INTRODUCTION

1. DEFINITIONS

Rock mechanics is the branch of *geotechnical engineering* concerned with the engineering mechanics and the properties of rocks. In general, civil engineers deal with two types of earth materials: *soils* and *rocks*. Geotechnology is a term used in the literature to describe both the science and engineering of soil deposits, rock masses, and the fluids they contain.

Soils are aggregates of mineral grains that can be separated by slight mechanical means such as agitation in water. On the other hand, rocks are aggregates of mineral grains that are connected by strong and permanent forces.

In general, soils are the end-products of the mechanical or chemical weathering of rocks. The zone where the rock is weathered, also called the "weathered zone", can be a few feet thick in arid areas and several hundred feet thick in tropical (hot and humid) areas. A rock that is not weathered is called "fresh" (weathering grade I), whereas a fully weathered rock is called a soil (weathering grade VI). Obviously, there is a wide range of materials in nature that fit in between those two extremes. Some of them have properties that are more rock dominant whereas others have properties that are more soil dominant. These materials are sometimes called *rock-like soils* or *soil-like rocks*. As civil engineers, we need to know the vertical and lateral extent of the weathering zone, the different grades of weathering, and the depth to the fresh unweathered rock.

A clear understanding of the difference between a soil and a rock is necessary as far as engineering contracts are concerned. Indeed, a misunderstanding might lead to costly legal disputes. Very often among geotechnical engineers, a soil refers to a material that can be excavated without blasting (with a mechanical ripper for instance), whereas a rock is a material for which blasting is required for excavation. From a behavioral point of view, we should also keep in mind that the engineering properties (deformability, strength, permeability, etc..) of a rock mass do change as it weathers.

You should be aware that the engineering definitions of soils and rocks are not universal and are not always accepted by geologists. For a geologist, the term "rock" means all the material found in the Earth's crust regardless of the degree of bonding between the mineral grains, whereas the term "soil" is reserved for the upper part of the ground surface that is supporting the vegetation. These definitions have also been adopted by peodologists and agronomists that are only concerned with the upper layers of soils bearing forest and agriculture.

The communication problem between geologists and engineers exists also with regard to rock classification. In general, geologists classify rocks into three major groups: igneous, sedimentary, and metamorphic. Rocks are classified from a genetic point of view, i.e. how they were formed. Each rock group is further divided into sub-groups based on the grain size, the rock texture, etc.. On the other hand, engineers are more interested in how rocks behave in practice. They classify rocks based on their performance in various engineering applications such as drilling, blasting, tunneling, rock/dam interaction, etc..

In rock mechanics and rock engineering, you should also be aware that there is a clear distinction between "rock" and "rock mass". The term "rock" refers to the intact material, whereas "rock mass" is used to describe the material *in situ* which can be seen as an "assemblage" of blocks of intact rock material separated by discontinuities, fractures, etc... Laboratory tests are usually done on core samples of intact rock and are therefore of limited value. On the other hand, field tests are conducted in boreholes, galleries, etc.. and involve a volume of the rock mass. The response of a rock mass to a test depends on both the intact rock and the discontinuities. In practice, the results of both laboratory and field tests are integrated into rock engineering design where the main objective is to understand the interaction of a rock mass with an engineering structure.

2. FIELDS OF APPLICATION OF ROCK MECHANICS

Rocks can be used in themselves as raw sources for construction materials (aggregates, construction stones, decorative stones, etc..). Also, many engineering activities involve rocks either as construction or foundation material. These include:

- Design of foundations for buildings, bridges, dams, towers, etc...
- Design of rock slopes and surface excavations for canals, highways, railways, spillways, pipelines, penstocks, dam abutments, open pit mines, quarries, etc..
- Design of underground excavations such as tunnels, mines and other underground chambers,
- Design of structures associated with energy development such as underground nuclear plants, repositories for storage of nuclear and chemical wastes, LNG and oil.

Although the methodology may differ from one activity to the other, all these activities have three basic similarities. First, they all require an evaluation of the site geology, i.e. rock types, extent of each rock unit, extent and type of weathering, etc... This is usually done by conducting detailed site exploration and investigation using surface mapping, boreholes, trenches, or geophysical survey. Site exploration and investigation is usually conducted in several steps (preliminary, advanced, etc..). Second, all the aforementioned activities require an assessment of the engineering properties (strength, deformability, permeability, etc..) of the rocks involved in the projects. This is done by testing samples of intact rock in the laboratory and by conducting field tests. Finally, engineers need to take into account possible geologic hazards and their impact on existing and future structures. In general, geological hazards can be divided into hazards from geological materials (reactive minerals, asbestos, gas hazards), and hazards from geological processes (volcanoes, earthquakes, landslides and avalanches, subsidence, floods, coastal erosion).

3. OBJECTIVES OF ROCK MECHANICS

For most engineering projects involving rocks, the objectives of rock mechanics are essentially of three fold:

(i) *Determine* the properties of the rock and the rock mass associated with the project of interest. These properties may be physical, mechanical, hydraulic or thermal. Not all properties need to be determined but only those that are deemed necessary. In addition to these properties, the *in situ* stress field needs to be measured as well. The intact rock and rock mass properties are usually determined in the laboratory and in the field, respectively.

(ii) *Model* and *predict* the behavior of the rock mass when subjected to the new loads associated with the engineering structure to be built.

(iii) Finally, once the engineering structure is built and upon its completion, the third objective is to *observe* and *monitor* its response and behavior with adequate instrumentation.

4. HISTORICAL DEVELOPMENT OF ROCK MECHANICS

Compared to the field of soil mechanics, the development of rock mechanics has been much slower. The need for understanding the behavior of rocks has been recognized by geologists and mining engineers at the turn of the 20th century. Several attempts to characterize and model the behavior of rock masses have been carried out by engineers involved in the construction of tunnels in the Alps at the beginning of this century. One has to wait until 1957 for rock mechanics to become a separate discipline with the first treatise written by Talobre.

Unfortunately, two major disasters have contributed to the advancement of rock mechanics and have forced the engineering profession to better understand the behavior of rock masses. One of these disasters took place in France. On December 02, 1959, the Malpasset dam burst due to an instability of its left abutment killing 421 people. Another disaster took place a few years later in Italy; on October 09,1963, a major rock slide caused the Vaiont disaster killing 2600 people.

The two previous disasters triggered the creation of the International Society for Rock Mechanics (ISRM) which organized its first congress in Madrid in 1966. Since then, the ISRM has sponsored multiple international congresses and workshops.

Over the past 30 years, the number of engineering projects involving rock either as construction or foundation material has increased drastically. These engineering activities have strongly enhanced the need for a better understanding of rock behavior. As a result, a large body of literature is now available on the subjects of *rock mechanics* and *rock engineering*.

Modern rock mechanics is an interdisciplinary field. Indeed, in order to solve rock mechanics problems, information from other fields are needed. These include: engineering geology, geology, mechanics, hydraulics, mathematics, physics, chemistry, and soil mechanics, among others.

5. ROCK AS AN ENGINEERING MATERIAL

Despite its analytical aspect, rock mechanics still remains an art since the rock mechanics engineer is faced with an engineering material of uttermost complexity for which engineering judgment and experience are required. In general, rock and rock mass properties cannot be assigned to a design calculation with the same degree of certainty as for other types of engineering materials such as concrete or steel for three reasons.

First, information obtained from the testing of rock specimens or from field observations through outcrops, trenches, boreholes, geophysics (surface and down hole), and excavations are not sufficient to provide a complete picture of the rock mass of interest. Uncertainties are inherent when dealing with rock masses; uncertainties in the material itself, uncertainties in data collection and testing and uncertainties in model prediction.

Second, rock is a very complex material that can be:

- *Discontinuous* with micro-discontinuities (pores, microcracks) and macro-discontinuities (joints, shears, faults) (Table 1);

- *Anisotropic* if its properties vary with directions as for sedimentary rocks, foliated metamorphic rocks and regularly jointed rocks;

- *Heterogeneous* if its properties vary from point to point as in multilayered rock masses.

Note that rocks and especially rock masses can rarely be described as isotropic, homogeneous continua. Thus, continuum mechanics is of limited value when modeling rocks. In addition, rock properties can be *time-dependent* and *scale-dependent* (Figure 1). The scale dependency implies that when modeling a rock mass, we need to take into account the relative scale of the structure of interest with respect to the scale of the major rock mass features. The scale dependency is in both time and space. By scaling we are concerned not only with the scaling of rock mass properties but also the scaling of the mechanisms involved in rock mass behavior.

Third, to complicate things even more, geological processes are coupled in a strong non-linear fashion. The processes can be mechanical, hydrological, thermal, chemical, and/or biological.

With the previous characteristics in mind, it is unlikely that rock mechanics will ever be successful in producing a fully coupled deterministic model of rock masses. An exact prediction of rock mass behavior is not possible. Because of the complex nature of rock as an engineering material, the design methods in rock engineering can vary depending on the geologic environment, the rock type, the type of engineering structure, the design loads that have to be considered, and the end uses for which the engineering structure is intended. This is discussed more extensively in a paper by Hoek (1991). Tables 2-5 were extracted from that paper.

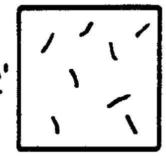
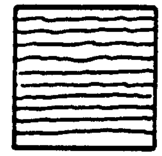
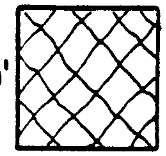
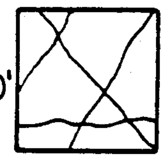
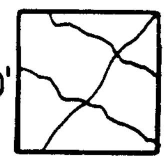
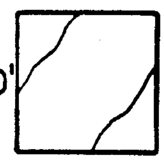
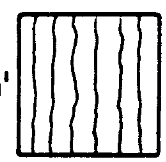
GROUP	TYPICAL DISCONTINUITIES	TYPICAL SCALE
Rock Type Defects	Microfissures	0.2' 
	Bedding Plane Partings Foliation Partings	2' 
Detailed Discontinuity Pattern	Joints Minor Shears Minor Seams	20' 
	Shears Seams	200' 
Gross Discontinuity Pattern	Major Shears or Crushing Zones	2000' 
	Regional Fault Zones	20,000' 
Induced Fractures	Buckling or Spalling Fractures	0.2' - 20' 

Table 1. Classification of discontinuities according to scale (after Brekke and Howard, 1973).

Table 2 : Failure modes, critical parameters, methods of analysis and acceptability criteria for rock slopes.

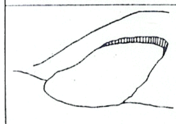
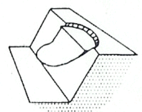
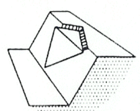
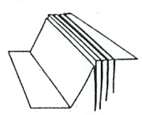
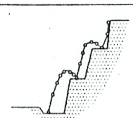
STRUCTURE	FAILURE MODE	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Landslides.	Complex failure along a circular or near circular failure surface involving sliding on faults and other structural features as well as failure of intact materials.	• Presence of regional faults. • Shear strength of materials along failure surface. • Groundwater distribution in slope, particularly in response to rainfall or to submergence of slope toe. • Potential earthquake loading.	Limit equilibrium methods which allow for non-circular failure surfaces can be used to estimate changes in factor of safety as a result of drainage or slope profile changes. Numerical methods such as finite element or discrete element analysis can be used to investigate failure mechanisms and history of slope displacement.	Absolute value of factor of safety has little meaning but rate of change of factor of safety can be used to judge effectiveness of remedial measures. Long term monitoring of surface and subsurface displacements in slope is the only practical means of evaluating slope behaviour and effectiveness of remedial action.
 Soil or heavily jointed rock slopes.	Circular failure along a spoon-shaped surface through soil or heavily jointed rock masses.	• Height and angle of slope face. • Shear strength of materials along failure surface. • Groundwater distribution in slope. • Potential surcharge or earthquake loading.	Two-dimensional limit equilibrium methods which include automatic searching for the critical failure surface are used for parametric studies of factor of safety. Probability analyses, three-dimensional limit equilibrium analyses or numerical stress analyses are occasionally used to investigate unusual slope problems.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Higher factors of safety may be required in order to limit deformations of the slope.
 Jointed rock slopes.	Planar or wedge sliding on one structural feature or along the line of intersection of two structural features.	• Slope height, angle and orientation. • Dip and strike of structural features. • Shear strength of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Limit equilibrium analyses which determine three-dimensional sliding modes are used for parametric studies on factor of safety. Failure probability analyses, based upon distribution of structural orientations and shear strengths, are useful for some applications.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Probability of failure of 10 to 15% may be acceptable for open pit mine slopes where cost of clean up is less than cost of stabilization.
 Vertically jointed rock slopes .	Toppling of columns separated from the rock mass by steeply dipping structural features which are parallel or nearly parallel to the slope face.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Crude limit equilibrium analyses of simplified block models are useful for estimating potential for toppling and sliding. Discrete element models of simplified slope geometry can be used for exploring toppling failure mechanisms.	No generally acceptable criterion for toppling failure is available although potential for toppling is usually obvious. Monitoring of slope displacements is the only practical means of determining slope behaviour and effectiveness of remedial measures.
 Loose boulders on rock slopes.	Sliding, rolling, falling and bouncing of loose rocks and boulders on the slope.	• Geometry of slope. • Presence of loose boulders. • Coefficients of restitution of materials forming slope. • Presence of structures to arrest falling and bouncing rocks.	Calculation of trajectories of falling or bouncing rocks based upon velocity changes at each impact is generally adequate. Monte Carlo analyses of many trajectories based upon variation of slope geometry and surface properties give useful information on distribution of fallen rocks.	Location of fallen rock or distribution of a large number of fallen rocks will give an indication of the magnitude of the potential rockfall problem and of the effectiveness of remedial measures such as draped mesh, catch fences and ditches at the toe of the slope.

Table 3 : Failure modes, critical parameters, methods of analysis and acceptability criteria for dams and foundations.

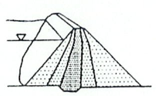
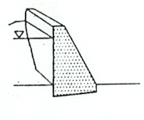
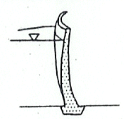
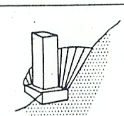
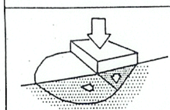
STRUCTURE	FAILURE MODE	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Zoned fill dams.	Circular or near-circular failure of dam, particularly during rapid drawdown. Foundation failure on weak seams. Piping and erosion of core.	• Presence of weak or permeable zones in foundation. • Shear strength, durability, gradation and quality of placement of dam construction materials. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Seepage analyses are required to determine water pressure and velocity distribution through dam and abutments. Limit equilibrium methods should be used for parametric studies of stability. Numerical methods can be used to investigate dynamic response of dam during earthquakes.	Factor of safety against sliding on weak seams in foundation should exceed 2 for full pool conditions. Higher factors of safety may be required to limit deformations. Factor of safety of upstream slopes should exceed 1.5 for rapid drawdown conditions. Hydraulic gradients should be low to prevent erosion.
 Gravity dams.	Shear failure of interface between concrete and rock or of foundation rock. Tension crack formation at heel of dam. Leakage through foundation and abutments.	• Presence of weak or permeable zones in rock mass. • Shear strength of interface between concrete and rock. • Shear strength of rock mass. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Parametric studies using limit equilibrium methods should be used to investigate sliding on the interface between concrete and rock and sliding on weak seams in the foundation. A large number of trial failure surfaces are required unless a non-circular failure analysis with automatic detection of critical failure surfaces is available.	Factor of safety against foundation failure should exceed 3 for normal full pool operating conditions. Factor of safety > 1.5 for Probable Maximum Flood (PMF). Factor of safety > 1 for extreme loading - maximum credible earthquake and PMF.
 Arch dams.	Shear failure in foundation or abutments. Cracking of arch due to differential settlements of foundation . Leakage through foundations or abutments.	• Presence of weak, deformable or permeable zones in rock mass. • Orientation, inclination and shear strength of structural features. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Limit equilibrium methods are used for parametric studies of three-dimensional sliding modes in the foundation and abutments, including the influence of water pressures and reinforcement. Three-dimensional numerical analyses are required to determine stresses and displacements in the concrete arch.	Factor of safety against foundation failure should exceed 3 for normal full pool operating conditions and should exceed 2 for Probable Maximum Flood conditions. Stresses and deformations in concrete arch should be within allowable working levels defined in concrete specifications.
 Foundations on rock slopes.	Slope failure resulting from excessive foundation loading. Differential settlement due to anisotropic deformation properties of foundation rocks.	• Orientation, inclination and shear strength of structural features in rock mass forming foundation. • Presence of inclined layers with significantly different deformation properties. • Groundwater distribution in slope.	Limit equilibrium analyses of potential planar or wedge failures in the foundation or in adjacent slopes are used for parametric studies of factor of safety. Numerical analyses can be used to determine foundation deformation, particularly for anisotropic rock masses.	Factor of safety against sliding of any potential foundation wedges or blocks should exceed 1.5 for normal operating conditions. Differential settlement should be within limits specified by structural engineers.
 Foundations on soft rock or soil.	Bearing capacity failure resulting from shear failure of soils or weak rocks underlying foundation slab.	• Shear strength of soil or jointed rock materials. • Groundwater distribution in soil or rock foundation. • Foundation loading conditions and potential for earthquake loading.	Limit equilibrium analyses using inclined slices and non-circular failure surfaces are used for parametric studies of factor of safety. Numerical analyses may be required to determine deformations, particularly for anisotropic foundation materials.	Factor of safety against bearing capacity failure should exceed 2 for normal loading conditions. Higher factors of safety may be required to limit deformations. Differential settlement should be within limits specified by structural engineers.

Table 4: Failure modes, critical parameters, methods of analysis and acceptability criteria for underground civil engineering excavations.

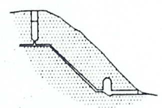
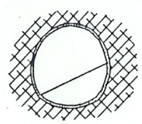
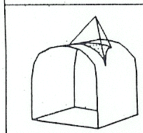
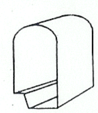
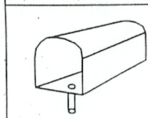
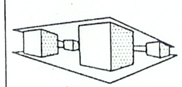
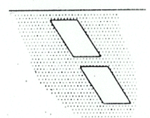
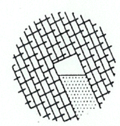
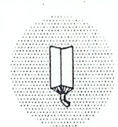
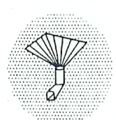
STRUCTURE	FAILURE MODE	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Pressure tunnels in hydro-power projects.	Excessive leakage from unlined or concrete lined tunnels. Rupture or buckling of steel lining due to rock deformation or external pressure.	• Ratio of maximum hydraulic pressure in tunnel to minimum principal stress in the surrounding rock. • Length, thickness and effectiveness of grouting of steel lining. • Water pressure distribution in the rock mass.	Determination of minimum cover depths along pressure tunnel route from accurate topographic maps. Stress analyses of sections along and across tunnel axis. Comparison between minimum principal stresses and maximum dynamic hydraulic pressure to determine steel lining lengths.	Steel lining is required where the minimum principal stress in the rock is less than 1.3 times the maximum static head for typical hydroelectric operations or 1.15 for operations with very low dynamic pressures. Hydraulic pressure testing in boreholes at the calculated ends of the steel lining is essential to check the design assumptions.
 Soft rock tunnels.	Rock failure where strength is exceeded by induced stresses. Swelling, squeezing or excessive closure if support is inadequate.	• Strength of rock mass and of individual structural features. • Swelling potential, particularly of sedimentary rocks. • Capacity and installation sequence of support systems.	Stress analyses using numerical methods to determine extent of failure zones and probable displacements in the rock mass. Rock-support interaction analyses using closed-form or numerical methods to determine capacity and installation sequence for support and to estimate displacements in the rock mass.	Capacity of installed support should be sufficient to stabilize the rock mass and to limit closure to an acceptable level. Tunnelling machines and internal structures must be designed for closure of the tunnel as a result of swelling or time-dependent deformation. Monitoring of deformations is an important aspect of construction control.
 Shallow tunnels in jointed rock.	Gravity driven falling or sliding wedges or blocks defined by intersecting structural features. Unravelling of inadequately supported surface material.	• Orientation, inclination and shear strength of structural features in the rock mass. • Shape and orientation of excavation. • Quality of drilling and blasting during excavation. • Capacity and installation sequence of support systems.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass surrounding the tunnel. Limit equilibrium analyses of critical wedges are used for parametric studies on the mode of failure, factor of safety and support requirements.	Factor of safety, including the effects of reinforcement, should exceed 1.5 for sliding and 2.0 for falling wedges and blocks. Support installation sequence is critical and wedges or blocks should be identified and supported before they are fully exposed by excavation. Displacement monitoring is of little value.
 Large caverns in jointed rock.	Gravity driven falling or sliding wedges or tensile and shear failure of rock mass, depending upon spacing of structural features and magnitude of in situ stresses.	• Orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Shape and orientation of cavern. • Excavation and support sequence and quality of drilling and blasting.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass. Stresses and displacements induced by each stage of cavern excavation are determined by numerical analyses and are used to estimate support requirements for the cavern roof and walls.	An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilize. Monitoring of displacements is essential to confirm design predictions.
 Underground nuclear waste disposal.	Stress and/or thermally induced spalling of the rock surrounding the excavations resulting in increased permeability and higher probability of radioactive leakage.	• Orientation, inclination, permeability and shear strength of structural features in the rock mass. • In situ and thermal stresses in the rock surrounding the excavations. • Groundwater distribution in the rock mass.	Numerical analyses are used to calculate stresses and displacements induced by excavation and by thermal loading from waste canisters. Groundwater flow patterns and velocities, particularly through blast damaged zones, fissures in the rock and shaft seals are calculated using numerical methods.	An acceptable design requires extremely low rates of groundwater movement through the waste canister containment area in order to limit transport of radioactive material. Shafts, tunnels and canister holes must remain stable for approximately 50 years to permit retrieval of waste if necessary.

Table 5: Failure modes, critical parameters, methods of analysis and acceptability criteria for underground mining excavations.

STRUCTURE	FAILURE MODE	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Pillars.	Progressive spalling and slabbing of the rock mass leading to eventual pillar collapse or rockbursting.	• Strength of the rock mass forming the pillars. • Presence of unfavourably oriented structural features. • Pillar geometry, particularly width to height ratio. • Overall mine geometry including extraction ratio.	For horizontally bedded deposits, pillar strength from empirical relationships based upon width to height ratios and average pillar stress based on tributary area calculations are compared to give a factor of safety. For more complex mining geometry, numerical analyses including progressive pillar failure may be required.	Factor of safety for simple pillar layouts in horizontally bedded deposits should exceed 1.6 for "permanent" pillars. In cases where progressive failure of complex pillar layouts is modelled, individual pillar failures can be tolerated provided that they do not initiate "domino" failure of adjacent pillars.
 Crown pillars.	Caving of surface crown pillars for which the ratio of pillar depth to stope span is inadequate. Rockbursting or gradual spalling of overstressed internal crown pillars.	• Strength of the rock mass forming the pillars. • Depth of weathering and presence of steeply dipping structural features in the case of surface crown pillars. • In situ stress levels and geometry of internal crown pillars.	Rock mass classification and limit equilibrium analyses can give useful guidance on surface crown pillar dimensions for different rock masses. Numerical analyses, including discrete element studies, can give approximate stress levels and indications of zones of potential failure.	Surface crown pillar depth to span ratio should be large enough to ensure very low probability of failure. Internal crown pillars may require extensive support to ensure stability during mining of adjacent stopes. Careful planning of mining sequence may be necessary to avoid high stress levels and rockburst problems.
 Cut and fill stopes.	Falls of structurally defined wedges and blocks from stope backs and hanging walls. Stress induced failures and rockbursting in high stress environments.	• Orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Shape and orientation of stope. • Quality, placement and drainage of fill.	Numerical analyses of stresses and displacements for each excavation stage will give some indication of potential problems. Some of the more sophisticated numerical models will permit inclusion of the support provided by fill or the reinforcement of the rock by means of grouted cables.	Local instability should be controlled by the installation of rockbolts or grouted cables to improve safety and to minimize dilution. Overall stability is controlled by the geometry and excavation sequence of the stopes and the quality and sequence of filling. Acceptable mining conditions are achieved when all the ore is recovered safely.
 Non-entry stopes.	Ore dilution resulting from rockfalls from stope back and walls. Rockbursting or progressive failure induced by high stresses in pillars between stopes.	• Quality and strength of the rock. • In situ and induced stresses in the rock surrounding the excavations. • Quality of drilling and blasting in excavation of the stope.	Some empirical rules, based on rock mass classification, are available for estimating safe stope dimensions. Numerical analyses of stope layout and mining sequence, using three-dimensional analyses for complex orebody shapes, will provide indications of potential problems and estimates of support requirements.	A design of this type can be considered acceptable when safe and low cost recovery of a large proportion of the orebody has been achieved. Rockfalls in shafts and haulages are an unacceptable safety hazard and pattern support may be required. In high stress environments, local destressing may be used to reduce rockbursting.
 Drawpoints and orepasses.	Local rock mass failure resulting from abrasion and wear of poorly supported drawpoints or orepasses. In extreme cases this may lead to loss of stopes or orepasses.	• Quality and strength of the rock. • In situ and induced stresses and stress changes in the rock surrounding the excavations. • Selection and installation sequence of support.	Limit equilibrium or numerical analyses are not particularly useful since the processes of wear and abrasion are not included in these models. Empirical designs based upon previous experience or trial and error methods are generally used.	The shape of the opening should be maintained for the design life of the drawpoint or orepass. Loss of control can result in serious dilution of the ore or abandonment of the excavation. Wear resistant flexible reinforcement such as grouted cables, installed during excavation of the opening, may be successful in controlling instability.

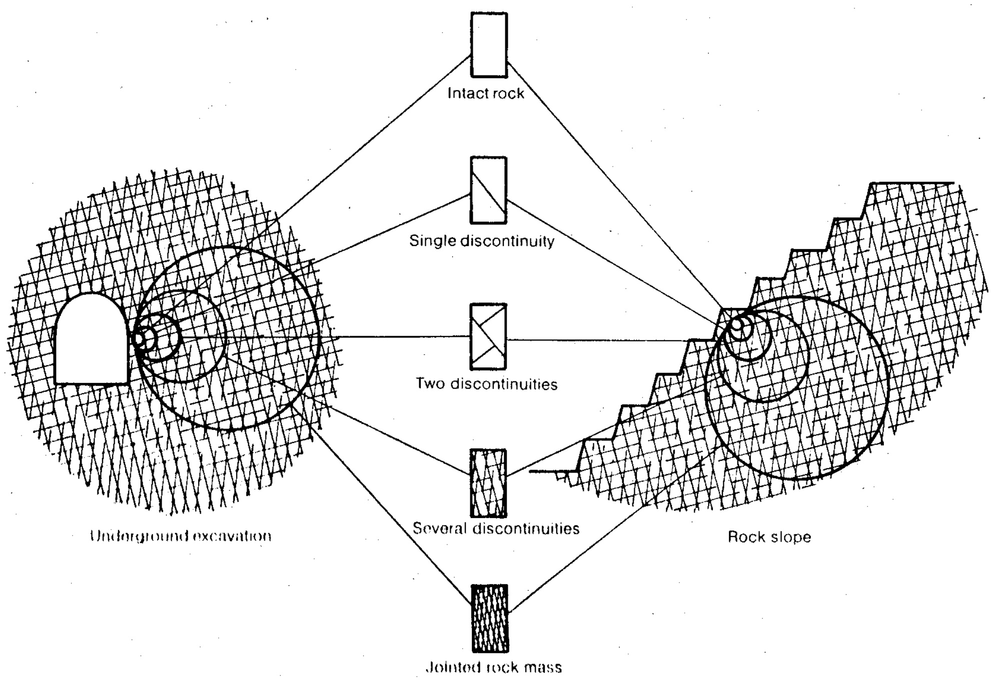


Figure 1. Effect of scale on the type of rock mass behavior model which should be used in designing underground excavations and slopes (after Hoek, 1983).

6. ROCK AND ROCK MASS CLASSIFICATIONS

In general, rock mass classifications can be divided into two categories: geological and engineering.

6.1 Geological Classifications

Geologists classify rocks according to how they are formed. The geological classification is therefore a genetic one. Rocks are usually separated into three groups:

- *Sedimentary rocks*: rocks formed by the accumulation, compaction and cementation of pieces of other rocks and possible organic debris;

- *Igneous rocks*: rocks formed by the solidification of a hot molten rock called magma. They consist of a complex crystalline assemblage of minerals such as quartz, feldspars, micas, pyroxenes, amphibole and olivine;

- *Metamorphic rocks*: rocks that are formed by transformation of existing rocks by the action of temperature and pressure.

In each group, subgroups are further defined according to mineral composition and *texture*. The latter refers to the shape, size of the rock constituents and how these constituents are assembled together (clastic, crystalline, porphyric, pegmatitic, etc...).

In general, the name of a rock is not sufficient to narrow down its engineering properties (see for instance Hatheway and Kiersch, 1989). It is not because we have a limestone or a granite that we have a strong rock. Information supplied by geologists to engineers may appear at a first glance of limited value. This is not so in geological engineering where geological data are of prime importance. The cooperative work between geologists, engineering geologists and engineers can help engineers in deriving useful engineering conclusions in the early stage of a project. Long term problems and costly remedial actions can be avoided.

The lithological name of a rock gives a range within which the engineering properties of the intact rock should fall. This can be useful in the preliminary design stage where test data are not yet available and preliminary decisions need to be made. Most textbooks in rock mechanics and engineering geology have tables of compressive strength, Young's modulus, etc.. for various rock types (Goodman, 1989; U.S. Bureau of Reclamation, 1953).

The lithological name, the age, the texture and the *fabric* of a rock (i.e the uniformity of the texture within the rock) can provide qualitative information on its engineering properties. For instance, rocks with a crystalline texture consists of highly interlocked crystals of silicates, carbonates or sulfates. They are usually strong, elastic and brittle when unweathered (fresh). Carbonates and sulfates may show a ductile behavior rather than a brittle behavior at medium to high temperatures and confining levels. The engineering properties of rocks with clastic textures will depend on the relative proportion of particles and cement and durability of the cement. For instance, a poorly cemented sandstone will certainly show a lower strength, higher deformability and weatherability than a highly cemented one. Foliation and bedding planes result in highly directional (anisotropic) rock strength and deformability. Finer rock grain size leads to higher fracture strength. Older rocks that have been buried at larger depths will show more compaction and smaller porosities. Rocks with fine textures and rich in silica can be expected to be more abrasive and wear drilling bits and machine cutters.

The petrographic description of the rock may give some information about minerals that could create some engineering problems such as gypsum, montmorillonite, chert, feldspars, asbestos, etc..

The lithological name may be associated with specific features that could cause engineering problems such as karsts in limestone formations or columnar jointing in basalt. Clay bearing rocks are expected to be very sensitive to water and weathering and to be susceptible to slaking and swelling. The combination of more than one rock can give properties much worse than each rock alone; a good example would be a sedimentary rock mass consisting of multiple layers.

Finally, field observations can give information about the degree of rock mass fracturing and weathering. How much rock needs to be excavated to reach the fresh rock? Is rock mass fracturing going to create block stability problems or seepage problems?

6.2 Engineering Classifications

Compared to the rock classification used by geologists, the purpose of an engineering classification of rocks is to group rocks and rock masses with similar engineering properties. There is no such thing as one universal engineering classification of rocks. For such a classification to exist, it would have (i) to be simple and meaningful in terminology, (ii) to be based on parameters that can be measured rapidly and inexpensively, (iii) to be functional for general use in solving the whole variety of engineering problems related to rock engineering, and (iv) to be exact enough to yield quantitative data that can readily be applied in engineering design. In general, it is not possible to find a classification that meets all four requirements. Instead, several engineering classifications of rocks have been proposed in the literature for more specific rock activities such as tunneling, mining, slopes, foundations, blasting, drillability, boreability, cuttability, and weatherability. Some of these classifications involve intact rock only whereas others involve the properties of the rock mass. Table 6 gives a list of major rock mass classifications currently in use.

Name of classification	*Originator and date*	*Country of origin*	*Applications*
Rock loads	Terzaghi, 1946	USA	Tunnels with steel support
Stand-up time	Lauffer, 1958	Austria	Tunneling
NATM	Rabcewicz, Pacher and Müller, 1964	Austria	Tunneling
Rock quality designation	Deere, 1967	USA	Core logging, tunneling
RSR concept	Wickham *et al.*, 1972	USA	Tunneling
RMR system (Geomechanics Classification)	Bieniawski, 1973 (last modified, 1979, USA)	South Africa	Tunnels, mines, slopes, foundations
RMR system extensions	Laubscher, 1977	South Africa	Mining
	Ghose and Raju, 1981	India	Coal mining
	Kendorski *et al.*, 1983	USA	Hard rock mining
	Serafim and Pereira, 1983	Portugal	Foundations
	Gonzales de Vallejo, 1983	Spain	Tunneling
	Unal, 1983	USA	Roof bolting/coal
	Romana, 1985	Spain	Slope stability
	Newman, 1985	USA	Coal mining
	Venkateswarlu, 1986	India	Coal mining
	Robertson, 1988	Canada	Slope stability
Q system	Barton *et al.*, 1974	Norway	Tunnels, chambers
Strength–Size	Franklin, 1975	Canada	Tunneling
Basic geotechnical description	International Society for Rock Mechanics, 1981		General, communication

Table 6. Major rock mass classification systems in use (after Bieniawski, 1993).

Rock masses are so complex that very often it is not possible to take under consideration in the design all possible stability problems related to intact rock, discontinuities, stresses and water. As an example, Figure 2 shows the decision process in the design of underground excavations in rock.

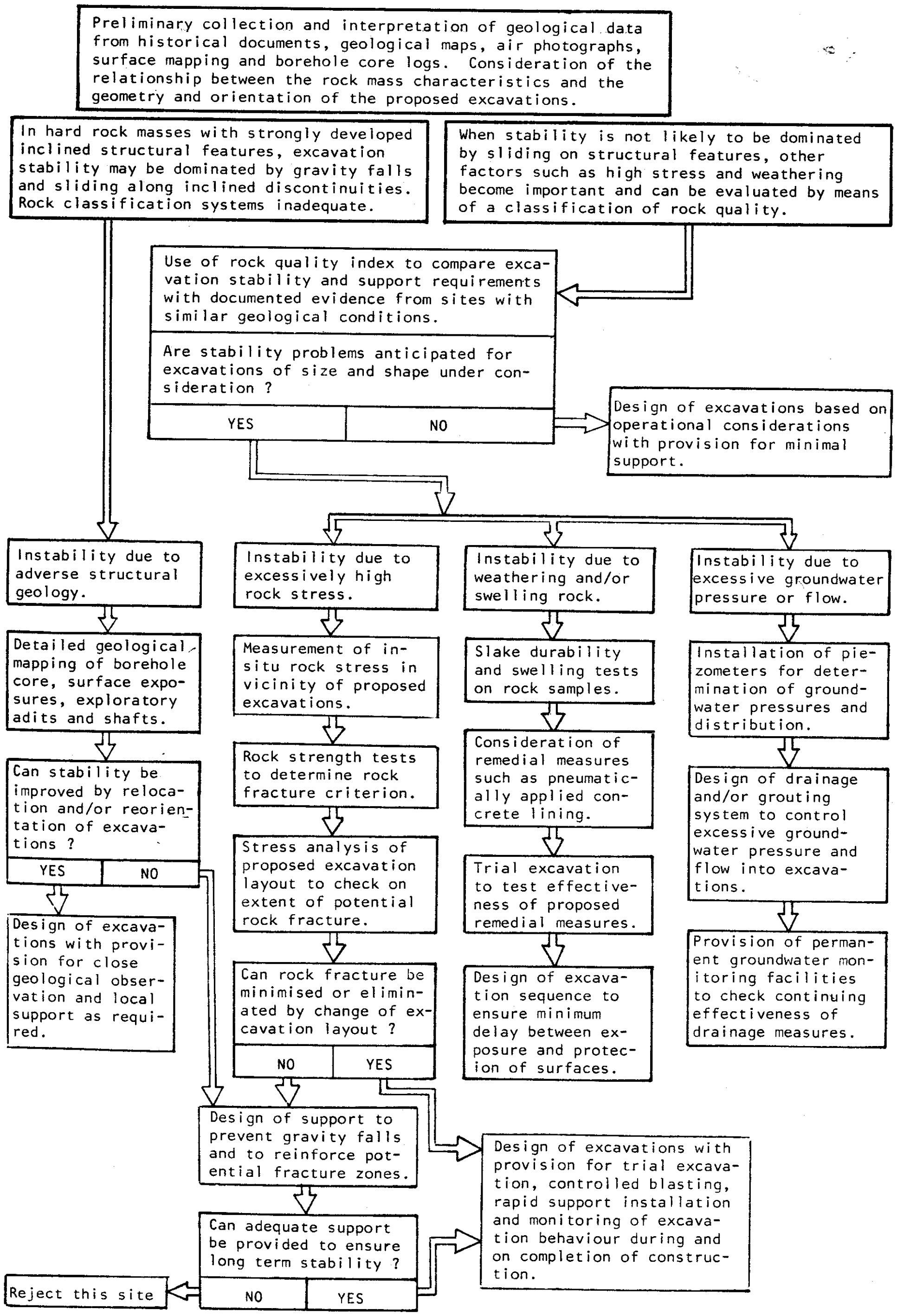


Figure 2. Design of underground excavations in rock (after Hoek and Brown, 1980a).

Due to the complex nature of rock masses, rock engineers sometimes use rock mass classifications for the design of rock slopes and underground excavations. Classifications can be seen as "black boxes" that contain a lot of case histories. The conditions anticipated at a proposed site are compared with experience gained on other completed projects.

In general, rock mass classifications form the backbone of the "empirical design" approach in rock engineering and are used quite extensively for the design of rock slopes and underground excavations in rock. They should not be used as the sole design tool but instead along with other analytical, numerical, and observational design methods.

Before using rock mass classifications for design it is important that the rock mass be divided into a number of structural regions, i.e regions where the rock mass has more or less uniform features such as rock type, discontinuity network, fabric, etc.. Very often, the boundaries of structural regions coincide with major geological features such as faults, dykes, shear zones, etc. The empirical design is then conducted on each structural region.

Major classification systems used for underground excavation design include those of Terzaghi (1946) (revised by Rose in 1982), Deere et al. (1970), Wickham et al. (1974), Bieniawski (1974), Barton et al. (1974, 1992), and Barton (1994). The classifications of Bieniawski and Barton are by far the two empirical methods most commonly used in rock engineering design around the world. The classification of Bieniawski is briefly discussed below (see also Appendix).

6.3 Classification of Bieniawski

The classification of Bieniawski (also known as the Geomechanics Classification system) consists of rating the importance of several intact and rock mass properties, separately. Detailed description of the rock mass is therefore necessary (see Table 1 in Appendix). A *Rock Mass Rating* coefficient called *RMR* is introduced and consists of the sum of six separate ratings as illustrated in Table 2 in the Appendix. The ratings are as follows:

- Rating R_1 is related to the intact rock strength;
- Rating R_2 is related to the *Rock Quality Designation* (*RQD*) index which itself depends on the frequency of the rock mass fractures (see Deere et al., 1988);
- Rating R_3 is related to the fracture spacing;
- Rating R_4 refers to the conditions of the joints (roughness, openness, filling and continuity);
- Rating R_5 is related to the ground water conditions;
- Rating R_6 is a negative rating that depends on the orientation of the discontinuities with respect to the tunnel section of interest.

The total rating, called *RMR*, varies between 0 and 100%. For underground excavations, correction factors can be used to account for blast damage, *in situ* stress conditions, and major faults and fractures (see Figure 1 in Appendix). Depending on the final rating, five rock mass classes can be defined from class I (very good) to class V (very poor). For each class, Table 2 in the Appendix gives estimates of the cohesive and frictional strength of the rock mass.

Rock mass ratings are available for foundation, slopes and mining applications. Several examples of application of the *RMR* rating in practice can be found in Bieniawski (1974, 1975, 1979a,b, 1984, 1993), Einstein et al. (1983), Kaiser et al. (1986), ASTM STP 984 (1988), and Romana (1993). You should be aware that since *RMR* was first proposed in 1974, the individual ratings in the determination of *RMR* have changed with time as more case histories have been included in the corresponding "black box". Always use the most current set of rating coefficients.

Of practical importance, empirical equations have been proposed to determine the modulus of deformation and the strength of rock masses once *RMR* is calculated. This is discussed more extensively in Bieniawski (1993). Rock mass deformability and strength are very important parameters in rock engineering design. They can be determined by conducting field tests (which are expensive and time-consuming) or empirically by using empirical equations. For instance, the modulus of deformation E_M (in GPa) of a rock mass is related to *RMR* as follows

$$E_M = 2RMR - 100 \tag{1}$$

if *RMR* > 50% and

$$E_M = 10^{(RMR-10)/40} \tag{2}$$

if *RMR* < 50%. The strength of a rock mass can be determined using the failure criterion of Hoek and Brown (1980b) which relates the major and minor principal stresses at failure as follows

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_c + s\sigma_c^2} \tag{3}$$

where m and s are two parameters that depend on the rock type, the degree of rock mass fracturing, and the *RMR*. For intact rock $m = m_i$ and $s = 1$. For fractured rock masses, m and s are related to the basic (unadjusted) *RMR* as follows

$$\begin{aligned} m &= m_i \exp[(RMR-100)/28] \\ s &= \exp[(RMR-100)/9] \end{aligned} \tag{4}$$

for smooth-blasted or machine-bored excavations in rock and

$$m = m_i \exp[(RMR - 100)/14]$$
$$s = \exp[(RMR - 100)/6] \qquad (5)$$

for slopes and blast-damaged excavations in rock. Typical values of m and s for various rock types and corresponding to various RMR values are listed in Table 7, which is a modified version of that originally proposed by Hoek and Brown (1980a).

A correlation has been proposed between the RMR and the Q rating proposed by Barton (1974). An example is shown in Figure 11 in the Appendix where the two ratings are related as follows

$$RMR = 9\ln Q + 44 \qquad (7)$$

Other empirical equations between RMR and Q have been proposed (see for instance Kaiser et al., 1986 and Bieniawski, 1993).

Finally, as remarked by Bieniawski (1993), "rock mass classifications were never intended as the ultimate solution to design problems, but only as a means toward this end". Although rock mass classifications yield conservative design recommendations, they can be powerful aids in rock engineering as long as the database on which they are developed is good. They should be used along with other analytical and observational methods. Also, it is suggested to use several rock mass classifications schemes on a given project. Ultimately, the final design recommendations are left to the engineer's judgment.

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Empirical failure criterion $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$ σ_1 = major principal effective stress σ_3 = minor principal effective stress σ_c = uniaxial compressive strength of intact rock and m and s are empirical constants		*Carbonate rocks with well-developed crystal cleavage: dolomite, limestone and marble*	*Lithified argillaceous rocks: mudstone, siltstone, shale and slate (normal to cleavage)*	*Arenaceous rocks with strong crystals and poorly developed crystal cleavage: sandstone and quartzite*	*Fine-grained polymineralic igneous crystalline rocks: andesite, dolerite, diabase and rhyolite*	*Coarse-grained polymineralic igneous and metamorphic crystalline rocks: amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*
Intact rock samples:						
laboratory size specimens free	m^a	7.00	10.00	15.00	17.00	25.00
from discontinuities.	s^a	1.00	1.00	1.00	1.00	1.00
RMR = 100, Q = 500	m^b	7.00	10.00	15.00	17.00	25.00
	s^b	1.00	1.00	1.00	1.00	1.00
Very good quality rock mass:						
tightly interlocking,	m	2.40	3.43	5.14	5.82	8.56
undisturbed rock with unweathered	s	0.082	0.082	0.082	0.082	0.082
joints at 1–3 m. RMR = 85,	m	4.10	5.85	8.78	9.95	14.63
Q = 100	s	0.189	0.189	0.189	0.189	0.189
Good quality rock mass:						
fresh to slightly weathered	m	0.575	0.821	1.231	1.395	2.052
rock, slightly disturbed with	s	0.00293	0.00293	0.00293	0.00293	0.00293
joints ar 1–3 m. RMR = 65,	m	2.006	2.865	4.298	4.871	7.163
Q = 10	s	0.0205	0.0205	0.0205	0.0205	0.0205
Fair quality rock mass:						
several sets of moderately	m	0.128	0.183	0.275	0.311	0.458
weathered joints spaced	s	0.00009	0.00009	0.00009	0.00009	0.00009
at 0.3–1 m. RMR = 4, Q = 1	m	0.947	1.353	2.030	2.301	3.383
	s	0.00198	0.00198	0.00198	0.00198	0.00198
Poor quality rock mass:						
numerous weathered joints	m	0.029	0.041	0.061	0.069	0.102
at 30–500 mm, some gouge.	s	0.000003	0.000003	0.000003	0.000003	0.000003
Clean compacted waste rock.	m	0.447	0.639	0.959	1.087	1.598
RMR = 23, Q = 0.1	s	0.00019	0.00019	0.00019	0.00019	0.00019
Very poor quality rock mass:						
numerous heavily weathered	m	0.007	0.010	0.015	0.017	0.025
joints spaced < 50 mm with	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
gouge. Waste rock with fines.	m	0.219	0.313	0.469	0.532	0.782
RMR = 3, Q = 0.01	s	0.00002	0.00002	0.00002	0.00002	0.00002

[a]First stated m and s values are for disturbed rock mass. [b]Second stated m and s values are for undisturbed rock mass.

Table 7. Approximate relationship between rock mass quality and material constants (after Bieniawski, 1993).

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