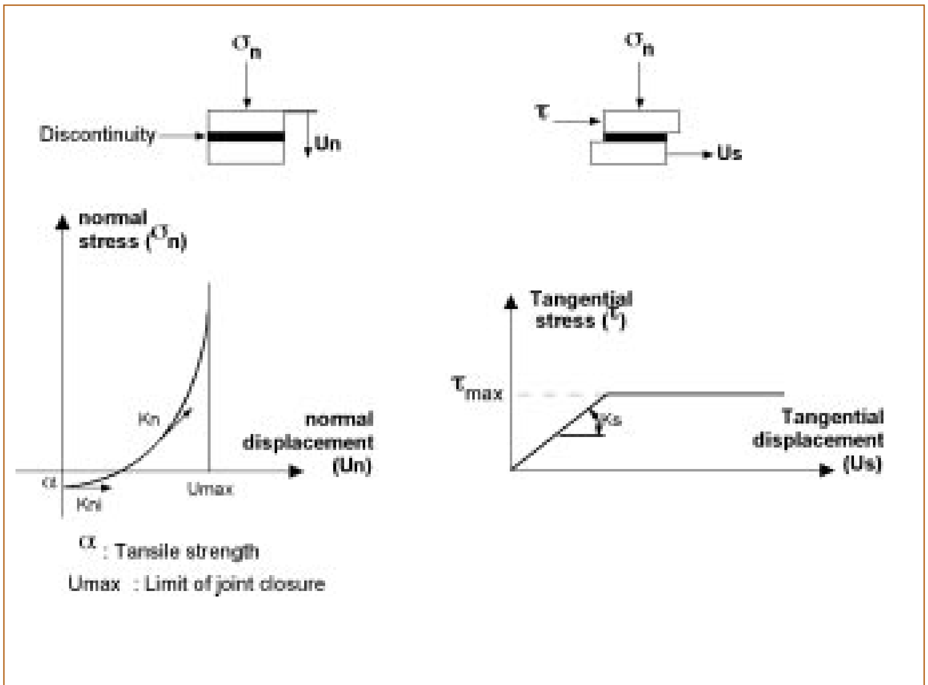


Figure 9 – Joint deformability parameters: normal stiffness Kn and tangential stiffness Ks

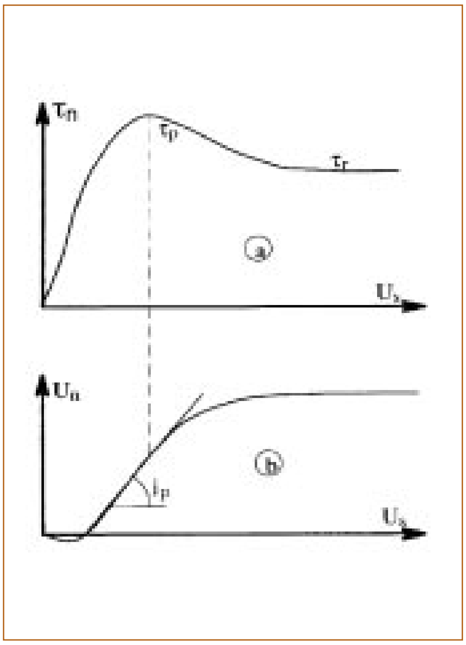


Figure 10 – Shear behaviour of a natural discontinuity
a) tangential stress τ vs tangential strain U_s
b) Normal strain U_n vs tangential strain U_s

• Dilatancy represented by the displacement of the joint walls in the direction normal to the joint plane. It is characterised by the dilatancy angle i (angle of slope of the dilatancy curve of normal displacement U_n vs tangential displacement U_s). This angle reaches a maximum value i_p at the inflection point on the dilatancy curve. This point corresponds to the peak shear stength τ_p reflecting, for a given level of normal stress, the shearing of the sharpest asperities. Beyond this point, dilatancy continues with a lower angle, governed by the inclination of the stronger asperities with a wider base and flatter angles.

Compared to a planar smooth joint, dilatancy leads to an increase in peak strength. It is dependent on joint wall roughness and weathering, and also how the walls interlock and the direction of shear.

The joint shear failure criterion is represented by two curves, characterising the peak and residual strength (figure 11).

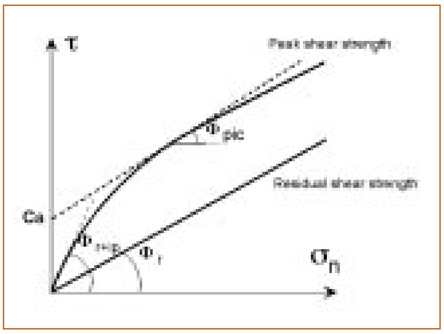


Figure 11 – Failure criterion for a natural discontinuity

Residual strength of discontinuities is not greatly influenced by scale effect and the failure criterion is readily obtained by laboratory testing in the form of a standard Coulomb law characterised by a residual friction angle ϕ_r which differs from the basic friction angle ϕ_b by no more than a few degrees, and a residual cohesion, usually minimal or zero, which is always considered to be 0:

$$\tau_r = \sigma_n \times \tan \phi$$

The **peak shear strength** curve has a progressive shape reflecting the non-linear relationship between shear strength τ and normal stress σ_n. The curve is steep at low normal stresses, reflecting the influence of the sharpest asperities, which are the cause of severe dilatancy. As normal stress increases, more and more asperities fail, dilatancy becomes less and the (τ, σ_n) curve flattens and progressively becomes a straight line. For a limited normal stress range, this curve can be approximated by a straight line:

$$\tau_{peak} = C_a + \sigma_n \times \tan \phi_{peak}$$

C_a is an apparent cohesion which does not express an intrinsic property of the joint wall material but the influence of irregularities in the walls on shear behaviour.

At very low normal stresses, apparent cohesion C_a is close to zero, and ϕ_{peak} is close to $\phi_r + i_p$.

At high normal stresses, apparent cohesion C_a is high and peak friction angle ϕ_{peak} tends towards ϕ_r.

In practice, laboratory tests are not easy to interpret and determination of peak joint strength characteristics involves many difficulties arising from scatter in the data and scale effect.

Using experimental data, Barton (1973) proposed a semi-empirical failure criterion in which peak strength depends on a dilatancy angle i allowing for the joint wall roughness (JRC), joint strength (JCS) and normal stress applied to the joint:

$$\tau_{peak} = \sigma_n \times \tan (\phi_b + i)$$

$$= \sigma_n + \tan [\phi_b + JRC \times \log_{10}(JCS/\sigma_n)]$$

in which ϕ_b is the basic friction angle which differs from the residual friction angle ϕ_r by a few degrees.

JRC is the Joint Roughness Coefficient, a dimensionless coefficient relating to joint wall roughness and size. It can be estimated by comparing joint roughness profiles in the direction of shear with Barton's standard profiles, ranked in ascending order from 0 for a flat smooth discontinuity to 20 for a wavy rough discontinuity (figure 12). JRC also varies with the joint deformability wall displacement: the more asperities are sheared, the lower the value of JRC.

RCS is the Joint Compressive Strength, frequently estimated indirectly *in situ* with the Schmidt hammer (Appendix 9).

σ_n is the normal stress applied to the discontinuity.

At very low normal stresses ($JCS/\sigma_n \geq 100$), the equation gives unrealistic values and Barton suggests using the simplified form:

$$\tau = \sigma_n \times \tan 70°$$

However, the determination of a representative value of JRC for three-dimensional joint wall roughness is not always a simple matter, even on laboratory size samples.

It should be recognised that this approach is confined to discontinuities with thin or no infilling material. When there is a sufficient thickness of infilling material for shearing to occur wholly within the infilling material, shear characteristics will be those of the infilling material, which must be investigated specifically.

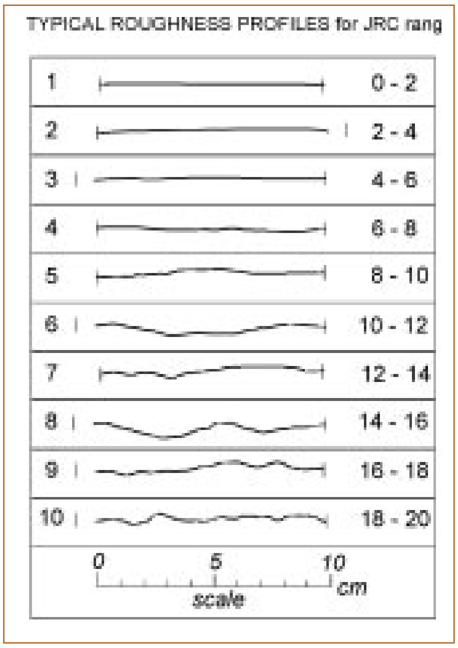


Figure 12 – Standard joint wall roughness profiles (after Barton & Choubey 1977)

3.3.3 – Hydraulic parameters

While the mechanical behaviour of rock joints is mainly controlled by joint wall composition, weathering, roughness and normal stress, other external factors affect behaviour: thickness, composition and moisture content of infilling material, presence of water in joints likely to induce pore pressures modifying normal stress, and boundary conditions affecting the magnitude of displacements.

Fracture fluid flow is a highly complex subject. Experiments have shown it not isotropic but occurs preferentially along channels whose geometry depends, of course, on the aperture of the discontinuity, but also on wall roughness and surface of contact between walls, applied normal and tangential stresses, and tangential joint displacements, plus of course the possible presence of infilling material.

Various more or less simplified approaches can be used to estimate the flow rate Q of a fluid circulating in a discontinuity (see appendix for details). For a planar and smooth discontinuities, the flow is generally assumed to be proportional to the cube of the discontinuty aperture (Appendix 10).

But even allowing for roughness, it must never be forgotten that measured flows often differ substantially from rates calculated by these methods.

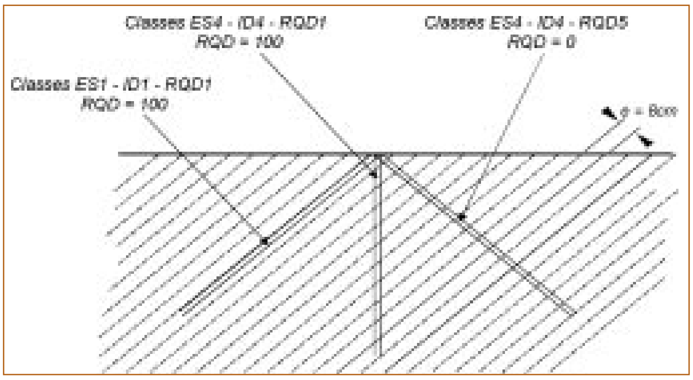


Figure 13 – Influence of direction on the characterisation of discontinuities in finely bedded formations

4 – CHARACTERISTICS OF ROCK MASS

4.1 – IDENTIFICATION PARAMETERS

4.1.1 – RQD

RQD determined from jointing (see para. 3.2.3.1) was originally considered as an index of rock mass quality determined by counting discontinuities in borehole cores.

If the discontinuities are all oriented more or less uniformly in all three dimensions ('isotropic' jointing), RQD can be taken as independent of the direction of the borehole and can effectively be considered as a overall index of the quality of the rock mass.

But if the distribution of discontinuities is strongly polarised (as in finely bedded rocks, schists, slates, etc.), the value of the RQD index will differ widely with different directions of drilling (figure 13). The RQD from a single borehole therefore will give only a 'snapshot' of the jointing in a given direction rather than a representation of the overall jointing of the rock mass. Because of this, AFTES recommends that RQD should be determined from several boreholes drilled in different directions to intersect all joint sets, especially those which may be unfavourable for the planned underground structure.

RQD, originally defined by its author on the basis of counting discontinuities found in borehole cores, can also be determined by counting them on exposed rock surfaces:

• Natural outcrops: the count proceeds along one or more lines intersecting the network of fractures such that the values obtained is truly representative of the homogeneous blocks of rock as defined elsewhere.

• Quarry faces, trench sides, adit walls: in these cases, cracking caused by the use of explosives should be ignored as far as possible.

If the rock mass displays strongly polarised discontinuities, the same reservations can be made as in the case of boreholes as to the representativeness of the discontinuities recorded and the RQD calculated with reference to the direction of the survey line.

4.1.2 – Degree of alteration

The degree of alteration of a rock mass is described by breaking it down into alteration zones for the different geological formations present. A distinction is made between weathering proper and hydrothermal alteration occurring at depth (frequently linked to contemporary or more ancient volcanism). Alteration of the rock mass as a whole is classified as the sum of the weathering of the rock matrix and of the major joints.

With weathering properly so called, descriptive terms, conforming to those recommended by ISRM (A_M = W), appear in Table 16. They apply predominantly to crystalline rocks.

4.1.3 – Rock mass continuity index ICM

Using the same procedure as described in para. 2.1.10 for the rock matrix, a rock mass continuity (or quality) index IC_M can be defined as the ratio between P wave velocity as measured over a base length L (Vp_M) and the velocity measured on a sample (Vp):

$$IC_M = Vp_M/Vp$$

This concept of a rock mass continuity index IC_M makes it possible to estimate the impact of the scale effect and deterioration in the mechanical properties the rock mass compared to results from laboratory samples (rock matrix).

AFTES CLASSES	DESCRIPTION OF THE ROCK MASS
AM1a	**Sound rock**
AM1b	**Poorly weathered rock** Weathering confined to surfaces of main discontinuities; rock sound in the mass
AM2	**Slightly weathered rock** Little weathering of rock in the mass but well developed in discontinuities
AM3	**Moderately weathered rock** Weathering clearly visible in whole rock mass but material not friable
AM4	**Well weathered rock** Severe weathering in the mass
AM5	**Completely weathered rock** Texture and large fractures still visible
AM6	**Completely decomposed rock** Texture and fractures unrecognisable Residual soil - Undisturbed

Table 16 – Rock mass weathering classes and descriptions

Whatever the absolute value of Vp, if Vp_M is equal to Vp, this means that the rock mass, at the scale L at which Vp_M was measured, displays the same properties as the sample and is unaffected by discontinuities or voids which would reduce P wave velocity.

If however, Vp_M is less than Vp, the lower velocity in the rock mass than on the laboratory samples can be attributed to discontinuities and voids in the rock mass over base length L at which Vp_M was measured.

Rock mass continuity classes, at the scale of L over which Vp_M was measured, are listed in Table 17.

Base length L is usually the same as standard seismic refraction test base lengths (60m, 120m, 240m) but rock mass continuity can be measured over shorter base lengths, based on borehole microseismics or adit wall seismic tests.

Of course, base length L must always be stated alongside the relevant IC_M value.

Note. IC_M may be greater than 100%, for example when the rock matrix contains cracks or microcracks closed tight by the confining pressure in the rock mass but which might open when coring releases these stresses. This is a not uncommon occurrence in some schistose rocks.

4.2 – MECHANICAL PARAMETERS

4.2.1 – Rock mass deformability, rock mass deformation modulus E_{Mas}

Because of the discontinuities, rock mass deformation at prototype scale is generally much greater than for the intact rock matrix as determined from small laboratory samples.

Depending on rock mass volume and the loads applied to it, deformability may be apprehended through two classes of *in situ* investigations:

• 'Indirect' geophysical methods, based primarily on wave velocities.

• 'Direct' methods consisting of measuring deformations on parts of the rock mass under changing states of stress. The changes may be brought about by specific load tests (*in situ* tests) or by construction of the tunnel (back analysis).

4.2.1.1 – Indirect (geophysical) methods

P (longitudinal compression) wave and S (transversal shear) wave travel time is measured over a known distance between the seismic source and pick-ups. Emitter and pick-ups can be arranged in various ways:

• In separate boreholes (seismic cross hole test)

• Source in a borehole and pick-up at ground level (seismic up-hole test)

• Pick-up in the borehole and source at surface (seismic downhole test).

All arrangements measure compression wave V_p and shear wave V_s velocities to derive the 'dynamic' deformation modulus and 'dynamic' Poisson's ratio of the rock mass (E_d and ν_d respectively) through the following equations:

$$E_d = \rho[V_p^2(1 + \nu_d) \times (1 - 2\nu_d)]/(1 - \nu_d)$$

$$\nu_d = [0.5 - (V_s/V_p)^2]/[1 - (V_s/V_p)^2]$$

(ρ is density, see para. 2.1.4).

Computing E_d and ν_d by geophysics means measuring both V_p and V_s.

If only the compression wave velocity V_p has been recorded, the E_d modulus can still be obtained by taking an assumed value for V_s (usually 0.25 or 0.30).

Other methods based on seismic velocities in the rock mass can yield estimates of the corresponding moduli, through experimental correlations with past construction sites (Schneider's 'Petite Sismique' method, SCARABEE method).

It must be realised that the term 'dynamic' here in fact refers to very small strains ($10^{-7} < \varepsilon < 10^{-5}$) under very small loads.

CLASS	IC_M	DESCRIPTION
IC_M 1	> 90 %	Very high continuity
IC_M 2	90 % to 75 %	High continuity
IC_M 3	75 % to 50 %	Moderate continuity
IC_M 4	50 % to 25 %	Low continuity
IC_M 5	< 25 %	Very low continuity

Table 17 – Rock mass continuity classes at L scale

4.2.1.2 – Direct measurement

The main *in situ* tests for measuring rock mass deformability are:

• **Rigid plate loading test**, quite widespread and routine. It characterises the deformability of the rock mass through deformation modulus E determined from the tangent to the envelope curve of the 'load-displacement' curves under increasing loading cycles (cf. Appendix 11).

With usual plate sizes (0.28m to 0.60m), this test yields rock mass deformability values at the scale of a few cubic metres of rock, provided the pressure is high enough to penetrate beyond the decompressed surface zone.

• **Borehole dilatometer test (French standard P 94-443)**. The instrument consists of a deformable cylindrical cell applying a controlled radial pressure to the borehole walls, and several strainmeters directly measuring the radial deformation of the borehole wall under the applied pressure.

The E modulus of thr rock mass is calculated with the following equation in which, in the absence of specific data, the Poisson's ratio n is frequently taken as 0.25:

$$\Delta\sigma_r/\Delta\varepsilon_r = E/2(1 + \nu)$$

$\Delta\varepsilon$ is the change in radial strain produced by the change in applied stress $\Delta\sigma$ on the borehole wall.

The strainmeters measuring the radial deformation at the borehole wall must display sufficient resolution to measure the usually high moduli encountered in rock. They are arranged in different directions at different places along the length of the dilatometer (3 pairs at 120° or 4 pairs at 90° in different models). This arrangement shows up any anisotropy in rock deformation.

The dilatometer can also be used for creep tests in which the applied pressure is held constant over time and time-dependent displacements are recorded.

• **Borehole pressuremeter test (standard P 94-110-1 & 2)** also measures mass deformability by means of a deformable cylindrical cell applying an increasing pressure to the borehole walls. The slope of the 'pressuremeter curve' showing the change in pressuremeter cell volume vs applied pressure is used to calculate a shear modulus G (Menard pressuremeter modulus E_M).

However, the characteristics of the instrument mean that **the pressuremeter test is unsuitable for rock mass determinations,** even if it is still widely used. The reason is that, with moduli of a few hundred MPa (a value which is abundantly exceeded in rock), the moduli from pressuremeter tests become increasingly underestimated whereas actual moduli increase. **The pressuremeter test must only be used for soils and some 'soft' materials (chalk, marl) on the borderline between soil and rock.** When dealing with the usual types of rock, the pressuremeter should not be reckoned among the panoply of relevant test methods.

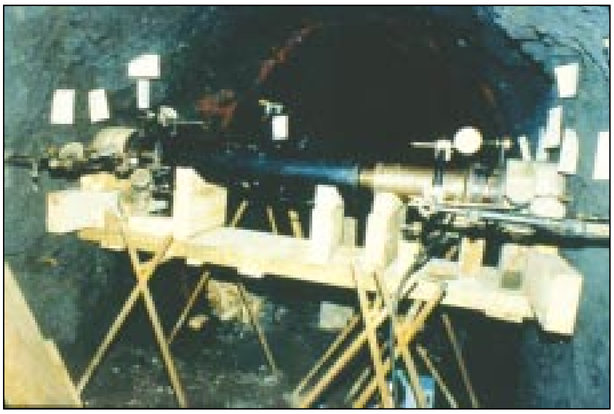


Photo 7 – Rigid plate loading test. Volcanic agglomerate, Takamaka, Reunion Is.

4.2.1.3 – Measurement on actual structures and estimation of deformability by back analysis

These are probably the most effective methods for finding the large scale deformability of a rock mass and the anisotropy parameters governing it.

On actual structures (generally exploratory adits driven prior to the full size construction stage), the measurements most commonly performed are the following:

• displacements of the adit wall (convergence)

• displacements of points within the rock (displacements relative to fixed or moving reference points) by means of borehole extensometers around the adit

• angular changes between studs fixed to the rock (inclinometers or deflectometers).

Back analysis, generally with 2D or 3D computer models (finite element and similar models) allows the engineer to work back from the known stress state to the most plausible rock moduli under the conditions of the completed structures. In the case of anisotropy in the moduli, which usually accompanies anisotropic states of stress, back analysis is more difficult.

When checking the design of very high head water pressure tunnels, the concrete lining subjected to the high water pressure is instrumented (to measure diameters and stresses). This type of 'chamber' test also provides a check on rock watertightness.

4.2.1.4 – Classification of rock mass deformability

Rock mass deformability classes based on the rock mass deformation modulus E_{Mas} are listed in Table 18.

4.2.1.5 – Time-dependent effects – long term modulus

The construction of an underground structure always causes deformations in the surrounding rock due to changes in the stress field around the opening. In many cases, rock deformations are accompanied by effects which appear over time and deformations increase asymptotically towards what is generally called the "long term state."

As discussed in para. 2.2.2, there may be several causes of time dependent rock mass behaviour:

• Rheological behaviour specific to the rock mass, viscoelastic or viscoelasticplastic behaviour (as in certain rocks such as evaporates, marls, etc.).

• The elastic limit may be exceeded with plastic zones developing around the opening.

• Time-dependent deformations may be linked to consolidation processes subsequent to changes in flow patterns, with the opening acting as a drain, or to the original pore pressure patterns gradually re-establishing after the disturbance caused by excavation.

These causes may be concomitant, and this aggravates the difficulties of correctly interpreting the observed time dependent behaviour.

At present, the most widely used simplifying approach to the understanding of time dependent rock mass behaviour is to consider the rock mass deformation modulus as a decreasing time function:

$$E_{Mas}(t) = E_{Mas0}/[1 + \Phi(t)]$$

in which

- E_{Mas0} is the instantaneous rock mass deformation modulus

- $E_{Mas}(t)$ is the rock mass deformation modulus at time t under a load that has not changed since time t = 0.

$\Phi(t)$ is a monotonous function increasing from $\Phi(0) = 0$ to $\Phi(\infty) = \alpha$.

In low- to moderate-strength rock, $\alpha = 1$ is often suggested even if it does not always appear justified by experimental and other data.

In stronger rocks, $\alpha = 0.3$ to 0.5 are also frequently proposed without any justification for the choice.

A less doubtful choice of these values would require long-term instrumental data from completed structures. At the present time, little feedback is available and records cover only a limited number of years.

CLASS	ROCK MASS DEFORMATION MODULUS E_{Mas} (GPa)	DESCRIPTION
D_M 1	> 30	Very low deformability
D_M 2	10 to 30	Low deformability
D_M 3	3 to 10	Moderate deformability
D_M 4	1 to 3	Fair deformability
D_M 5	0.1 to 1	High deformability
D_M 6	< 0.1	Extremely high deformability

Table 18 – Rock mass deformability classes

4.2.2 – Rock mass limit strength

Mechanical properties are strongly influenced by the geometrical dimensions of the volumes of rock involved, in the general sense of these properties becoming worse as the rock volume grows (scale effect). In situ rock mechanics tests to failure such as shear tests and hydraulic fracturing involve considerable effort and still only concern limited volumes of rock.

There are not really any tests for characterising the mechanical strength of a rock mass. Only an empirical approach based on feedback from past construction jobs is possible. This leads to modifying the criteria for samples (see para. 2.24) by downscaling the characteristic parameters when extrapolating from the intact rock matrix to the large scale rock mass (see 5.3.4).

4.3 – HYDROGEOLOGICAL CONDITIONS

Ground water is the cause of many difficulties encountered in underground engineering:

• Flowing water slows down excavation work

• Water pressures may destabilise the tunnel walls or lead to fearsome squeezing ground into the tunnel

• Dewatering may have severe environmental consequences: flow depletion from springs and wells, subsidence due to groundwater lowering.

Basically, flow rate Q through a section S of the rock mass is related to permeability K and hydraulic gradient i by Darcy's law:

$$Q = K.S.i$$

Flow is associated with body forces proportional to the hydraulic gradient i.

Characterisation of the hydrogeological conditions in a rock mass therefore proceeds in three steps:

1. Identify aquifers and how they function.

2. Measure hydraulic head H on the tunnel.

3. Measure rock mass permeability K_M.

In practice, these three steps do not necessarily proceed in the order described because it may not be possible to elucidate the existence of certain aquifers and their functioning until exploratory works have been undertaken. It is also important to realise that such investigations must cover the whole aquifer system affected by the tunnel, not only the part through which the tunnel passes, as is the case with investigations for mechanical parameters.

4.3.1 – Identification of aquifers

The extent of the aquifer system liable to be affected by the underground structure is determined with the aid of the geological model mentioned in section 1 above, identifying the main individual aquifers if applicable. The hydrogeological functioning of the system is then tentatively modelled, with rough estimates, for each aquifer crossed by the project, of

a) The type of permeability concerned, which can be classified under five headings:

1. Granular material (sand and gravel);

2. Jointed rock (granite, gneiss, basalt, etc.) with water circulating only through the discontinuities;

3. Double porosity ground in which water circulates both through discontinuities and the porous rock matrix (chalk, sandstone) or weathered rock matrix (severely weathered granite);

4. Karstic rock (limestone, gypsum) in which most of the water circulates through randomly distributed voids of various sizes;

5. Fault zones with breccia infill frequently acting as drains within fractured rockmasses.

b) Boundary conditions, i.e.

- sources (rainfall, infiltration, river, lake, sea, etc.)

- flow rate at point of outflow

- watertight boundaries.

This first step involves a desk search and inspection of a project survey plot; it uses data from the geological model and demands considerable engineering experience. It should not overlook local sources of information from groundwater users, amateur kart environment, etc. In karstic settings, it is important to know whether the karsts are active or fossil karsts more or less filled in.

4.3.2 – Measurement of initial piezometric conditions

Knowledge of the pre-construction piezometry in the rock mass at the project site is a critically important factor in good design. The project may be a very long tunnel and pre-construction piezometry must be determined along its whole length. It seems unnecessary to stress the risks arising from ignorance of the initial piezometric conditions (even over a short length of tunnel) both during construction and subsequent operation of the structure.

Where more than one aquifer and different pressure heads are suspected and in zones where the relief may be an influencing factor (hillsides, valley bottoms, etc.) local piezometric testing may be indicated, using pressure cells; open well piezometers should not be used.

Piezometry is usually subject to **seasonal fluctuations** and it is strongly recommended that piezometric monitoring of each aquifer identified should commence at the very earliest stage of the design process. It will often be necessary to have several years' records before being sure of the amplitude of the piezometric fluctuations to be expected and designed for. In most cases, only continuous records will show up sometimes short-lived transients which may have a serious impact on the project (karstic aquifers, tidal river reaches, etc.). The designer must also assess the risk of the water table rising, especially in urban areas, due to local abstractions being unexpectedly interrupted. Lastly, knowledge of changes in the peizometry and flow rates leads indirectly to certain aquifer hydrodynamic parameters.

Hydraulic head classes are listed in Table 19.

CLASS	INITIAL HYDRAULIC HEAD H (in metres above tunnel invert	DESCRIPTION
H 0	Lower than invert	Zero head
H 1	< 5	Low head
H 2	5 to 20	Moderate head
H 3	20 to 100	High head
H 4	> 100	Very high head

Table 19 – Hydraulic head classes

As a general rule, every cored borehole sunk to investigate a tunnel site should be fitted out as a piezometer, in view of the importance of this parameter.

4.3.3 – Measurement of rock mass permeability K_M

Determining the permeability of the rock mass calls for interpretation of the results of hydraulic tests, which must be chosen to suit the type of aquifer concerned. Available tests are as follows.

• Localised tests in boreholes:

- either steady-state tests where permeability is moderate to high, such as the standard **Lefranc** test for soils and **Lugeon** test in jointed rock (water is injected at 1 MPa pressure); the Lugeon test measures apparent permeability and supplements the jointing data obtained from cores and mechanical behaviour;

- or transient-state tests, causing instantaneous changes in the open (slug test) or closed (pulse test) test interval, or a combination of them (drill stem test); these more complex tests are suitable for low permeability conditions ($K < 10^{-7}$ m/s).

• **Larger-scale tests,** such as:

- pumping-out tests with measurement of far-field water table drawdown in suitably located boreholes;

- measurement of tunnel drainage (the tunnel may be divided up into separate lengths for this purpose).

When conducting these tests, it is important to ensure that the disturbance induced by the test is of the same order of magnitude as the disturbance that will be caused by the tunnel, so as not to disrupt the environment. In all cases, upscaling measured data to the whole aquifer calls for great caution.

Rock mass permeability classes are listed in Table 20.

CLASS	ROCK MASS PERMEABILITY K_M (m/s)	DESCRIPTION
K 1	$< 10^{-8}$	Low permeability
K 2	10-8 à 10^{-6}	Moderate permeability
K 3	10-6 à 10^{-4}	High permeability
K 4	$> 10^{-4}$	Very high permeability
K 5	Pratically infinite	Karst permeability

Table 20 – Rock mass permeability classes

Permeability in jointed rock is very often **anisotropic**: it may typically be ten times greater parallel to the bedding or cleavage than in the perpendicular direction. Anisotropy may also depend on the orientation of the principal stresses. Equivalent permeability for the whole rock mass is given by a tensor; for classification purposes, the highest permeability coefficient is used, stating the direction in which it applies; the anisotropy ratio Kmax/Kmin is also used.

When running borehole tests, it is recommended to proceed in contiguous 5m to 10m stages and plot a **permeability log** which can be usefully compared to the jointing log.

Other useful data that can be logged concerns:

• hydraulic head, by measuring pressures between packers,

• borehole inflow/outflow, by continuous measurement with a miniature flowmeter.

When dealing with deep-lying mine workings, tunnels, mined storage chambers and other deep structures, understanding the hydrogeology is complicated by the possibility of exploratory drilling causing unwanted interconnections between separate aquifers. This calls for more sophisticated techniques as used in the mining and oil industries, such as those described in the AFTES Working Group 24 Recommendations. They accurately locate aquifers through the use of fluid logs and multilayer tests, and make it possible to test them selectively with packers and probes.

Photo 8 – Water flow into Saint Guillaume II tunnel from Grange Pellorce fault, France

4.3.4 – Gas

Methane (CH_4), nitrogen (N_2), hydrogen sulphide (H_2S), carbon monoxide and dioxide (CO and CO_2), radon 220 or 222 (Rn) and other gases may be present in the free state or dissolved in the ground water within certain sedimentary formations (carbonaceous, carbonate, argillaceous and saline rocks) or igneous formations (e.g. granite). When such formations host an underground opening, the gases they contain tend to migrate towards the excavation, creating a risk of explosion, poisoning, suffocation or disease (cancer and other occupational illnesses), not only during excavation, but equally during the service life of the structure. While such risks are more the province of health and safety, they must be addressed at the tunnel and ventilation system design stage and must therefore be accorded special attention when proceeding with the geotechnical characterisation of the ground to host the structure when such gases may be present within it (coal measures for example).

4.3.5 – Other parameters

In addition to the H and K parameters for each aquifer identified, other parameters may be of use in characterising the rock mass, in particular:

• Storage coefficient S, representing the capacity of the rock mass to store water. This must be investigated for modelling transient flow conditions; it can be derived from pumping-out test data and may range from 10^{-5} (10 cm^3 of water released when the hydraulic head in a 1 m^3 volume of saturated ground is lowered by 1m) to 0.1 to 0.15 in clean sands.

• Groundwater temperature, pH and chemistry (and sometimes isotopes). These parameters serve more as indicators of where the water comes from and help understand the hydrogeological functioning of the aquifer system; and they make it possible to assess how aggressive the water will be for tunnel support and linings.

4.4 – INITIAL STATE OF STRESS IN ROCK MASS

The initial stress state is a determining factor in the response of the rock mass to excavation: the convergence pattern on a tunnel section, the location and extent of zones where the limit strength of the rock may be reached during tunnel driving, are all strongly dependent on the initial stress state, and it is vital to consider it at the design stage.

4.4.1 – Initial state of stress and approximations

Computation and modelling in the design stage make it possible to investigate and analyse the impact of the initial state of stress.

The state of stress is represented everywhere by a tensor whose principal components are the σ_1 (major), σ_2 (intermediate) and σ_3 (minor) stresses.

In the absence of data, it is commonly assumed that the vertical is the principal direction and that the vertical stress is equal to the "weight of overlying ground," i.e.

$$\sigma_v = \gamma \times z$$

These two assumptions are valid in subhorizontal sedimentary formations but are not generally valid in mountain areas where the relief and tectonics introduce considerable distortions, especially under mountainsides and at valley bottoms.

In sedimentary basins, a third assumption is frequently made, that the horizontal principal stresses σ_h and σ_H are identical and equal to a constant fraction of σ_v:

$$\sigma_h = \sigma_H = K_0 \times \sigma_v$$

This assumption **may be very far from reality** since the horizontal stresses are rarely isotropic and K_0 frequently ranges from 0.5 to 2 or more [5].

4.4.2 – Characterisation of stress tensor

When designing any underground structure, it is important to try to determine the initial state of stress. Because of the difficulties involved in this, a step-by-step procedure is usually followed, based firstly on indirect approaches and then, if possible, on in situ test data which can be checked during construction by specific observations (Table 21).

In the project planning stage when an approximate estimate of stresses is sufficient, the designer focuses on indirect analyses, using published information and conclusions that can be drawn from the geological history and local topography of the project area. This first step should postulate a stress range to be expected.

The estimated stress range and its impact on design may subsequently justify performing *in situ* field tests. If severe horizontal anisotropy is considered possible at this stage, data on the azimuth of sH may lead to the orientation of the underground opening being optimised (provided such freedom is possible, in view of the purpose of the structure).

At the detailed site investigation stage, the available methods for stress measurement are always considered, not without reason, expensive, difficult to perform and interpret and above all extrapolate. On top of this, one can never directly "measure" a state of stress; at best, one measures fluid pressures considered as equivalent to the normal component acting on a given surface, or strains caused by stress changes. Despite these difficulties, it is important to have instrumental data, especially when the presumed mean stress is of the same order of magnitude as rock strength (cf. para. 4.4.4).

Lastly, once construction has commenced, the stress assumptions derived from the field tests have to be viewed alongside the actual response of the tunnel walls and data from monitoring instruments.

If the initial state of stress is found to be a determining factor in project design and feasibility, it may be decided to undertake field testing at an earlier project planning stage.

PROJECT STAGE	OBJECTIVES	METHODS AND MEANS
Project planning	• Regional tectonic regime (compressive, strike-slip, extensional) • Orientation of major principal stresses • Local disruptions due to Quaternary geological processes (palaeorelief, glaciations, erosion, etc.) • Influence of relief on state of stress	Published data: • Stress maps • Mechanisms at regional earthquake hypocentres* • Geological and topographic maps • Geotechnical reports on existing structures • Palaeogeographic assumptions
Design	• Estimate of σ_H/σ_V ratio • Orientation of major horizontal stress σ_H • Determination of complete tensor if possible	Data from deep boreholes • Ovalisation of bore • Disking in cores Borehole stress measurements • Overcoring and borehole slotter • Hydraulic fracturing and HTPF Flat jack tests in adits
Construction	Validation of design assumptions based on exploratory works	• Observation of tunnel walls • Interpretation of strain data

**As an initial approximation, the P and T axes of focal mechanisms can be taken as s1 and s3*

Table 21 – Stress estimation methods at different project stages

4.4.3 – Commentary on field test methods

When measuring *in situ* stresses, it is strongly recommended to plan an abundant number of mutually complementary tests, not only because of metrological difficulties, but more importantly because of the often very sudden local variations in the stress tensor; these variations may be due to lithological heterogeneities or the proximity of geological discontinuities, fracture zones or even a free surface (favouring stress release). Such conditions make extrapolation of test data even more problematical and it is vital to check the data during construction work.

The various test methods available have been described in *Tunnels et Ouvrages Souterrains* No. 123 (Briglia et al. 1994). Only a few general recommendations will be given now, focusing on borehole methods.

a) Methods based on **stress release**, by overcoring, undercoring, or cutting a slot with a borehole slotter, usually in boreholes that are differently oriented; data on rock deformability is needed. The CSIRO, USBM and other instruments involved are quite difficult to use properly and are mainly suitable for poorly jointed rock, in which they yield purely local information (on a decimetric scale); they can only be used in the elastic range.

b) The **hydraulic fracturing** method measures the normal component of the stress acting on a discontinuity, by means of an elevated water pressure on a section of borehole. It has the advantage of involving a volume of rock several cubic metres in size, being feasible at great depths (in excess of 1000m) and needing no assumptions on rock behaviour. There are two variants:

• "Standard" hydraulic fracturing, creating artificial fractures perpendicular to the minor principal stress s3 to determine its magnitude and direction.

• The hydraulic test on pre-existing fractures (HTPF test) extends the scope to natural fractures with different orientations. Provided enough tests are performed, HTPF is one of the most reliable means of determining the complete stress tensor.

[5] Strictly speaking, coefficient K_0, more commonly used in soil mechanics, is the effective stress ratio

If an exploratory adit is available, stresses can be measured at different points in the adit wall by the **flat jack** method. It is only suitable in rock having few or no joints that has been substantially unaffected by excavation. The only assumption needed is that rock response will be reversible, but calculation of the complete tensor relies on a model of the rock mass around the adit. Tests must be made over several straight sections of the adit.

4.4.4 – Classification of stress states

In the project planning stage, the designer can make his first assessment of the general stability of the unsupported opening on the basis of the σ_c/σ_0 ratio, with σ_c representing the uniaxial compressive strength of the intact rock matrix (the most readily accessible parameter at this stage) and σ_0 being the value of the major stress in a plane normal to the tunnel.

On the basis of the σ_c/σ_0 ratio, one can proceed to the general qualification of the initial state of stress with respect to its consequences on the planned structure.

Table 22 lists stress state classes based on the σ_c/σ_0 ratio.

It is absolutely essential to estimate the stresses by means of field tests when the structure can be expected to lie in class CN2 or CN3 conditions (assuming $\sigma_0 = \gamma \times z$).

CLASS	σ_c/σ_0 RATIO	DESCRIPTION OF STRESS STATES
CN 1	> 4	WEAK Rock matrix satisfactorily strong but support may be needed because of jointing
CN 2	2 à 4	MODERATE Failure or plastic zones possible at tunnel walls
CN 3	< 2	STRONG Rock matrix strength manifestly insufficient

Table 22 – Preliminary classification of relative state of stress around a tunnel

4.5 – TEMPERATURE

It is only necessary to predict natural temperatures in the rock mass for some underground structures which are temperature critical during their service life (mined storage) or need special cooling systems to be provided during and after construction (deep tunnels and storage facilities).

The estimate is frequently based on a mean value of the local **geothermal gradient** Γ, which usually ranges from 2° to 4° per hundred metres depth, although it may take other values in tectonic zones and volcanic ground.

4.5.1 – Geothermal parameters

The most important thermal property of a rock mass is its **conductivity** λ (in W/m.°C). Values for this parameter for samples of different rocks are confined to the range 1.5 to 5.5 W/m.°C (cf. Appendix 12). Because of the low porosity of rock, the presence of water and air has very little impact on the thermal properties of the rock mass under **pure conduction** conditions (where there is no flowing fluid) and it is usually taken that rock conductivity at depth is close to the value measured on samples.

In contrast, the effect of anisotropy must be considered because conductivity may be up to twice as high in the direction parallel to the cleavage than in the perpendicular direction.

A rock mass is naturally supplied with heat

• from underneath, due to the **geothermal flux** Φ, which can be considered constant at the scale of civil engineering construction. There is little precise local date on this point. It usually ranges from 50 to 80 mW/m². In pure conduction, we have

$$\Phi = \Gamma \times \lambda$$

• from the surface: heat from sunlight varies with time of day and season but such changes do not affect rock temperature beyond about twenty metres' depth; for temperature modelling purposes, sunlight governs surface temperature (on which abundant meteorological data is available).

Other very localised sources of heat may be chemical reactions, radioactivity, hot water from deep underground, or cold water from the surface.

4.5.2 – Methods for estimating temperatures for underground structures

At *the project planning stage*, the designer is usually confined to a desk search for regional geothermal data (geothermal flux map of France, Mechler 1982; isogradient maps, Gable 1980) and temperature data from oil wells and geothermal boreholes. Geothermal springs should be identified where they exist, along with nearby volcanic ground ("hot" anomalies), and the possibility of there being aquifers near the underground structure site supplied from surface water ("cold" anomalies).

At *the design stage*, a more detailed search is made for anomalies by systematically measuring water temperature in exploratory boreholes (temperature-conductivity logs).

During construction, temperature measurements as tunnelling proceeds allow the parameters determined in the previous phase to be checked and validated. Any thermal anomalies detected in boreholes drilled ahead of the face are excellent warning signs that the drive is nearing zones of flowing underground water and geological discontinuities.

With very large projects, temperature modelling with thermal models or suitably modified hydraulic models (Goy 1996) is possible after precise determination of surface temperature, structural geology, rock conductivities and local geothermal flux (pure conduction models are valid if there is no significant water flow, Fabre 2001).

5 – USE OF ROCK MASS CHARACTERISATION FOR UNDERGROUND STRUCTURE STABILITY ANALYSIS AND CONSTRUCTION

5.1 – CHARACTERISTIC VALUES AND PARAMETERS FOR DESIGN

5.1.1 – Individualisation of sub-units

As mentioned in para. 1.2.2, the first step is to divide the rock mass up into sub-units exhibiting substantial uniformity in their characterisation parameters. A sub-unit may correspond to a section of the tunnel (for linear structures) or part of the underground structure (for non-linear structures).

Individualisation of a sub-unit generally starts with the lithology and geological structure as seen on the tentative geological cross section; however, the lithology may admit of sub-divisions by reason of variations in

- the mechanical properties of the material (with the same petrographic signature),
- alteration and weathering,
- hydrogeology,
- rock cover,
- joint density, etc.

In practice, this operation of dividing the underground structure up into uniform sub-units usually involves several iterations before the resulting breakdown emerges as entirely logical and coherent. As it is not possible to set boundaries between classes of values for the different parameters used as the most pertinent criteria for individualisation, a trial and error process is generally necessary.

5.1.2 – Geotechnical characterisation of sub-units

Consider the measurements and tests for a given sub-unit.

Once site investigations have been made, the raw data consists of measured values from tests performed by the geotechnical operator.

Depending on the geotechnical terms of reference (cf. French standard P 94-500), the next step is for the geotechnical operator and/or designer to proceed with a critical analysis of these **measured values**. This critical analysis eliminates any anomalous data and validates the **significant values** to be retained.

The **significant values** relate to a type of test and a in-situ test:

- They may be 'local' values associated with a type of test or measurement at a given sampling or in-situ test;
- They may on the contrary be 'global' values, associated with a type of test or measurement, on a wider geographic area (such as geophysical test data for example).

It may be useful to make a sort of 'consolidation' of the reliability of the significant values by using simple but proven correlations such as E/Vp, σ_c/Vp, σ_c/γ_d, σ_c/σ_{tb}, etc. to detect any anomalies.

Lastly, a quick statistical analysis should be run if the number of data points available is at least of the order of 10. This analysis, with number of data points, maximum and minimum values, mean and standard deviation, makes it possible to assess scatter and the possible existence of several homogeneous populations, as may happen with an anisotropic rock or a material consisting of thin alternating layers of different lithologies, e.g. marl-limestone.

For each uniform sub-unit, these manipulations can be used to determine a '**characteristic**' value for each parameter. The **characteristic value** of a parameter represents a reasonably cautious value, not a maximum or minimum (cf. AFTES GT 29 Recommendations on the use of general design standards and rules for reinforced and plain concrete tunnel linings).

For any one parameter, it may be useful to set one or more characteristic values according to the intended use. For example, a low characteristic value for the uniaxial compressive strength would be needed for the stability analysis and a higher value for the drillability assessment.

5.1.2.1 – Rock matrix

The description for non quantitative parameters (common name, petrography and mineralogy, alteration of the rock matrix material) must be precise and, where necessary, supplemented with quantitative values (mineral contents, methylene blue test value, etc.).

For most physical identification parameters (density ρ, ρ_d, ρ_s or volume weight γ, γ_d, γ_s, moisture content w, porosity n, wave velocity Vp and continuity index IC) and for hardness DU and abrasiveness A_{IN} or A_{BR}, the arithmetic mean of the values obtained is the starting choice for the characteristic value, provided that the statistical analysis confirms that the population is normal.

The choice is not valid in the following cases:

- Discovery of more than one population of values (marl limestone, schists, etc.)
- Variation linked to the spatial distribution of samples
- Severe scatter militating for caution in favour of a lower characteristic value than the mean minus standard deviation.

With parameters for mechanical behaviour, a distinction must be made between the following:

- Young's modulus E and Poisson's ratio n which admit of the same approach as for the physical identification parameters,
- Swelling parameters σ_g and C_g, for which the pair of characteristic values must lie at the upper end of the measured values,
- Uniaxial compressive strength σ_c, tensile strength stb and Franklin index Is, for which the characteristic value may be

- either the 5% fractile when focusing on the stability of the structure,

- or the 95% fractile when dealing with drillability.

When dealing with stability, caution dictates giving consideration to the case where rock strength might locally be lower, whereas with drillability, attention focuses on the higher values of σ_c in order that tunnel driving aspects are not underestimated. In all situations, a reasonably prudent choice of characteristic value must be the outcome of a thinking process that must be explicit and substantiated on the basis for example of a statistical analysis, site specifics, reference to completed structures in similar settings, etc.

- Cohesion C and friction angle ϕ, for which the analysis preceding determination of the pair of characteristic values C and ϕ must include for all the circles determined on the same material, with no distinction between one test and another.

5.1.2.2 – Discontinuities

Before determining characteristic values qualifying discontinuities, the designer must address the following points:

- Joint density may itself be an individualisation criterion for the sub-unit, and one must always ask whether it is better to take a mean value or consider two sub-units instead of only one.
- Values found for the overall indexes (RQD, ID, FD) may be closely dependent on the orientation of the survey line.

When determining a characteristic value to quantify discontinuities, the arithmetic mean of overall index values (RQD, ID, FD) measured in the same direction is satisfactory provided scatter is reasonable. The most appropriate survey line direction must be chosen with reference to tunnel alignment and this is the direction along which the characteristic value is determined.

For parameters describing discontinuities in the same joint set, a distinction may be made between

- strike OR, whose characteristic value can be taken as the most frequently occurring data point on the Schmidt density chart,

• joint spacing ES, whose characteristic value can be taken as the arithmetic mean in simple cases displaying little scatter; in other cases, the consequences of variable spacings must be examined in detail,

• joint persistence, whose value can be approximated by observations on outcrops but whose characteristic value can only be the result of engineering judgement and experience,

• joint wall roughness, waviness and weathering, joint width, infilling and water if any, for which, once again, engineering judgement only can arrive at an average characterisation for the whole joint set.

5.1.2.3 – Rock mass

The rock mass may be characterised either directly from the results of appropriate *in situ* tests, or indirectly with the aid of empirical classifications and correlations, relying mainly on the characteristic values determined from laboratory samples of the intact rock material as well as all other sources of data (geophysics, borehole tests and measurements, etc.).

Because of the time and cost involved, *in situ* tests are usually kept for a relatively late stage in the design process once the project layout has been more or less finalised. In the earlier stages, indirect methods as discussed in para. 5.2 – Geotechnical Classifications and 5.3 - Correlations are mostly used.

in situ tests proper in boreholes, shafts and adits aim primarily at determining rock mass deformability, *in situ* state of stress and hydrogeological conditions. Knowledge of the rock mass will be more or less extensive and precise, depending on the resources assigned and the size of the project. There can be no doubt that an **exploratory adit** driven over part or all of the alignment of the permanent structure will yield more, and more precise, data than many boreholes and shafts. The designer will also have at his disposal several different ways of arriving directly or indirectly at any given parameter and comparing the various values obtained, assessing scatter and scale effect.

For example, the rock mass **deformation modulus** can be approached in various ways:

• borehole dilatometer tests

• plate loading tests on adit or shaft walls

• dynamic moduli derived from wave velocities obtained by seismic methods

• analysis of displacements measured in the exploratory adit.

Determination of the characteristic value then becomes a "reasoned" choice among different significant values from the various tests and measurements. The choice must however take into account

• scale effect, illustrated by variations in the field modulus with the rock volume considered, and by the empirical rule

$$E_L > E_D > E_P > E_G$$

in which E_L is the laboratory modulus, E_D is the dilatometer value, E_P is the plate loading test value, and E_G is derived from adit wall displacements;

• strain range: compared to other modulus values and means of measuring them, the dynamic modulus is based on very small strains.

A mean permeability value can only be estimated in so far as local values exhibit little scatter, to ensure there is a good probability of their belonging to the same sub-unit. If this is the case, the mean is calculated on the log K values (log normal distribution). If there is significant scatter in the data or significant differences between two or more test results, the designer should ask whether it would not be more appropriate to consider several units with their own specific hydrogeological characters.

By the same logic, where several *in situ* stress measurements are available, it might be preferable to select the stress state that agrees best with drilling records (disking, ovalisation, wall failure).

Generally speaking, the approach to obtaining characteristic values is to find and analyse the greatest number of cross-checks between data from different sources in order to arrive at a considered judgement as to the correct characteristic value.

In the absence of any direct means of determination, when significant values have not yet become available (as is often the case with rock mass deformability and nearly always with rock mass limit strength), a possible approach is to refer to a similar, completed, structure and/or rely on classification systems, always provided that the project falls within their category of validity.

5.2 – GEOTECHNICAL CLASSIFICATIONS

5.2.1 – General

Various authors have proposed classification systems such as Protodiakonov 1909, Terzaghi 1946, Lauffer 1958, Deere 1964, Wicham 1972, Bieniawski 1973, and Barton, Lien & Lude 1974. The Bieniawski and Barton systems are by far the most widely used.

Geotechnical classification systems are based on an empirical rock mass "quality score" drawn from values determined for certain design-critical parameters. The parameters involved vary slightly from one system to another but are basically

• rock matrix strength

• joint density

• mechanical behaviour of discontinuities

• hydrogeological conditions

• state of stress (partially).

The scoring process produces a final value obtained through a simple calculation, which also differs from one system to another.

In 1978, at the time of first writing these Recommendations on the description of rock masses, AFTES adopted a restrictive position towards these systems, arguing that quantifying rock mass quality by means of a single score was too reductionist and did not reflect the complexity of the real world.

Now these systems are widely used to derive, via the various correlations proposed, mechanical parameters for rock masses (modulus, Hoek & Brown coefficients, etc.) which can be used as design input (see para. 5.3.1). It is nevertheless extremely important to remain sceptical about the simplifying assumptions inherent in these systems and the choice of data on which they are based.

Using a classification system for any particular project presupposes that the designer has first assured himself that the project is truly compatible with the system used (cf. para. 5.2.4).

Furthermore, a classification system must never be considered as a substitute for site investigations or be an excuse for cutting down on efforts to arrive at the geotechnical characterisation of the rock mass. None of these classification systems are universally applicable.

5.2.2 – Bieniawski's Rock Mass Rating

The Rock Mass Rating (RMR) has been developed by Bieniawski since 1973 to provide a quantitative estimate of the properties of the rock mass and support necessary for stability. This approach was initially based on records from more than 300 tunnels, most of them lying at moderate depth in sedimentary rock. The database was based primarily on South African experience but has since been considerably enriched from many examples round the world. After the first version had been widely circulated in 1976, Bieniawski made many changes to the parameters for estimating RMR. The current version, described here, is the RMR_{89} (Bieniawski 1989).

The RMR index is the sum of five scores quantifying five characteristic rock mass parameters and an adjustment factor dependent on azimuth and dip of the discontinuities. The RMR has been calculated to span the range 0-100.

The five ratings A1 to A5 and rating adjustment B (cf. Appendix 13) are defined as follows:

- **A1:** Strength of rock matrix: Range of values 0 to 15 based on uniaxial compressive strength or point load strength Is.

- **A2:** Drill core quality: Range of values 3 to 20 for rock core quality, from RQD.

- **A3:** Spacing of discontinuities: Range of values 5 to 20 (lowest ratings for each joint set).

- **A4:** Condition of discontinuities: Range of values 0 to 30 (joint persistence, width (separation), roughness, infill (gouge) and wall rock weathering).

- **A5:** Groundwater: Range of values 0 to 15 (inflow rate and/or pressure).

- **B:** Adjustment for joint orientation: Range of values –12 to 0, for strike and dip of discontinuities with respect to tunnel alignment.

The basic Rock Mass Rating (RMR_{basic}) characterising the rock mass is simply the sum of ratings A1 to A5 (B = 0). In underground engineering work, the standard RMR (or RMR_{89}) is written as

$$RMR_{89} = A1 + A2 + A3 + A4 + A5 + B$$

Basically therefore, RMR is a rating assigned to the rock mass ranging from 0 to 100, more than 70% depending on discontinuities and only 15% on rock matrix properties and 15% on hydrogeology. The rating completely ignores the state of stress in the rock mass at the tunnel site.

This should theoretically limit the use of the RMR only to strong rock whose response is governed by the discontinuities. This would automatically exclude rock strength classes RC6 to RC7 and stress class CN3.

The rating system produces five rock mass classes (Appendix 13) and five corresponding support classes, and this is nowadays inadequate to cover the variety and progress encountered in excavation and support techniques.

5.2.3 – Barton's Q index

The Q index is the central parameter in a system developed by the Norwegian Technical Institute in 1974 based on data from more than 200 completed tunnels, mostly situated in the crystalline Scandinavian Shield with high horizontal stresses (Barton et al. 1974). The system was revised in 1993 to include data from more than 1000 tunnel case histories (Grimstad & Barton 1993).

The Q system method provides a quantitative estimate of support needed for tunnel stability on the basis of the following information:

- Largest dimension (diameter) of the planned opening
- Planned use of the completed structure (implicitly, acceptable level of risk)
- Rock mass Q index.

The Q index is a total score from 0.001 to 1000 (this is the theoretical range, reduced in most practical cases to 0.005-50), calculated from (Appendix 14)

- Rock Quality Designation (Deere 1964)
- Joint set number Jn
- Joint roughness number Jr (concerns the most unfavourable discontinuities)
- Joint alteration number Ja (concerns the most weathered discontinuities and infill material)
- Joint water reduction factor Jw (flow rate and pressure)
- Stress reduction factor SRF.

The Barton Q index is written

$$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF)$$

In other words, the Q index is the product of three factors for

- the potential size of rock blocks
- the geomechanical quality of the contact surfaces between blocks
- the initial state of the rock mass as regards water and stresses (Barton's "active stress").

Calculating the range of variation of Q, first with the most unfavourable values, then with the most favourable values, may produce very large differences if the calculations are done for sub-units displaying very different characteristics.

Table 23 recapitulates the ranges of variation of the different parameters to assess their relative weight in the final Q index value.

The weight of the SRF factor in the third term Jw/SRF is particularly high, which is the unique feature of the Q index, which refers to :

- the possibility of sheared, brecciated or very clayey zones;
- the level of stress in brittle rocks;
- potential creep and swelling stresses in deformable rocks.

The Q index is thus strongly dependent on non-intrinsic rock properties, especially the state of stress in the rock mass. The formulation of the Q index does however have the drawback of not directly reflecting the characteristic parameter of the mechanical strength of the rock material.

5.2.4 – Summary and precautions

The growing popularity of classification systems in France is probably due to :

- their apparent simplicity of use;
- their very widespread use outside France, especially by French engineers working abroad;

PARAMETERS	MOST UNFAVOURABLE CONDITIONS	MOST FAVOURABLE CONDITIONS	RANGE (highest ratio)
RQD	10	100	10
Jn	20	0,5	40
Jr	0,5	4	8
Ja	20	0,75	27
Jw	0,05	1	20
SRF	20 (6)	0,5	40

Table 23 – Ranges of variation of parameters used in calculating the Barton Q index

[6]

PARAMETERS	RMR	Q system
Overall characterisation of rock mass	• Jointing patterns well described except for anisotropic rock (schist, slate, etc.)	• Mechanical properties of discontinuities well described • Natural stresses described
Assessment of mechanical characteristics at scale of whole rock mass	• Empirical correlations between RMR and deformability and strength parameters	• Empirical correlations between Q and physical and mechanical parameters (P longitudinal wave velocities, deformability)
	• Must be used with great caution, especially for strength parameters: avoid correlations 'in cascade' of the type Q ⇒ RMR ⇒ (m, s) ⇒ (C, φ)	
Use for project	• Allowance for orientation of discontinuities with respect to axis structure • Quick means of setting length of pull • Stand-up time (conservative approach) • No use in deciding excavation method	• Not relevant to orientation of discontinuities with respect to centreline • Quick means of stipulating support needed at roof, sidewalls and intersections but gives false impression of accuracy in setting bolt lengths • Use in design stage and for monitoring tunnel driving • Allows for changes in support techniques

Table 24 – Comparison between RMR and Q system in underground engineering applications

• the convenience of using a rating system for making comparisons between design predictions and actual conditions encountered during construction at different sites;

• the possibility of amending scores in the light of conditions encountered during construction;

• the possibility of using correlations to find the quantitative data need for design analyses.

In underground engineering, the ultimate purpose of these classification systems is to design tunnel support; this approach has been tried and found satisfactory in much drill and blast tunnelling. But these systems are not always suitable with other excavation methods (road headers, tunnel boring machines).

Generally speaking, the RMR and Q systems are unsuitable for soft rock (R6 to R7). Table 24 summarises the features and limitations of these two systems.

In addition to the general and specific limitations discussed above and in Table 24, it must also be stressed that classification systems must be used with the following precautions:

• Do not use only one system.

• Explain in detail how the scores were calculated; most importantly, identify the joint sets considered at each step.

• Examine the sensitivity of the RMR or Q index to changes in the values of the parameters and present results as envelope values for the final rating.

• Do not use the ratings as a "rule-of-thumb recipe," but be critical and vigilant as to the proper field of application.

• Remember that classification systems are empirical and reflect certain tunnelling and support practices, and these practices may change.

5.3 – CORRELATIONS

Warning: It must never be forgotten that treating a jointed rock mass as a continuum material is in itself a considerable simplification. Secondly, highly anisotropic rock masses display special behaviour not covered by the usual classification systems. Correlations "in cascade" must never be used.

5.3.1 – General

In view of the difficulties of making direct tests of deformability and (even more so) limit strength at rock mass scale, many authors have sought to start from actual case histories to establish empirical relationships linking these parameters to rock matrix characteristics and rock mass jointing.

These relationships have been established for particular contexts and must be used with great caution; they must always, as far as possible, by set side-by-side with *in situ* field test results.

5.3.2 – Estimating rock mass deformability

The rock mass deformation modulus E_{Mas} is one of the critical parameters for modelling stresses and strains around an underground opening. There are several means of measuring this parameter at a volume scale of up to a few cubic metres (see para. 4.2.1.2) and estimating it at a larger scale (but for very small load values) by seismic tests (para. 4.2.1.1). As already stated in para. 5.1.2.3, the scale effect is very important here.

Many schemes have been proposed for indirectly estimating the rock mass deformation modulus. The more important ones are tabulated in Appendix 15, along with author references.

These schemes may directly combine parameters for the rock matrix (E, σ_c) and rock mass (RQD) or be derived indirectly via the RMR and/or Q index.

Figure 14 shows a few examples of the empirical relationship between E moduli and RMR and Q index.

5.3.3 – Hoek's GSI index

The Geological Strength Index is not directly a classification system, it is an intermediate step to determining the mechanical properties of a rock mass, using the empirical formulae proposed by Hoek & Brown (see below).

GSI was introduced in 1995 by Hoek (Strength of Rock and Rock Masses, ISRM News Journal, 1994, vol. 2). It derives from variants of the RMR and/or Q index, designated **RMR**' and **Q'** respectively.

[6] Much higher values, up to 400, have been suggested by Barton for very deep underground openings where there is a risk of sudden violent decompression

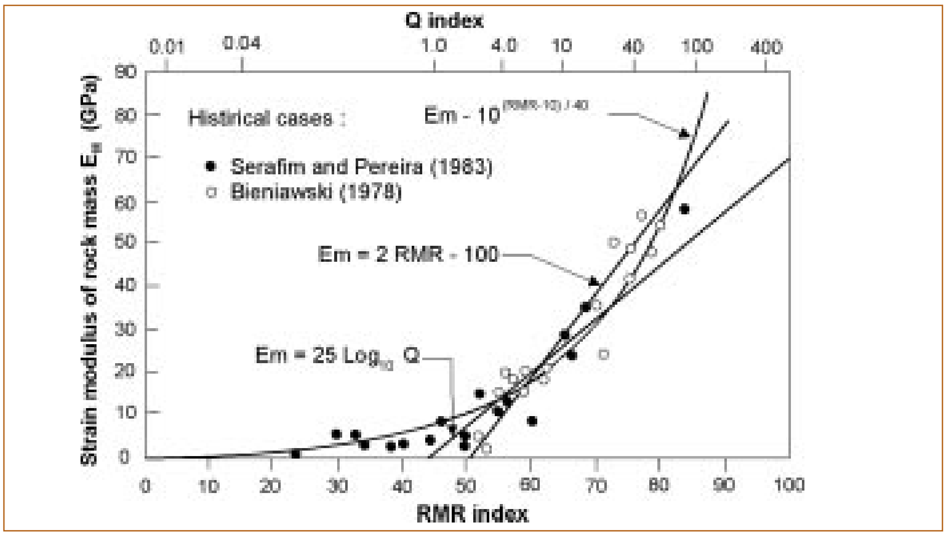


Figure 14 – Estimating rock mass deformation modulus E_{Mas} from RMR and Q values (Hoek, Kaiser & Bawden 1997)

RMR' is calculated like the basic RMR but using only the first four criteria (Strength, RQD, Spacing of Discontinuities and Condition of Discontinuities), systematically taking the fifth groundwater value as 15 (it is rock behaviour under "completely dry" conditions that is considered) and the rating adjustment for joint orientation as 0.

By a similar process, Q' ignores the third ratio for the fifth and sixth parameters (water and 'active stress').

Q' = (RQD/Jn) x (Jr/Ja)

GSI is determined from RMR'_{89} values as follows:

- for $RMR'_{89} > 23$

  $GSI = RMR'_{89} - 5$

- for $RMR'_{89} < 23$

  GSI = 9(Log Q' + 44).

5.3.4 – Estimating rock mass limit strength

Estimating rock mass limit strength at underground structurescale calls for fine judgement. As mentioned above (para. 4.2.2) no *in situ* test – except a full size test to failure, which is unfeasible for obvious reasons – is capable of yielding useable results. The only possible approach is to downscale, on empirical evidence, the properties of the intact rock matrix with reference to rock mass jointing. The most significant research in this area is due to Hoek and Brown (brought together in Hoek, Kaiser & Bawden 1977) who suggest extending the parabolic failure criterion proposed for the rock matrix (para. 2.2.5.2) to the rock mass, suitably modified to the following generalised form:

$$\sigma_1 = \sigma_3 + \sigma_{ci} [m_b . \sigma_3/\sigma_{ci} + s]^a \qquad (1)$$

in which a, s and m_b are characteristic constants for the rock mass.

Except in highly weathered rock with practically no remaining cohesion, the value generally adopted for a is

a = 1/2 (parabolic criterion).

Values for the constants can be derived from equations containing Hoek's GSI (see para. 5.3.3).

$$m_b = m_i^{e[(GSI \times 100)/28]}$$

For GSI > 25

$$s = e^{[(GSI \times 100)/9]}$$

$$a = 0.5$$

For GSI < 25

$$s = 0$$

$$a = 0.65 \times (GSI/200)$$

The relationship between mb and mi, the rock matrix characteristic constant close to the brittleness index FR (see para. 2.2.5.2) is very important for calculating numerical values in equation (1). From Hoek & Brown's compilation, the ratio m_b/m_i may range from low values (<0.1) for jointed low-friction rock masses to 0.4 to 0.6 for hard rock containing few, very rough-walled joints.

There are also formulae proposed by various authors (Hoek 1990) to derive friction angle and cohesion values from coefficients s and m_b. They are obtained by linearising the parabolic criterion and replacing it by a tangent or secant over a set stress range. Despite the attractions this may appear to offer, making it possible to extend standard soil mechanics calculations based on the linear Mohr-Coulomb criterion to rock masses[7] , it must be stressed that this criterion is generally irrelevant to the characterisation of rock mass behaviour. It may produce quite suspect results and must always be used with great caution and scepticism.

Lastly, this approach must never be used when dealing with rock masses whose discontinuities are strongly polarised (thinly bedded rock, schist, slate, etc.): assigning an isotropic failure criterion derived from the RMR to a situation where actual behaviour is strongly anisotropic can only lead to results that have little relationship with reality.

Anisotropic heterogeneous rock mass – Mequinenza region, Spain

Photo 9 – Anisotropic heterogeneous rock mass – Mequinenza region, Spain

[7] The success that this has had with many sometimes unsuspecting users is due to the ease it offers them when having deal with rock mass problems. But such popularity cannot in any way be advanced as an argument for the scientific merits of the approach.

5.4 – PRESENTATION OF ROCK MASS CHARACTERISATION DATA

5.4.1 – Basics and general remarks

The variety of geologies and geotechnical conditions which may be encountered in underground engineering and the specific features of each project make it difficult to envisage any single format for presenting synthetic data suitable for all possible cases. The general presentational scheme suggested here will have to be amended as required for each individual project.

Data presentation may differ at different stages of project development (project planning, preliminary design, final design, etc.) and according to the quantity and reliability of the data available.

When summarising data characterising the rock mass, one must bear in mind the ultimate destination of the data (general design, construction method, design analyses, etc.) and the fact that some (or all) items may have contractual relevance, depending on the stage reached in the project implementation process: the form and content of the summary presentation must consider this point.

5.4.2 – Example of data presentation in tabular form

In order to remain consistent with the hierarchy of the classes for the various parameters examined in these Recommendations, the homogeneous sub-units determined from the field investigations and studies should be ordered into geotechnical classes from E1, representing the globally highest geotechnical classes, to EN, representing the poorest geotechnical properties (see figure 15). It must be stated whether the values for these parameters are significant values or characteristic values (cf. para. 5.1.2 above).

Naturally, this is a recommendation which one should try to follow, sometimes with a little difficulty in that certain parameters examined may act in contrary directions, making it more problematical to arrive at a perfect hierarchy.

The parameters listed in the example summary Table 25 must be considered as the 'basic minimum' to be collected for any underground project. Other parameters may be added as required for any specific project (e.g. swelling pressure σ_g and swelling index Cg for potentially swelling rocks, etc.).

For each homogeneous sub-unit, the summary table may include the AFTES classes for rock matrix, discontinuities and rock mass parameters. One should not however be tempted to have so many items as to make what is a 'summary' table difficult to read, because the point of a summary is to be clear and easily understood.

NOTE. Figures shown in the E4 column of Table 25 and details in the Orientation line for discontinuities are not taken from an actual project and are shown simply for illustrative purposes.

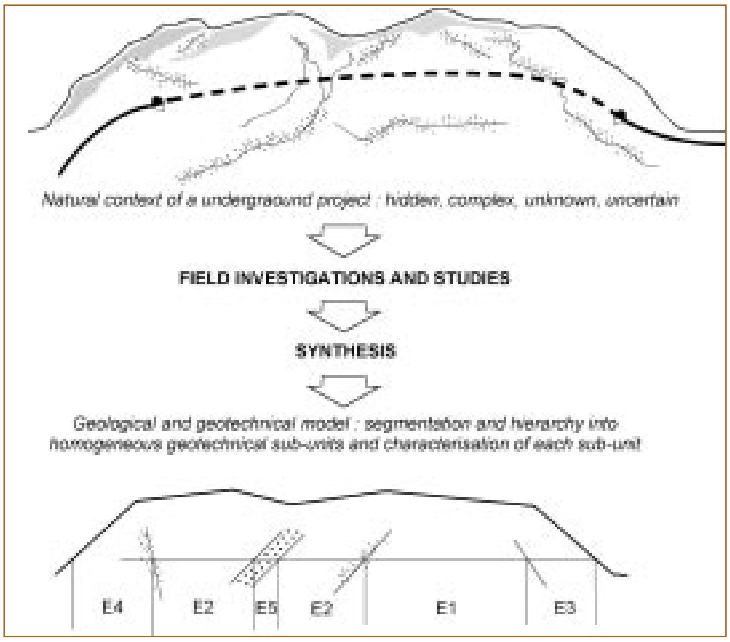


Figure 15
Principle of segmentation into homogeneous geotechnical sub-units and data hierarchy

5.4.3 – Synoptic presentation of rock mass characterisation data and cross-referencing to geological profile

5.4.3.1 – Longitudinal profile

As an aid for using the parameters characterising the homogeneous geotechnical sub-units for the various uses to be made of the data (project design, construction methods, etc.), the summary table is usually shown alongside the geological profile which, for any given project, is the basic document bridging the gap between geologists and geotechnical engineers, and civil engineers. The most common method is to have a single drawing with the geological profile at the top and the breakdown into sub-units below, along with the summary table. **The geological profile should be drawn to true scale** so as not to give an inaccurate picture of the structures with distorted contacts and dips.

5.4.3.2 – Cross sections and horizontal section at project depth

The complexity of the geological and geotechnical context may make it extremely useful or essential to accompany the geological longitudinal profile in some places with cross sections. This is particularly the case when structural details of significant importance for the project are strongly skewed in relation to the tunnel. Again, under some conditions when the geological longitudinal profile alone is unable to give a true picture, a horizontal geological profile at project depth should be added for a better understanding of the geological context and potential risks: this situation arises for example when the tunnel alignment is expected to draw near to, without necessarily intercepting, a major structural features (fault, unconformable contact, permeability interface, etc.) running near and parallel to the tunnel.

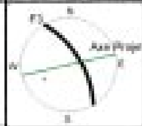
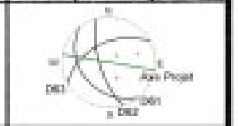
Ensembles géotechniques homogènes →			E4	E2	E5	E2	E1	E3
Paramètres ↓			PM 0 à PM X1	PM X1 à PM X2	PM X2 à PM X3	PM X3 à PM X4	PM X4 à PM X5	PM X5 à PM X6
Matrice	Identification	Dénomination usuelle	Schistes siliceux					
		Pétrographie minéralogie	26%SiO$_2$, 9%X$_s$Y$_m$, etc					
	Comportement mécanique	Résistance en compression uniaxiale σ_c (MPa)	moyenne σ_c = 46 s = 34, n = 14					
		Résistance à la traction σ_{tb} ou à la compression entre pointes Is (MPa)	moyenne σ_{tb} = 3,7 s = 1,3 , n = 11					
		Dureté Abrasivité	15 < DU < 38 1,6 < A_{IN} < 2,8					
Discontinuités	Identification	Orientation (vecteur pendage)	D11(schistosité) : N210°-62°	D21 : N37°-63° D22 : N122°-85° D23 : N243°-51°	F3 (Faille) : N239°-62°	D41 : N05°-42° D42 : N95°-72°	D51 : N33°-52°	D61 : N51° -27° D62 : N73°-67° D63 : N152° -43°
		Stéréogrammes (projection hémisphère supérieur)						
		Espacement moyen ES (cm) , Densité ID(cm)	6					
		RQD %	25 à 30					
Massif	Identification	Vitesse sismique / Vitesse microsismique	2500 à 3200 m/s / 3600 à 4200 m/s					
		Indice de continuité IC_M %	65 % à 75 % IC_M 3					
	Comportement mécanique	Module de déformation E_M (GPa)	1,3 à 4,5 D_M4 - D_M3					
	Conditions Hydrogéologiques	Etat piézométrique initial **H** (m)	25 à 35 H 3					
		Perméabilité K_M (m/s)	5.10^{-7} à 2.10^{-6} K 2 - K 3					
	Etat de contrainte initial	σ_h/σ_v	6 à 10 CN 1					
	Autres classes AFTES	Altération du massif A_M	A_M1b Faiblement altéré					
		....	....					
	Classifications	**RMR (Bieniawski)**	35 à 40					
		Indice Q (Barton)	0,5 à 1					

Tableau 25 : Tableau de synthèse des paramètres de caractérisation des sous-ensembles géotechniques homogènes

APPENDIX

CONTENTS

APPENDIX 1

RECOMMENDED ROCK DESIGNATIONS AND PRINCIPAL GROUPS

IGNEOUS ROCKS	Granites	Granite, granodiorite, syenite, microgranite, *rhyolite, rhyodacite, trachyte, etc.*
	Diorites	Diorite, quartzitic diorite, microdiorite, andesite, dacite, trachyandesite, etc.
	Basalts & Gabbros	Gabbro, dolerite, peridotite, serpentine, *basalt, pozzolana,* etc.
METAMORPHIC ROCKS	Massive metamorphic rocks	Gneiss, amphibolite, cornelian, quatzite, marble, leptynite, etc
	Schistose metamorphic rocks	Schist, micaschist, slate, calcschist, etc.
SEDIMENTARY ROCKS	Carbonate rocks	Limestone, chalk, dolomite, cargneule, travertine, marl, etc.
	Detrital rocks	Sandstone, arkose, claystone, pelite, conglomerate, etc.
	Saline rocks	Rock salt, gypsum, anhydrite, potash, etc
	Carbonaceous rocks	Coal, lignite, etc.

Names in italics are extrusive or volcanic equivalents

APPENDIX 2

MASSE VOLUMIQUE ET VITESSE THEORIQUE VP DES ONDES P DANS LES MINERAUX

Minerals	Density ρ_s (g/cm^3)	Vp (m/s)*
Amphiboles	2.98 - 3.20	7 200
Augite	3.2 - 3.4	7 200
Biotite	2.90	5 130
Calcite	2.71	6 660
Dolomite	2.87	7 900
Magnetite	5.17 - 5.18	7 410
Muscovite	2.83	5 810
Oligoclase	2.64 - 2.67	6 260
Olivine	3.25 - 3.40	8 400
Orthose	2.57	5 690
Quartz	2.65	6 050

**from French standard P 18-556*

APPENDIX 3

ORDER OF MAGNITUDE THEORETICAL P WAVE VELOCITIES VP* INSOME ROCKS ASSUMED TO BE SOUND AND NON POROUS

Rock	Vp* (m/s)
Granites and rhyolites	6 000
Diorites	6 500
Gneiss	6 000
Amphibolites	6 500
Calcaires	6 500
Silica rocks	6 000

** A utiliser au besoin dans la détermination de l'indice de continuité Ic*

APPENDIX 4

SWELLING TESTS

4.1 – Huder-Amberg test

4.1.1 – Test procedure

A – First step: Reconfinement of sample

The sample of height h is accurately dressed to fit snugly in the test cell. It is placed in the oedometer between porous stones. The top stone is in contact with the piston applying a pressure opposing all (or part of) the increase in sample height Δh. The test starts with only the weight of the piston exerting a very low stress σ_m of the order of 0.025 MPa, considered as the origin on the semi-log paper on which the test is plotted.

Any imperfections (decompression, microcracking) from the sampling or sample preparation process are corrected for by:

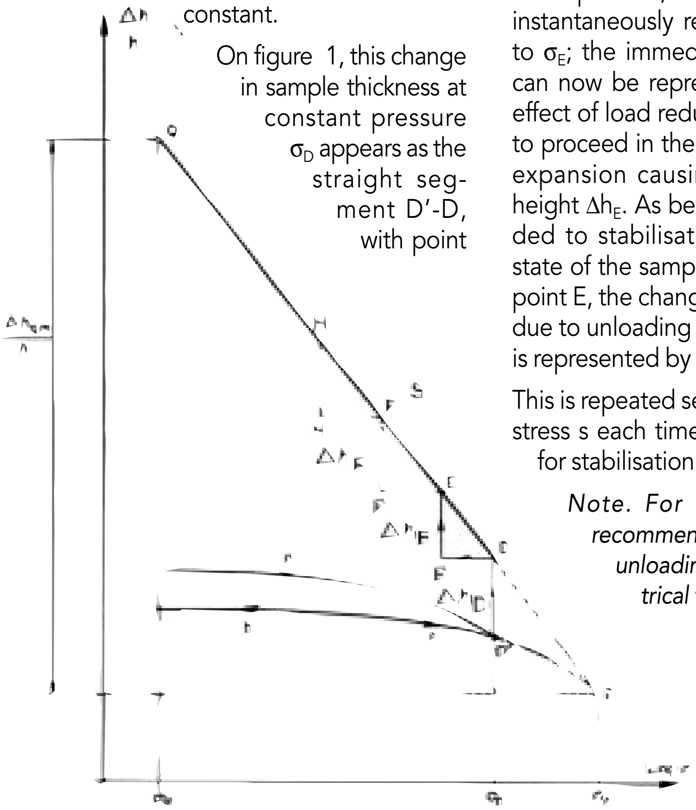
- applying load (a) to produce a stress σ_D equivalent to the in situ stress;
- then decreasing this load (b) to stress σ_m;
- increasing the load (c) to σ_D (if the sample was unaffected by sampling, the points for the two σ_D load conditions coincide).

B - Second step: Wetting

The sample is them brought into contact with water through the bottom porous stone, with the top porous stone allowing air in the sample to escape.

Wetting causes the material to expand, causing by an increase in sample height Δh under stress σ_D. The change in height Δh_D is recorded over time until there is no further change and Δh_D remains constant.

On figure 1, this change in sample thickness at constant pressure σ_D appears as the straight segment D'-D, with point D representing the state of the sample after stabilisation of volume expansion.



Appendix 4 – Figure 1 – Huder-Amberg swelling test

C – Third step: Decremental unloading

From point D, the load on the sample is instantaneously reduced from pressure σ_D to σ_E; the immediate state of the sample can now be represented by point E'. The effect of load reduction is to allow swelling to proceed in the form of a further volume expansion causing a change in sample height Δh_E. As before, the change is recorded to stabilisation, at which point, the state of the sample can be represented by point E, the change in height of the sample due to unloading from stress σ_D to stress σ_E is represented by straight line E' – E.

This is repeated several times, reducing the stress s each time and, each time, waiting for stabilisation of the height increase Δ_h.

Note. For consistency with ISRM recommendations (see para. 1.4), the unloading should follow a geometrical function:

$$\sigma_i+1 = 0.5\ \sigma_i = 0.25\ \sigma_{i-1}$$

and limit load decrements to a recommended minimum value of 25 kPa.

4.1.2 – Interpretation

The test is interpreted in a semi-log plot in which the sample volume increase Δh is represented by the height change of the sample in the oedometer expressed as a percentage of the initial height:

$$(\Delta h/h) \times 100$$

The changes in pressure on the sample are represented by the axial pressure on the piston, expressed in log σ_i.

A – Huder-Amberg law

Experience shows that, in this semi-log plot, points D, E, F, etc. representing the state of the sample after stabilisation of the height increase at each stage of load reduction plot on a straight line. From this, a very simple relationship can be found between the stress change $\Delta\sigma$ and sample thickness change Δh.

The thickness change Δh when unloading the sample from stress σ_i to stress σ_J can be written

$$\Delta h/h = C_g \times \log (\sigma_j/\sigma_J)$$

in which the swelling index C_g is a constant governed by the intrinsic nature of the material.

B – Huder-Amberg meaning of 'swelling pressure'

The swelling pressure as understood by Huder & Amberg is defined as the stress σ_G beyond which wetting ceases to cause further sample thickness change. This value is given by the intersection of the Huder-Amberg straight line S with the extension of the reloading curve c.

Note. The precision of this determination is improved when stress σ_D on first wetting (point D') is very close to σ_G (this reduces the approximation of the extrapolation).

4.2 – ISRM tests

(see References, Appendix 16.2)

4.2.1 – Axial swelling pressure at constant volume

The sample of height h is inserted in the oedometer as in the Huder-Amberg test. After wetting, the axial pressure on the sample is controlled to oppose any height change Δh in order to keep the sample volume constant. The test is continued until reaching the maximum pressure needed to achieve this.

4.2.2 – Axial swelling pressure versus axial strain

The test procedure is exactly the same as for the Huder-Amberg test except for the way in which the test data is plotted.

Test results are plotted as a curve of percent thickness change strictly due to swelling vs applied axial pressure.

'Thickness change strictly due to swelling' means the total change resulting from the change in pressure minus pseudo-elastic strain corresponding to the same load reduction.

APPENDIX 5

MEANING OF PARAMETERS IN HOEK & BROWN

Hoek & Brown (1980) wrote the intact rock (rock matrix) parabolic failure criterion thus:

$$\sigma_1 = \sigma_3 + \sigma_{ci} (m_i \times \sigma_3/\sigma_{ci} + 1)^{1/2}$$

and introduced constant m_i, thereafter commonly known as the Hoek & Brown coefficient.

The standard form of the parabolic criterion giving components σ_n and τ of the stress on the failure face versus σ_{ci} and σ_{ti} (rock matrix compressive and tensile strengths) is

$$\tau = \sigma_{ti} [(1 + \sigma_{ci}/\sigma_{ti})^{1/2} - 1] \times (1 + \sigma_n/\sigma_{ti})^{1/2}$$

From this, the relationship between the Hoek & Brown coefficient m_i, and σ_{ci} and σ_{ti} is

$$\sigma_{ti} = \sigma_{ci}/2 \times [m_i - (m_i^2 + 4)^{1/2}]$$

This is equivalent to

$$FR^1 = \sigma_{ci}/\sigma_{ti} = 2/[m_i - (m_i^2 + 4)^{1/2}]$$
$$= 1/2 \times [m_i + (m_i^2 + 4)^{1/2}]$$

From the compilation of m_i values (Hoek, Kaiser & Bawden 1995), it is found that this coefficient ranges from 4 (for some clayey rocks) to more than 30 (igneous rocks and some metamorphic rocks). In practice therefore, the following equation can be used:

$$m_i = FR$$

This has the advantage of giving physical meaning to coefficient m_i and allows it to be estimated from standard laboratory tests. Uniaxial compression and Brazilian test results readily lend themselves to statistical analysis. The observation in para. 2.2 in the main text on anisotropy obviously applies.

Finally, the parabolic criterion can be written as

$$\sigma_1 - \sigma_3 = \sigma_{ci} (FR \times \sigma_3/\sigma_{ci} + 1)^{1/2}$$

[1] FR = brittleness index (see para. 2.2.4.3.

APPENDIX 6

CHART DEFINING DRILLING RATE INDEX AND CHARACTERISTICS OF S 20 AND SJ TESTS

The Drilling Rate Index (DRI) was originally developed in the sixties at the Norwegian Institute of Technology at Trondheim to assess the drill hammer . Since the eighties, it has mainly been used to predict the performance of rock tunnelling boring machines (TBM)(see References, Appendix 16.2).

6.1 – Laboratory tests

DRI is calculated from the results of two Norwegian standard laboratory tests:

• The S_{20} brittleness (fragmentation) test (figure 1)

This test estimates the resistance of the rock to crushing under repeated blows, as in the French dynamic fragmentation test.

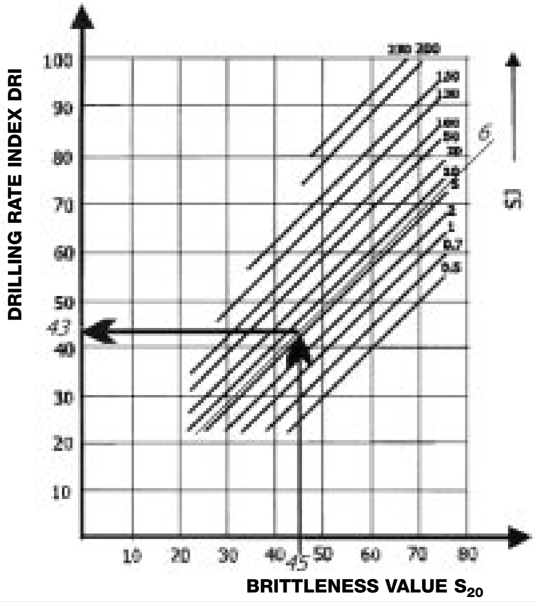
• The Siever J-value (SJ) penetration test (figure 2)

This test estimates rock resistance to penetration, as in the Cerchar-Ineris hardness test.

6.2 – Interpretation

DRI is obtained from S_{20} and SJ using the chart shown in figure 3. It ranges from 20 to 90. A high DRI value indicates easier penetration of the TBM cutters.

Appendix 6 – Figure 3
Chart for calculating Drilling Rate Index



Example:

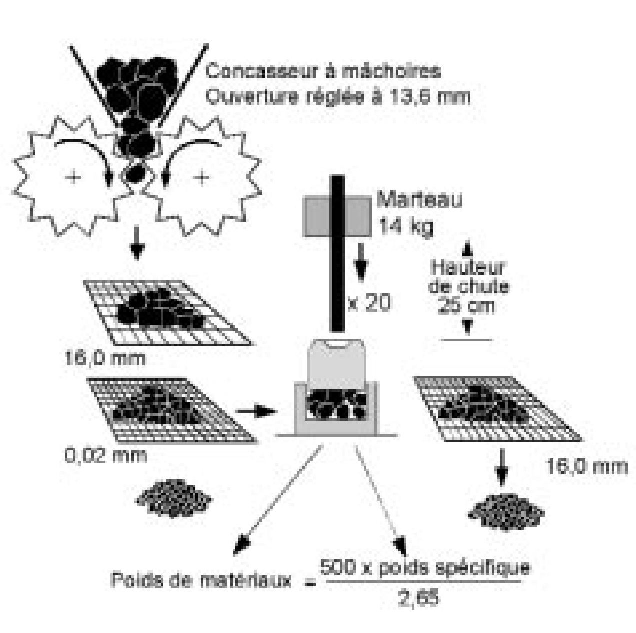
Rock = gneiss of South Africa

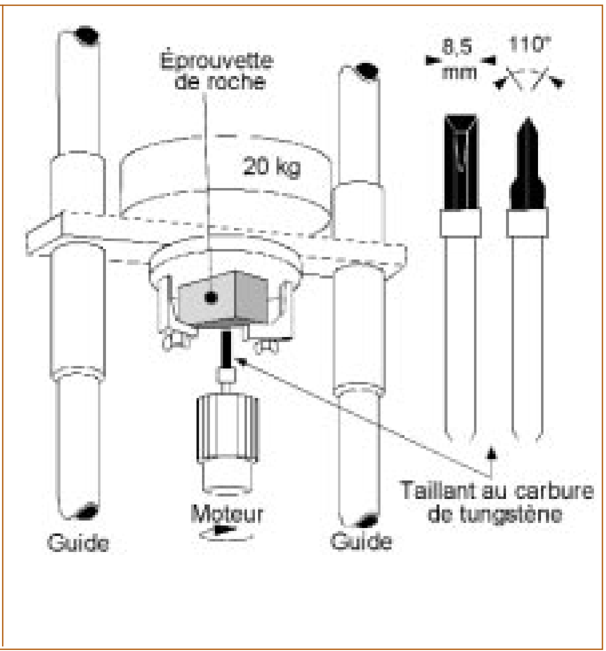
S_{20} = 45

SJ = 6.0

From index A4.3, DRI = 43

More DRI value is ligh,
more the penetration of TBM is high





Appendix 6 – Figure 1 – Brittleness test

Appendix 6 – Figure 2 – Siever test apparatus

APPENDIX 7

TEREOGRAPHIC PROJECTION

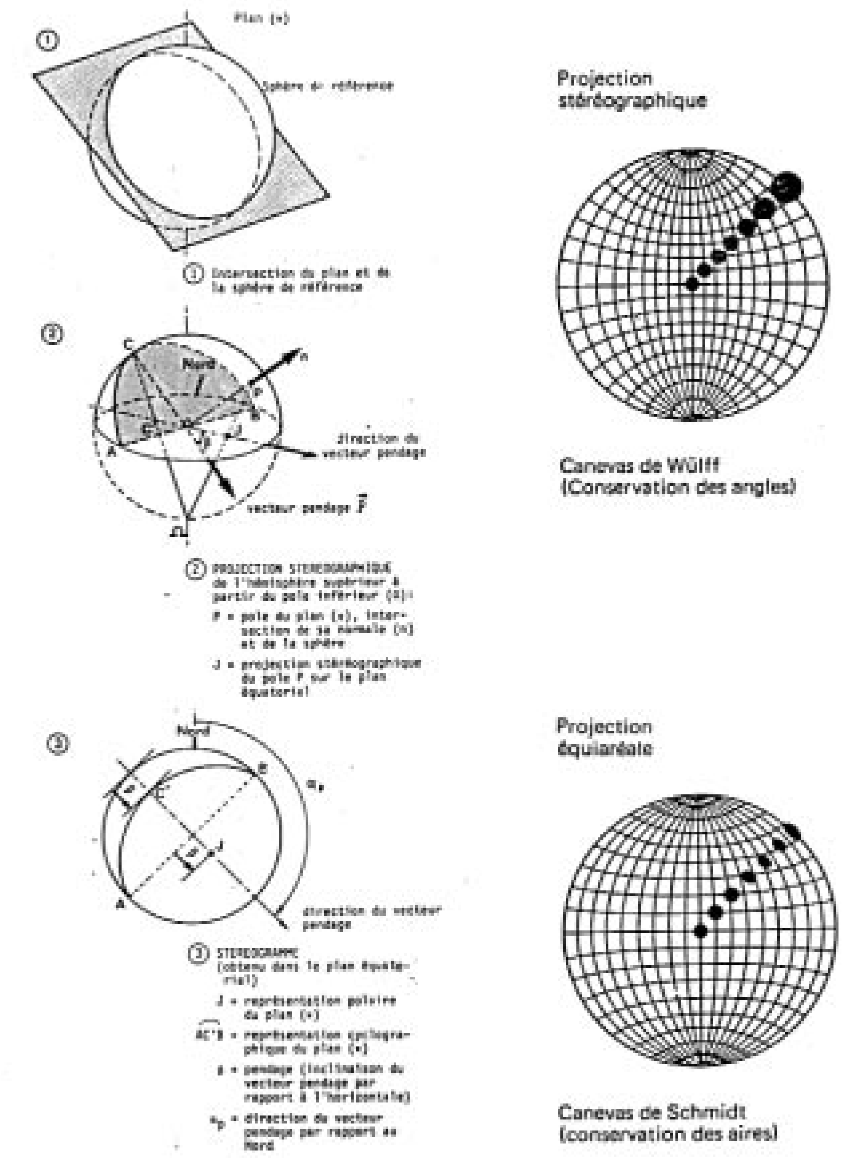
The stereographic projection is the most widely-used method of plotting discontinuities. Various techniques are available but two only are in routine use:

- Equal angle projection to study relationships between discontinuities (Wülff net)
- Equal angle projection to measure spatial distributions (Schmidt net).

Each joint plane is plotted as the projection of its pole (intersection of the normal to the plane with the upper or lower hemisphere of the reference sphere) or the projection of its trace (intersection of the plane with the upper or lower hemisphere of the sphere).

Two types of plots are routinely used to analyse the pattern of discontinuities into sets:

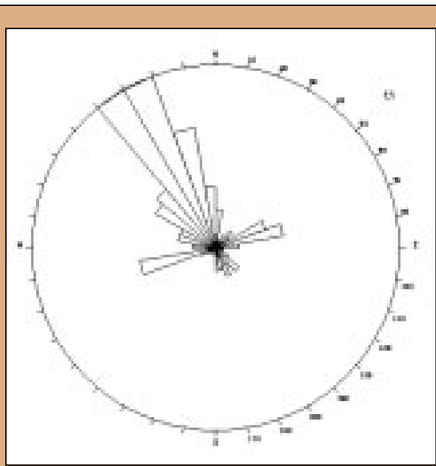
- The cluster of azimuths of the dip vector: this consists of grouping observations in angular sectors of the dip vector azimuth, with the absolute or relative number of observations represented on the radial axis. This type of plot ignores dip angles and is only meaningful for finding directional sets of discontinuities with similar dip angles.
- Density stereograms on the Schmidt net: densities are obtained by counting the number of poles within the target 1% of the diagram area. The count is done on a counting net: count by circles centred on a regular grid or Dimitrijevic count by ellipses centred on an irregular grid. The trace of the joint number or density contours bounds the zones of pole concentrations and may identify main joint sets.



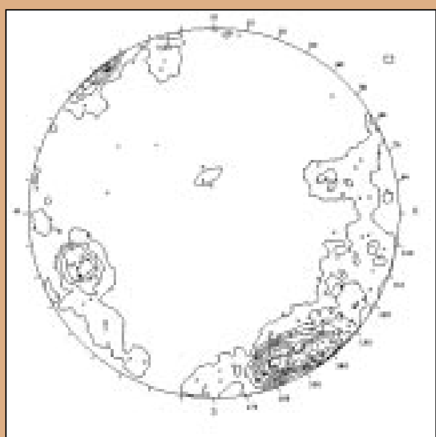
Représentation d'un plan (α_p,β) à l'aide de la projection stéréographique (hémisphère supérieur)

Canevas de Wülff et de Schmidt

Appendix 7 – Figure I – Cyclographic plot and polar plot of a plane of a discontinuty by stereographic projection



Rose diagram of azimuth dip vector (example)



Density stereogram on Schmidt net (lower hemisphere) with isovalues curves

Appendix 7 – Figure 2 – Analysis of patterns of discontinuities (examples)

APPENDIX

APPENDIX 8

SET PERSISTENCE AND 3 D GEOMETRICAL MODELING OF FRACTURE NETWORKS

The persistence of discontinuities can only be estimated from measurements on exposed surfaces (outcrops, adit walls) and must be analysed with caution.

Persistence is only accessible through two-dimensional data – trace lengths – whose measurement is affected by multiple geometrical bias factors. The first is that the survey line preferentially intercepts the longer discontinuities, the second is that the longer discontinuities extend beyond the survey surface, introducing a truncation in the measurements, the last is that some limiting value is generally imposed on short discontinuities, which also has a truncating effect. Data must be corrected for any rigorous estimate of this parameter (Priest & Hudson 1981, Pahl 1981).

Over the last thirty years, many authors have developed three-dimensional geometrical models of networks of discontinuities. Among the more recent ones, the random disk model is one of the most widely used (Baecher 1977, Long 1985, Cacas 1989, Xu 1992). For a given joint set, each discontinuity is represented by a disk of zero thickness, defined by the position of its centre, its radius and orientation, each parameter being drawn stochastically from its own distribution law, characteristic of the set.

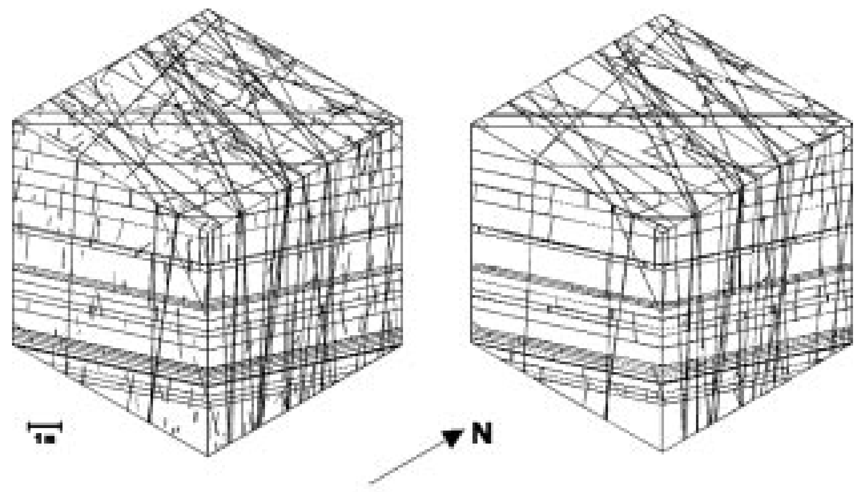
Estimating disk radii, i.e. the true extension of the discontinuities, raises problems. In addition, passing from 2D continuity to 3D persistence is not simple and requires hypotheses on

• The geometrical shape of the discontinuities: in the disk model, it is possible to write mathematical relationships between the 2D distribution of trace lengths and 3D distribution of disk radii (Warburton 1980).

• The persistence distribution law: in this way, the parameters in this law can be determined on the basis of trace length distribution parameters.

From this, one can obtain a 3D model of the network of discontinuities and study its connectivity to analyse the fluid flow through the network or the mechanical behaviour of the assembly of rock blocks created by the fractures, from fracture behaviour and, sometimes, the rock matrix behaviour (figure 1).

More complex models also introduce a ranking of the joint sets (Heliot 1988) or use geostatics (Billaux 1990).



Appendix 8 – Figure 1 – Example of geometrical modelling of network of discontinuities with SIMBLOC program (ENSP-CGI, after Xu 1991)

APPENDIX 9

SCHMIDT HAMMER

The Schmidt hammer is an apparatus for measuring the rebound as a mass strikes a surface, powered by a compression spring, with known energy. Rebound is measured as an index from 0 to 100.

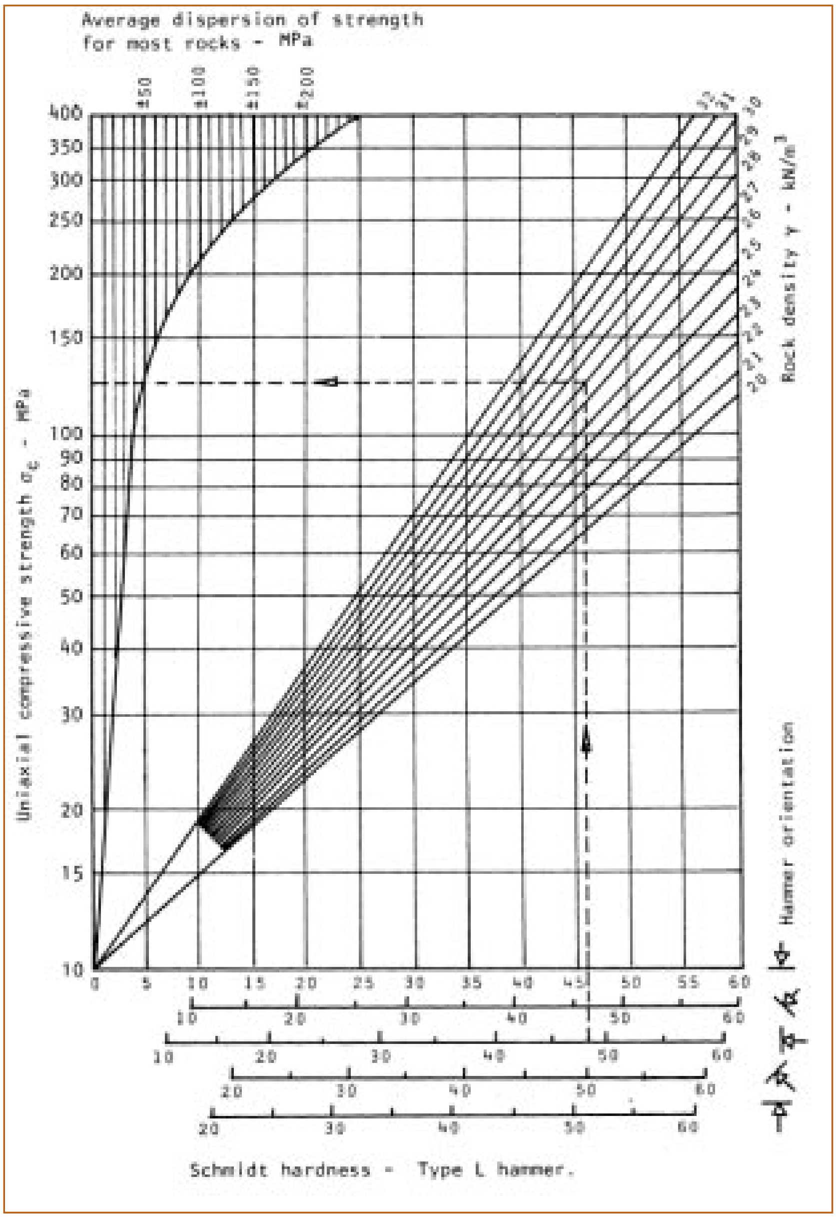
Various types of hammer are available with different impact energies. In rock mechanics, the most widely-used model is the Type L hammer, for which correlations between the rebound index and uniaxial compressive strength have been established by Miller in 1965 (see References, Appendix 16.5) (figure 1).

The measurement of the uniaxial compressive strength of a joint wall is governed by a number of procedural parameters (Barton & Choubey 1977):

• Hammer orientation: rebound is minimum when the apparatus is held vertically downwards, and maximum when it is held vertically upwards. Readings must be corrected when the apparatus is used in other directions. In all cases, it must be held perpendicular to the test surface.

• Sample size: the sample must be large enough for the impact not to cause it to move. Small samples must always be fixed on a rigid base.

• Number of measurements: experiences shows that at least ten readings must be made at different points on a representative sample or per square metre area. The rebound value to be used is the average of the five highest readings, the lowest readings being considered as unrepresentative because the samples are assumed to have moved or the grains to have crushed.



Appendix 9 – Figure 1 – Relationship between Schmidt hammer rebound and uniaxial compressive strength (after Deere & Miller)

APPENDIX 10

HYDRAULIC PARAMETERS

Fluid flow through discontinuities is a highly complex problem. Experimental work (Gentier 1986) has shown it is not isotropic but follows channels whose geometry, while of course dependent on joint width, is also governed by wall roughness and their contact surfaces, by applied normal and tangential stress levels, and by tangential movements of the walls, without forgetting the possible presence of filling material.

Detailed determination of the network of channels and changes thereto due to shear movements is therefore essential for modelling the hydraulic behaviour of a discontinuity, but this is as yet still at the research stage.

Various more or less simplified approaches are however available for estimating fracture flow rate Q with the equation

$$Q = Va = K_f\, J_f\, a \text{ (for laminar flow)}$$

in which

Q is the flow rate through the discontinuity per unit width

V is the mean fluid flow velocity in the discontinuity

K_f is the hydraulic conductivity of the discontinuity

J_f is the orthogonal projection of the hydraulic gradient on the fracture plane

a is the width or aperture of the discontinuity.

With a smooth-walled planar discontinuity, hydraulic conductivity $K_f(LT^{-1})$ depends solely on its physical aperture and, by analogy with flow between flat plates, is written

$$K_f = (a^2g)/12\nu$$

in which

a is the physical aperture of the discontinuity (L)

g is the acceleration due to gravity (LT^{-2})

ν is the kinematic viscosity of the fluid (L^2T^{-1})

whence flow Q is proportional to the cube of the aperture (cubic law).

In a natural discontinuity, the geometry of the voids between the walls is not constant and depends on wall roughness. Various semi-empirical formulae include for roughness with the coefficient

$$r_a/2a$$

called relative hydraulic roughness, in which r_a represents the difference between the highest peak and the lowest trough in joint wall geometry:

$$K_f = (a^2g/12\nu)\,[1/\{1 + B(r_a/2a)^{1.5}\}]$$

with, for example, B = 8.8 (Louis 1969), B = 20.5 (de Quadros 1982).

With $r_a \leq 2a < 0.033$, the effect of roughness is negligible and the cubic law for parallel plate flow can be used.

Barton (1985) established a correlation between relative hydraulic roughness and the Joint Roughness Coefficient JRC. This led him to introduce the concept of hydraulic aperture A of a discontinuity, related to physical aperture a and JRC as

$$A = JRC^{2.5}/(A/a)^2$$

The dimensions of a and A are millimetres.

APPENDIX 11

RIGID PLATE LOADING

The test consists of applying load to the rock by jacking a rigid plate against it. The test is performed in an adit (2-2.5m section) with the reaction being provided by the opposite rock face, allowing two measurements to be made simultaneously. It is usually performed either horizontally or vertically, although other set-ups may be justified in strongly anisotropic rock.

The test apparatus consists of three parts (figure 1):

∑ A 2000-3000 kN hydraulic cylinder is aligned with a stanchion of variable length between two rigid plates. They must be small (280mm diameter) in order to apply a sufficiently high force to neutralise the most decompressed zone at the adit wall, affected by the excavation process. A ball joint makes up for any misalignment and lack of parallelism between the two faces.

∑ A two-speed pump and two 1% class pressure gauges.

∑ A rigid reference frame fixed beyond the zone of influence, carrying displacement gauges (C4, C5, etc.) to measure the displacement of the plate and surrounding area. The displacement gauges may be dial gauges or have an analogue or digital output, accurate to one-hundredth of a millimetre.

One or two extensometers may be installed in small boreholes under the rigid bearing plate to identify decompressed surface zones.

The plate must bear on a flat rock surface, free from any loose fragments caused by blasting, and dressed by bush hammering. Cement dressing should be kept to a minimum, never more than a few millimetres thick. If measurements are to be made with both plates, the bearing surfaces of the rock must be strictly parallel.

Places where the frame is anchored must also be surface-dressed to ensure that rods, supposed to be fixed, are not set on rock spalls, not intimately attached to the surrounding rock.

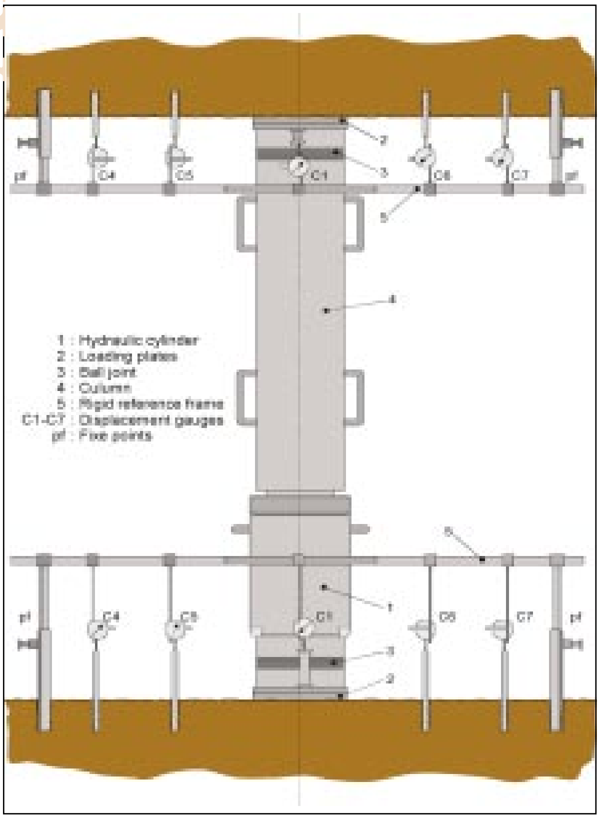
As in the dilatometer test, the rigid plate loading test proceeds by loading/unloading cycles to increasing maxima, maintaining load constant for short times in each cycle to read the gauges and for a longer time at each cycle maximum to detect any creep that might occur. The test may be entirely automated, especially for creep measurements.

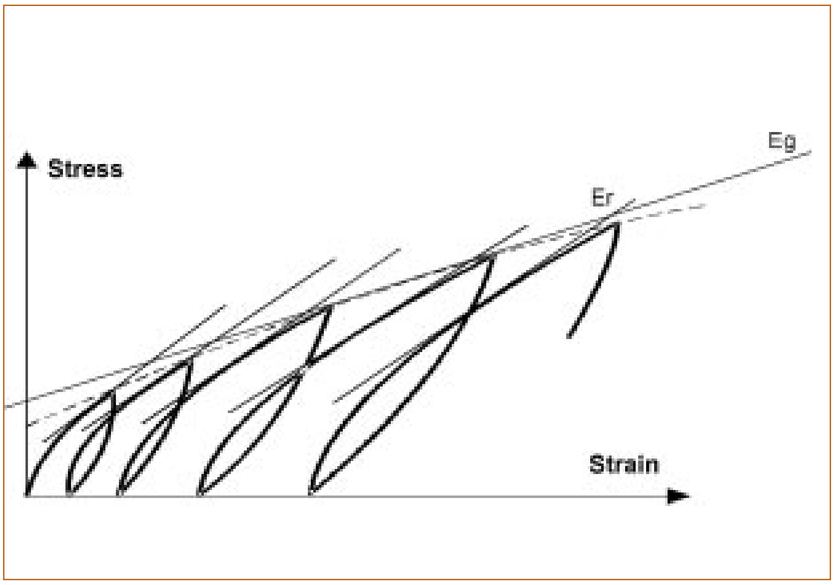
Test data is interpreted with the Boussinesq equation for a circular rigid plate of diameter f and a stress s applied to a semi-infinite, homogeneous, isotropic, elastic material with modulus of elasticity E and Poisson's ratio ν. The displacement of the plate Δd at each stress interval considered is

$$\Delta d = \pi/4 \times (1 - \nu^2) \times \Delta\sigma \times \phi/E$$

If ν is not known, it is usually assigned a value of 0.25.

The stress strain curves for successive cycles are plotted for each test (figure 2) and so-called deformation global moduli are found, along with higher reversible moduli. A global deformation modulus is also determined, which corresponds to the mean slope of the tangent to the cycle curves.





Appendix 11 – Figure 2 – Curves from plate loading test and derived moduli

Appendix 11 – Figure 1 – Rigid plate loading test apparatus set-up

APPENDIX 12

THERMAL ROCK PARAMETERS

Rock	Conductivity // λ_1 (W/m.°C)	Conductivity ⊥ λ_2 (W/m.°C)
Granites	2.7 – 3.5	2.7 – 3.5
Basalt	2.2	2.2
Gneiss	3 – 4	2V6 - 3
Crystalline schists	3.3 – 4.7	2.5 - 3
Quartzites	5-5.6	5
Anhydrite	5.5	5.5
Sandstone	2.7	2.7
Limestones	2.5-3.6	1.9-3.6
Dolomite	4.2	4.2
Clay	1.9	1.9
Rock salt	5.7	5.7

Appendix 12 – Table 1
Thermal conductivities of selected rocks (after Mechler 1982, Handbook of Chemistry and Physics, 1987, Goy 1996)

The lower the thermal conductivity value, the more the rock acts as a heat insulant

APPENDIX 13

BIENIAWSKI'S RMR CLASSIFICATION OF ROCK MASSES

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		Range of values						
1	Strength of intact rock material	Point-load strength index Is	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range-uniaxial compressive test is prédered		
		Uniaxial comp. strength σ_c	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill Core Quality RQD		90% - 100%	75% - 90 %	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0,6 m - 2 m	200 mm - 600 mm	60 mm - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (see E)		Very rough surfaces Not continuous No separartion Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm or Separation 1 -5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground Water	Inflow per 10 m tunnel length (l/min)	None	< 10 l/min	10 - 25 l/min	25 -125 l/mn	> 125 l/mn		
		(Joint water press)/ (Major principal σ_1)	0	< 0,1	0,1 - 0,2	0,2 - 0,5	> 0,5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

RMR = Summ of rating of parameters 1 to 5

B. RATING ADJUSTEMENT FOR DISCONTINUITY ORIENTATION (See F)

Strike and dip orientation		Very favourable	Favourable	Fai	Unfavourable	Very unfavourabble
Ratings	Tunnelsand mines	0	- 2	- 5	- 10	- 12
	Foundations	0	- 2	- 7	- 15	- 25
	Slopes	0	- 5	- 25	- 50	- 60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

RATING	100 ? 81	80 ? 61	60 ? 41	40 ? 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 weak for 5 m span	10 hrs for 2,5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400 kPa	300 à 400 kPa	200 à 300 kPa	100 à 200 kPa	< 100 kPa
Friction angle of rock mass (°)	> 45°	35° à 45°	25° à 35°	15° à 25°	< 15°

Appendix 13 – Table 1 (cont'd) – Definition of RMR (after Bieniawski 1989)

APPENDIX 13

BIENIAWSKI'S RMR CLASSIFICATION OF ROCK MASSES

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS					
Discontinuity length (persistence)	< 1 m	de 1 - 3 m	de 3 - 10 m	10 - 20 m	> 20 m
Rating	**6**	**4**	**2**	**1**	**0**
Separartion(aperture)	None	< 0,1 mm	0,1 - 1 mm	1 - 5 mm	> 5 mm
Rating	**6**	**5**	**4**	**1**	**0**
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	**6**	**5**	**3**	**1**	**0**
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	**6**	**4**	**2**	**2**	**0**
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	**6**	**5**	**3**	**1**	**0**

Nota : Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be over shadowed by the influence of the gouge. In such cases use A.4 directly

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip		Dip 45° à 90° : **very favourable**	Dip 20° à 45° : **fair**
Dip 45° à 90° : **very favourable**	Dip 20° à 45° : **favourable**		
Drive against dip		Dip 0° à 20° - Irrespective of strike : **fair**	
Dip 45° à 90° : **fair**	Dip 20° à 45° : **unfavourable**		

Appendix 13 – Table 1 (cont'd) – Definition of RMR (after Bieniawski 1989)

APPENDIX 14

BARTON'S Q INDEX

DESCRIPTION	VALUE	NOTES	
ROCK QUALITY DESIGNATION	**RQD**		
Very poor	0-25	i) Where RQD is reported or measured	
Poor	25-50	as < 10 (including 0) a nominal	
Fair	50 - 75	value of 10 is used to evaluate Q	
Good	75 - 90	ii) RQD intervals of 5, i.e. 100, 95,90 etc. are suffi-	
ciently		accurate	
Excellent	90 -100		
JOINT SET NUMBER	**Jn**		
Massive, no or few joints	0.5 – 1.0		
One joint set	2		
One joint set plus random	3		
Two joint sets	4		
Two joint sets plus random	6		
Three joint sets	9	i) For intersections use (3.0 x Jn)	
Three joint sets plus random	12		
Four or more joint sets, random, heavily jointed, "sugar cube", etc	15	ii) For portals use (2.0 x Jn)	
Crushed rock, earthlike	20		
JOINT ROUGHNESS NUMBER	**Jr**		
a) Rock wall contact			
b) Rock wall contact before 10 cm shear			
Discontinuous joints	4		
Rough and irregular, undulating	3		
Smooth undulating	2		
Slickensided undulating	1.5		
Rough or irregular, planar	1.5		
Smooth, planar	1.0		
Slickensided, planar	0.5	i)Add 1.0 if the mean spacing of the relevant joint set is greater than 3	
c) No rock wall contact when sheared			
Zones containing clay linerals thick enough to prevent rock wall contact	1.0 (nominal)	ii) Jr = 0.5 can be used for planar, slickensided joints having lineations,	
Sandy, gravely or crushed zone thick enough o prevent rock wall contact	1.0 (nominal)	Provided that the lineations are oriented for minimum strength.	
JOINT ALTERATION NUMBER	**Ja**	**_r degrees (approx.)**	
a) Rock wall contact			
Tightly healed, hard, non-softening, impermeable filling	0.75		i) Values of _r, the residual angle friction
Unaltered joint walls, surface staining only	1.0	25 - 35	Are intended as an approximate
Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free, disintegrated rock, etc.	2.0	25 - 30	Guide to the mineralogical properties of the
Silty, or sandy-clay coatings, small clay fraction (non-softening)	3.0	20 - 25	Alteration products, if present
Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays (Discontinuous coatings 1 – 2 mm or less in thickness)	4.0	8 - 16	

DESCRIPTION	VALUE		NOTES
JOINT ALTERATION NUMBER	Ja		_r degrees (approx.)
b) Rock wall contact before 10 cm shear			
Sandy particles, clay free, disintegrating rock, etc.	4.0	25 -30	
Stronly over-consolidated, non-softening clay minerals fillings (continuous <5 mm thick)	6	16 - 24	
Medium or low over-consolidation, softening clay mineral fillings (continuous <5 mm thick)	8	12 - 16	
Swelling clay fillings, i.e. montmorillonite, (continuous <5 mm thick). Values of Ja depend on percent of swelling clay-size particles, and access to water	8.0 –12.0	6 -12	
c) No rock wallcontact when sheared			
Zones or bands of disintegrated or crushed	6.0		
Rock and clay	8.0		
Zones or bands of silty-or sandy clay, small clay fraction, non-softening	5.0		
Thick continuous zones or bands of clay	10.0 – 13.0		
JOINT WATER REDUCTION	Jw	Approx.water pressure (kgf/cm^2)	
Dry excavation or minir inflow i.e. < 5 l/m locally	1.0	< 1.0	
Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 – 2.5	i) Factors are crude estimates;
Large inflow or high pressure in competent rock with installed	0.5	2.5 – 10.0	increase Jw if drainage
unfilled joints			
Large inflow or high pressure	0.33	2.5 – 10.0	
Exceptionally high inflow or pressure at blasting, decaying with time	0.2 – 0.1	> 10	ii) Special problems caused
Exceptionally high inflow or pressure	0.1 – 0.05	> 10	By ice formation are not considered

STRESS REDUCTION FACTOR	SRF			
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock and depth)	10.0			i)Reduce these values of SRF by 25-50% if the relevant shear zons only influence but do not intersect the excavation
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m)	5.0			
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)	2.5			
Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5			
Single shear zone in competent rock (clay free) (depth of excavation <50 m)	5.0			
Single shear zone in competent rock (clay free) (depth of excavation >50 m)	2.5			
Loose open joints, heavily jointed or "sugar cube", (any depth)	5			
b) Competent rock, rock stress problems	σ_c / σ_1	σ_t / σ_1	SFR	
Low stress, near surface	> 200	> 13	2,5	i) For strongly anisotropic virgin stress field (if measured): when
Medium stress	200 - 10	13 –0.66	1.0	$5<\sigma_1 / \sigma_3 <10$ reduce σ_c to $0.8\sigma_c$
High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 – 0.33	0.5-2	and σ_t to $0,8\sigma_t$. When $\sigma_1 / \sigma_3 >10$
Mild rockburst (massive rock)	5-2.5	0.33-0.16	5-10	Reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\ \sigma_t$
Heavy rockburst (massive rock)	< 2.5	< 0.16	10-20	
c) Squeezing rock plastic flow og incompetent rock under influence of high rock pressure				
Mild squeezing rock pressure			5 – 10	ii) Few case records
Heavy squeezing rock pressure			10 -20	avalaible where depth of crown below
d) Swelling rock, chemical swelling activity depending on presence of water				surface is less than span width. Suggest
Mild swelling rock pressure			5 – 10	SRF increase from
Heavy swelling rock pressure			10 -15	2.5 to 5 for such cases

APPENDIX 15

EMPIRICAL FORMULAE FOR EVALUATING ROCK MASS DEFORMATION MODULI

Many empirical relationships have been proposed by various authors to estimate the rock mass deformation modulus from the characteristics obtained from laboratory samples and other parameters. Table 1 lists the more important correlations with RMR, RQD and Q index used in rock mass classification systems.

AFTES expresses no opinion on the relevance or validity of these correlations in respect of the use to which they are put. Users should refer to the writings of the relevant authors.

ROCK MASS DEFORMATION MODULUS E_M (MPa)	AUTHORS
2*(RMR – 50) ou 1.7* (RQD –60)	Cording *et al* (1971)
25 Ln Q ou 10 Ln Q	Fujita (1977)
0,7*(RMR/100)2 *E_i	Barton (1980)
RMR/10 + ($RMR^3/10^5$)	Hoek & Brown (1982)
$10^{(RMR-10)/40}$	Serafim & Pereira (1983)
0,5*$(RQD/100)^2$ *E_i	Bieniawski (1989)
10*exp[(RMR-10)/40]	Grimstad & Barton (1993)
0,07*RQD+0.05 σ_c+55*E_i	Hönisch (1993)
1000 *$[\sigma_c/100]^{0,5}$*$10^{(GSI-10)/40}$ (σ_c<100 MPa)	Hoek & Brown (1997)

Appendix 15 – Table 1 – Empirical relationships proposed by various authors for assessing rock mass deformation modulus

- Ei Modulus of elasticity of rock measured on laboratory samples
- σ_c Uniaxial compressive strength of rock measured on laboratory samples
- RQD Rock Quality Designation (Deere 1967)
- Q Quality factor (Barton 1980)
- RMR Rock Mass Rating (Bieniawski 1989)
- RMR = 50 + 15 log10 Q (Barton 1995)
- GSI Geological Strength Index
- GSI = RMR89 – 5 (Hoek & Brown 1994, 1995); GSI = 9 Ln Q + 44 (Bieniawski 1989)

APPENDIX 16

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