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Discontinuous

Rock Mechanics

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Summary

Rock mechanics for engineering geology and civil engineering purposes is mostly governed by the discontinuities in the rock mass. These lecture notes therefore describe discontinuities and the behavior of rock such that the behavior of a rock mass becomes clear. Civil engineering applications are used to illustrate what consequences can be expected of a civil engineering use of the rock mass and what mechanisms govern the interaction between the rock mass and the civil engineering application.

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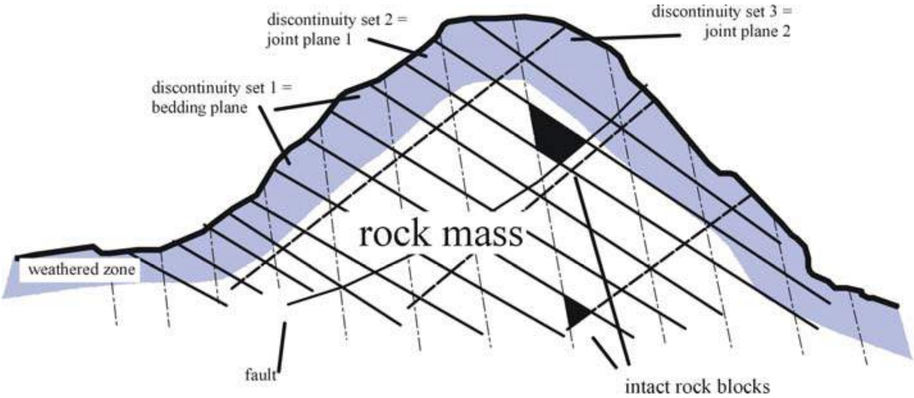

Fig. A-1. Intact rock vs rock mass

In the practice of constructing engineering structures, such as buildings, tunnels and slopes, an interaction takes place between the 'ground' and the engineering structure. The knowledge of the consequences of the influence of the 'ground' on the engineering structure and vice versa are often critical for the economic and safe design of an engineering structure. In particular, the mechanical response of the 'ground' under influence of the engineering structure should be known before an engineering structure is built. 'Ground' is a very broad term. The 'ground' is any natural material present at the site where the engineering structure is to be built on or in. 'Ground' is normally divided in 'soil' and 'rock'. 'Soil' consists of loose particles not cemented together whereas the particles in rock are cemented together, resulting in a tensile strength. This difference in characteristics between 'soil' and 'rock' has also resulted in the development of different methodologies for the calculation of the mechanical behavior of the 'soil' or 'rock'. Most 'rocks' are not continuous, but contain fractures, faults, bedding planes or more general: 'discontinuity[1] planes' that divide the 'rock' into blocks of rock bounded by discontinuities. The whole array of blocks of rocks and discontinuity planes is then designated the 'rock mass' or 'discontinuous rock mass' (Fig. A-1).

A.1.1 Discontinuous rock masses

In the last decades, the study of discontinuous rock mechanics has developed tremendously. For constructions, such as slopes, foundations, and shallow tunnels it has been recognized that discontinuities have a major influence on the mechanical properties of a rock mass. This perception has major consequences for the assessment of the engineering behavior of a rock mass. Descriptions and characterizations, engineering geological maps and calculations for engineering structures in or on a rock mass have to include discontinuity properties. Variations in properties, however, can be considerable along the same discontinuity plane. As there may be hundreds of discontinuities in a rock mass, each with its own variable properties, these, taken together with inhomogeneities in the rock material, require that in order to describe or calculate the mechanical behavior of the rock mass accurately, a large amount of data is required. Laboratory and field tests are available to obtain discontinuity properties. Testing in large quantities is, however, time consuming and troublesome. Analytical or numerical calculations are only in some occasions a realistic solution for the design of an engineering structure in or on a rock mass. Alternatively, empirical, e.g. classification systems are often used to design engineering works. Empirical relations however, are only applicable in circumstances for which the relation has been developed.

[1] The terms discontinuous rock mass and discontinuity are used in a rock mechanical sense. A discontinuity is a plane that marks an interruption in the continuity and normally has low or zero tensile strength. A discontinuous rock mass is a rock mass containing discontinuities.

Recognizing which, where and for what a particular design methodology may be used requires a proper understanding of the behavior of rock masses and the interactions between the engineering structure and the rock mass. Therefore, these lecture notes concentrate on describing the mechanical behavior of rock masses, and show with examples the interactions between engineering structures and the rock mass. Material behavior of intact rock is often of limited importance and is described in these notes only as far as necessary to understand the behavior of a rock mass. In ch. A a brief introduction to discontinuous rock masses is given. This is further detailed in the following chapters B through G. In chs I and J applications in underground excavations are given, and in chs M and O methodologies to design in rock masses are given.

A.2 Intact rock versus rock mass

A rock mass may consist of intact rock only, but is more commonly formed from an array of intact rock blocks with boundaries formed by discontinuities (). Within the rock mass, the mechanical properties of both the intact rock blocks and the discontinuities may be inhomogeneous and anisotropic. A common relation between rock, rock mass and engineering is (Price, 1984):

$$material\ properties + mass\ fabric = mass\ properties$$

$$mass\ properties + environment = the\ engineering\ geological\ matrix$$

$$\frac{the\ engineering\ geological\ matrix}{changes\ produced\ by\ the\ engineering\ work} = the\ engineering\ behaviour\ of\ the\ ground$$

Exact descriptions of rock material and rock mass are required for understanding rock masses and follow below.

A.2.1 Rock mass components

Intact rock material

Intact rock blocks are blocks of rock that do not contain mechanical discontinuities and do have tensile strength.

Discontinuities

Fig. A-2. Bedding planes can be discontinuities

A discontinuity is a plane or surface that marks a change in physical or chemical characteristics in rock material (Fig. A-2). A division is made between integral discontinuities and mechanical discontinuities. The latter are planes of physical weakness. Bedding planes, joints, fractures, faults, etc. are mechanical discontinuities if the tensile strength perpendicular to the discontinuity or the shear strength along the discontinuity is lower than of the surrounding rock material (ISRM, 1978b, 1981a). Integral discontinuities are discontinuities that are as strong as the surrounding rock material. Integral discontinuities can change into mechanical discontinuities due to weathering or chemical reactions that change the mechanical characteristics. ‘Discontinuities’ denote mechanical discontinuities throughout these notes except where stated otherwise.

Discontinuity set

Discontinuities exist as single features (fault, isolated joint or fracture, etc.) and as discontinuity sets or families (bedding planes, schistosity, cleavage, joints, etc.). Various geological processes create discontinuities at a broadly regular spacing. For example, bedding planes are the result of a repeated sedimentation cycle with a change of sedimentation material at regular intervals, folding creates joints at regular separations to allow for shrinkage or expansion of the rock material, etc. Normally discontinuities with the same origin have broadly the same characteristics in terms of roughness, infill, etc. The orientations of discontinuities with the same origin are related to the process that has created them and to the geological history of the rock mass. A set denotes a series of discontinuities for which the geological origin (history, etc.), the orientation, spacing, and the mechanical characteristics (friction angle, roughness, infill material, etc.) are broadly the same. In some circumstances, a discontinuity is treated as a single discontinuity although it belongs to a discontinuity set, in particular if the spacing is very wide compared to the size of the engineering application or to the size of the geotechnical unit.

Inhomogeneity

Inhomogeneity is the spatial variation of rock material or rock mass properties. For example, an intact rock strength variation within a block of intact rock material causes the intact rock material to be inhomogeneous; a variation in the orientation of discontinuities causes a rock mass to be inhomogeneous. Generally, it is taken that inhomogeneity results in new boundaries in the rock mass. This is not a discontinuity boundary but a boundary defined by a change in intact rock material or rock mass properties. Normally this boundary will coincide with a geotechnical unit boundary. Similarly, a gradual change in the orientation of a discontinuity set causes a rock mass to be inhomogeneous, also leading to an arbitrarily established geotechnical unit boundary.

Anisotropy

An isotropic body has equal properties in all directions. Discontinuities in a rock mass induce anisotropy. A simple case of anisotropy is illustrated in Fig. A-3. The rock is regularly intersected by series of discontinuities (‘d’) filled with a material different from the rock material (‘i’) between the discontinuities. The properties in any direction in the xy-plane are of rock material or of the infill material. Properties in the z direction depend on the combination of the properties of the rock and the infill material. The total block including discontinuities will have, as a result, different properties in different directions.

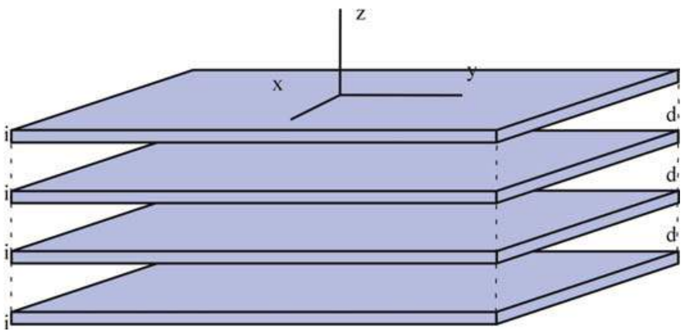

Fig. A-3. Anisotropic rock mass

Rock mass

A rock mass is an assemblage of rock blocks with discontinuities, with or without inhomogeneity and with anisotropy. The overall effect of discontinuities is that a rock mass that contains discontinuities is weaker than the intact rock because shear and tensile strengths of the discontinuities are lower than those of the intact rock material are. A rock mass containing discontinuities will be more deformable than intact rock. Such deformation will normally take place by relative movement along discontinuities and be plastic rather than elastic. The tensile strength of a rock mass containing discontinuities is low and for many rock masses zero. The porosity of a discontinuous rock mass is higher due to the storage capacity of the discontinuities and the permeability is often considerably higher due to the conductivity via the discontinuities. Discontinuities always lead to an anisotropic behavior of the rock mass and all rock mass properties, such as deformability, permeability, etc. Therefore a discontinuous rock mass is a three-dimensional feature that is anisotropic in three dimensions.

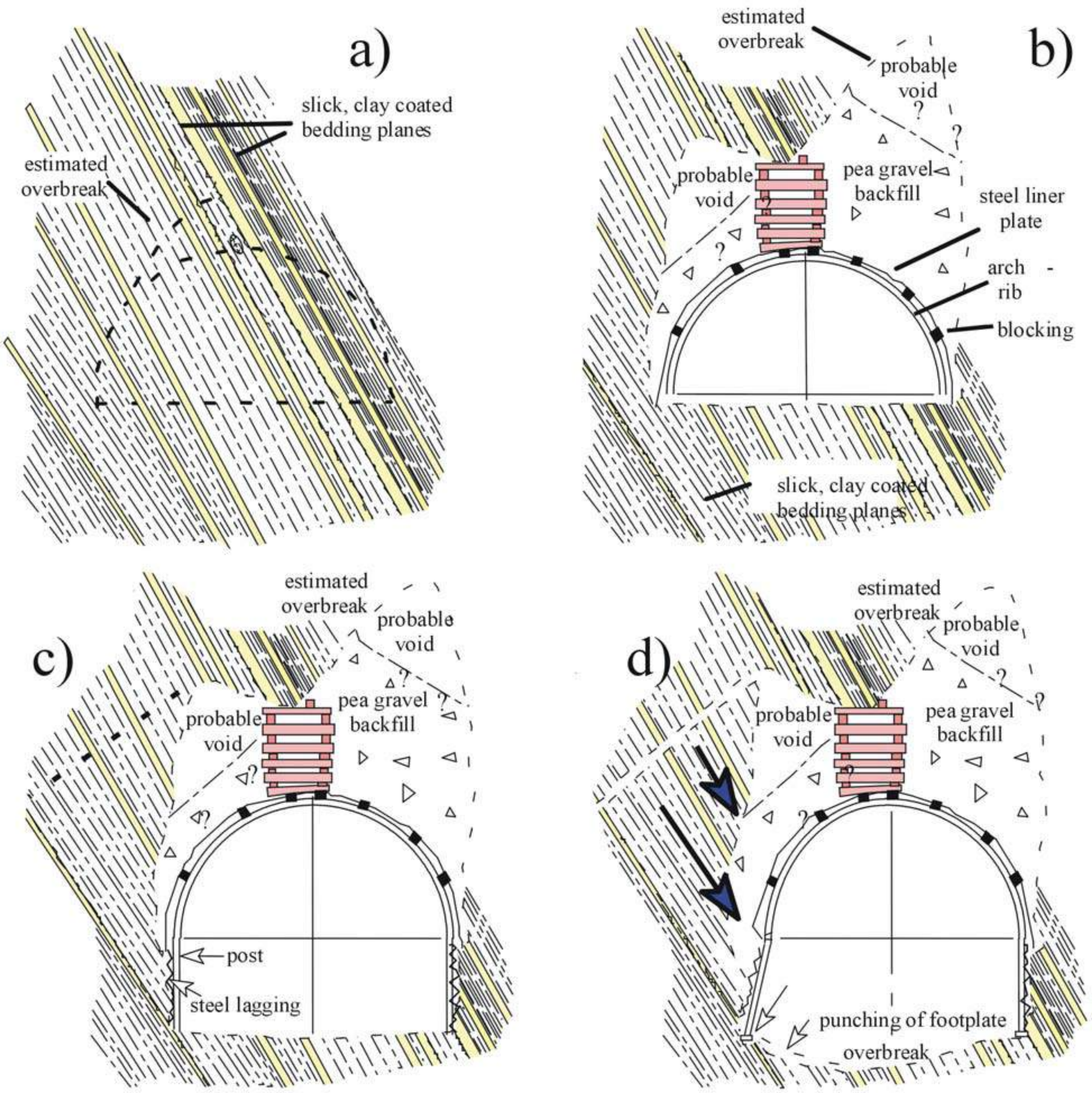

Fig. A-4. The influence of discontinuities on the stability of a tunnel in the progress of construction (after Arnold et al., 1972)

A classical example of the influence of discontinuities in a rock mass on the stability of a tunnel is illustrated in Fig. A-4. During excavation of the diversion tunnel for the Castaic dam (40 miles north of Los Angeles, USA) an overbreak occurred. The overbreak was improperly backfilled, which allowed de-stressing of the rock mass around the tunnel. By de-stressing the rock mass, the clay-lined bedding plane on the left of the tunnel was de-stressed in the direction normal to the plane resulting in lower shear strength along the bedding plane. This allowed movement of part of the rock mass in the direction of the tunnel, destroying the support and resulting in a complete collapse of the tunnel.

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A.2.2 Geotechnical units

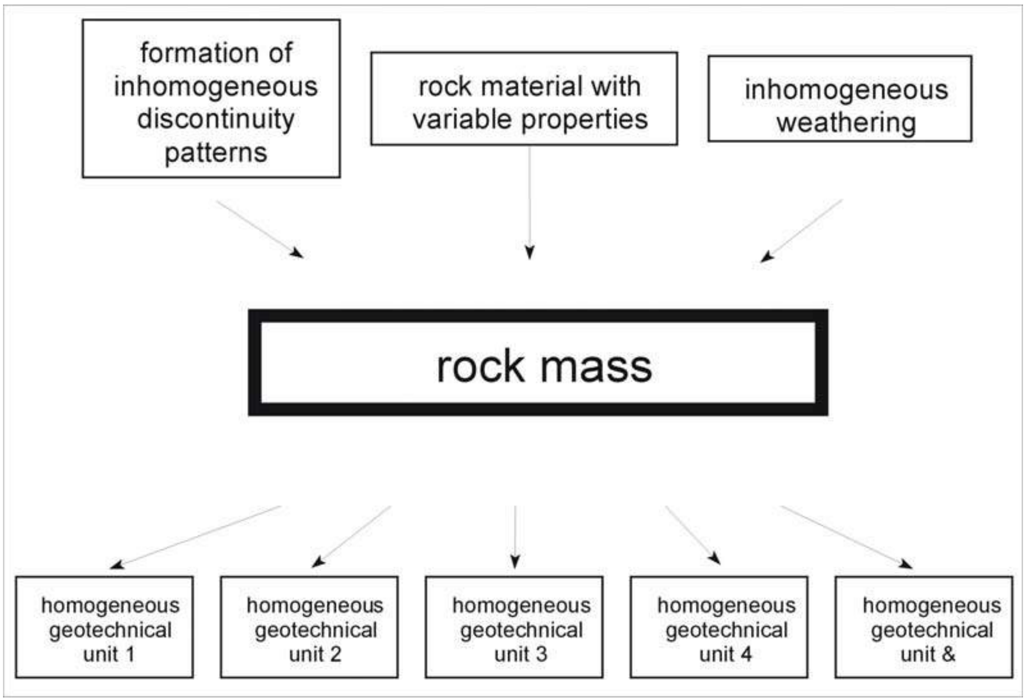


Fig. A-5. Rock mass components (after Hack, 1998)

Theoretically, a proper description or geotechnical calculation to determine the behavior of a rock mass and engineering structure in or on the rock mass should include all properties in a rock mass including all spatial variations of the properties. This would be unrealistic and is not possible without disassembling the rock mass. Therefore, a standard procedure is to divide a rock mass into homogeneous geotechnical units. A geotechnical unit is, in theory, a part of the rock mass in which the mechanical properties of the intact rock material are uniform and the mechanical properties of the discontinuities (including anisotropy of properties) within each set of discontinuities are the same. The anisotropy of properties in a geotechnical unit should be also uniform. This additional condition is not always specified in the literature; however, in engineering it is an obvious requirement because of the large influence of anisotropic features (e.g. discontinuities, etc.) on engineering.

Fig. A-5 shows a schematic visualization of a rock mass and its division in geotechnical units. In practice, homogeneity is seldom found and material and discontinuity properties vary within a selected range of values within the unit. The allowable variation of the properties within one geotechnical unit depends on: 1) the degree of variability of the properties within a rock mass, and 2) the context in which the geotechnical unit is used.

A rock mass containing a large variation of properties over a small distance necessarily results in geotechnical units containing larger variations in properties. This is because it is impossible to establish with sufficient accuracy all boundaries between the various areas with different properties within the rock mass. The smaller the allowed variability of the properties in a geotechnical unit the more accurate the geotechnical calculations can be. Smaller variability of the properties of the geotechnical units involves, however, collecting more data and is thus more costly. The higher accuracy obtained for a calculation based on more data has, therefore, to be balanced against the economic and environmental value of the engineering structure to be built and the possible risks for the engineering structure, environment, or human life. The variations allowed within a geotechnical unit for the foundation of a highly sensitive engineering structure (for example, a nuclear power station) will be smaller than for a geotechnical unit in a calculation for the foundation of a standard house.

Fig. A-6. Different geotechnical units present in a single slope. Greenish and bluish gray layers consist of calcareous shale and brownish, pinkish off-white layers consist of dolomite and limestone.

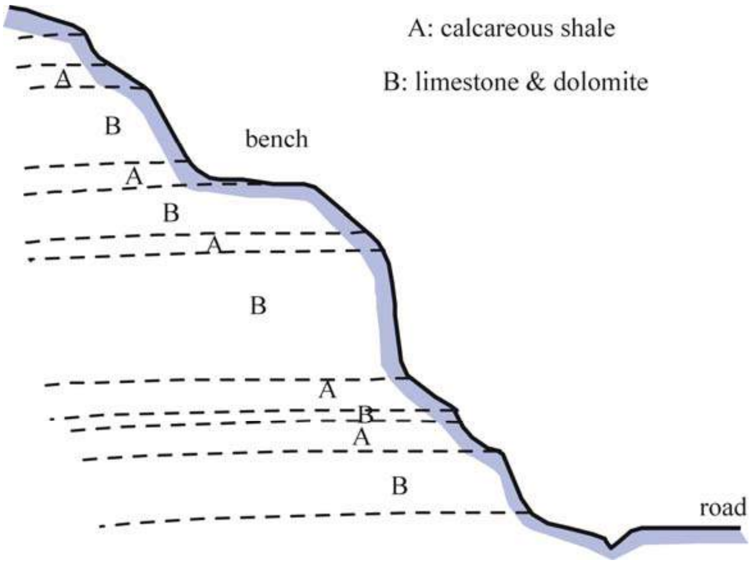


Fig. A-7. Section through the slope of Fig. A-6

No standard rules are available for the division of the rock mass into geotechnical units and this transformation depends on experience and 'engineering judgment'. However, features such as changes in lithology, faults, shear zones, etc., are often the boundaries of a geotechnical unit. In Fig. A-6 and Fig. A-7, a slope is shown in which different geotechnical units are present. The influence of the different geotechnical units on the form of the slope is clearly visible through the changes in slope surface steepness.

A.2.3 Water

Water influences the mechanical characteristics of a rock mass. Water adds to the weight of the rock mass, acts as a lubricant in discontinuities, causes softening of some infill materials (e.g. clay), and water pressure in discontinuities reduces the shear strength along discontinuities and thus also the (yield-) strength of a rock mass. Therefore, it is necessary to consider whether water should be treated as part of the rock mass and hence, of the geotechnical units. In this respect, it must be noted that water is often not a continuous feature in time. Water can be present during and just after rainfall and absent during long dry periods. In addition, the engineering structure to be built might influence the presence of water (e.g. drainage around tunnels, saturation of the rock mass due to an impounded reservoir, etc.).

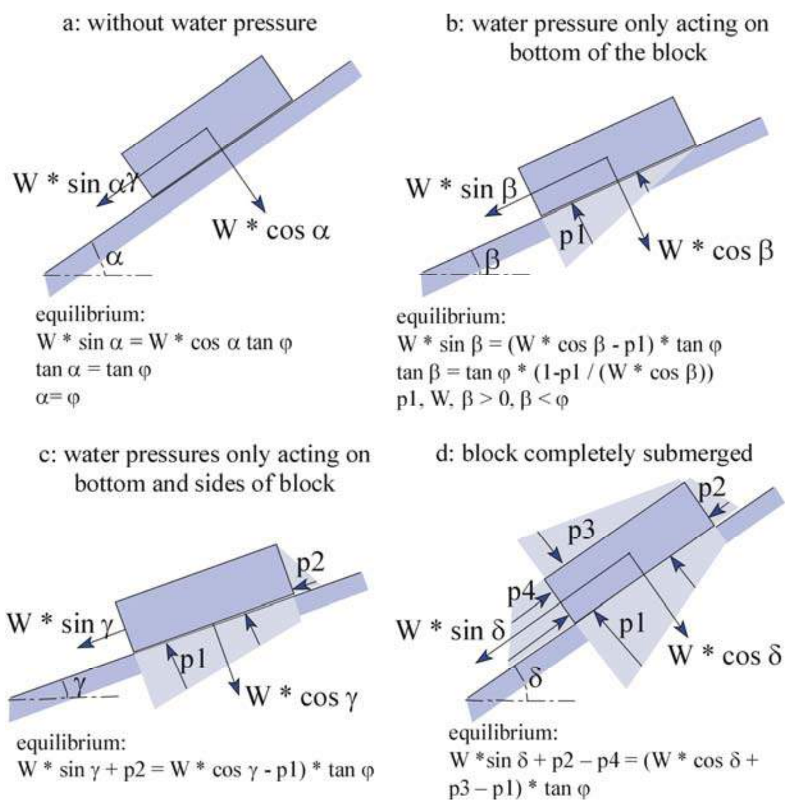


Fig. A-8. Block on discontinuities with and without water pressure (W is the weight of the block; cohesion along discontinuities is zero)

With time, most rock masses weather, a process strongly influenced by the presence of water, which causes the intact rock strength and the discontinuity strength parameters to decrease. To what extent weathering influences the mechanical behavior of a rock mass depends on the type of engineering application, type of intact rock material and discontinuity infill material, amount and chemistry of percolating water, etc.

Reduction of shear strength of discontinuities due to water

Water pressures in a discontinuity reduce the normal stress on the discontinuity and therefore reduce the shear strength along the discontinuity. Sliding over a discontinuity plane is then possible at a lower dip angle than over a discontinuity without water pressure (Fig. A-8a, b and c). In traditional limiting-equilibrium calculations for slope stability, water pressures in discontinuities are therefore a main reason for slope instability to occur (Hoek et al., 1981, Giani, 1992). In Fig. A-8a, b and c it can easily be seen that the discontinuity dip angle for which equilibrium exists decreases ($\alpha > \beta > \gamma$)[2].

Accordingly, because both effects (pressure and weathering) of the presence of water might or might not be present, water is not included in the rock mass or in the geotechnical unit. The influence of water, however, should be included in any calculation of the behavior of the geotechnical units.

A.2.4 Characteristics of intact rock and rock mass

A description of some geotechnical properties and characteristics of rock and rock mass is given hereafter. The properties and characteristics of rock masses are further detailed in the following chapters. For full

[2] If rock blocks are completely submerged in water (Fig. A-8d) the normal stress on the discontinuity is reduced (p1 > p3) causing a reduction in shear strength, but also the driving forces are reduced (p4 > p2). In a completely submerged slope the equilibrium between driving forces and shear strength is, therefore, less disturbed than in a situation with water pressures acting only on bottom and rear sides of the block (Fig. A-8b and c). In slopes the rock blocks near the surface of the slope are normally not completely submerged in water and therefore water pressures cause a reduction in normal stress along the discontinuity plane (Fig. A-8b) and driving forces may increase if a discontinuity at the rear of the block is filled by water (Fig. A-8c).

descriptions is also referred to the standard literature for further details (Giani, 1992, Goodman, 1989, Hoek et al., 1980, 1981, etc.).

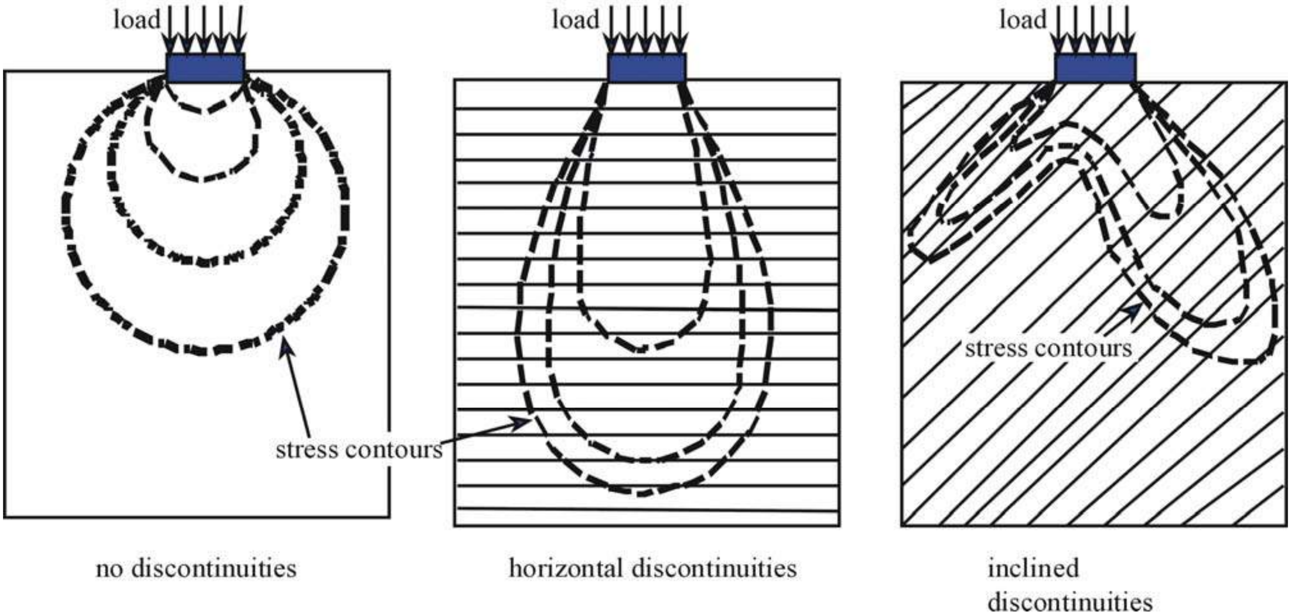


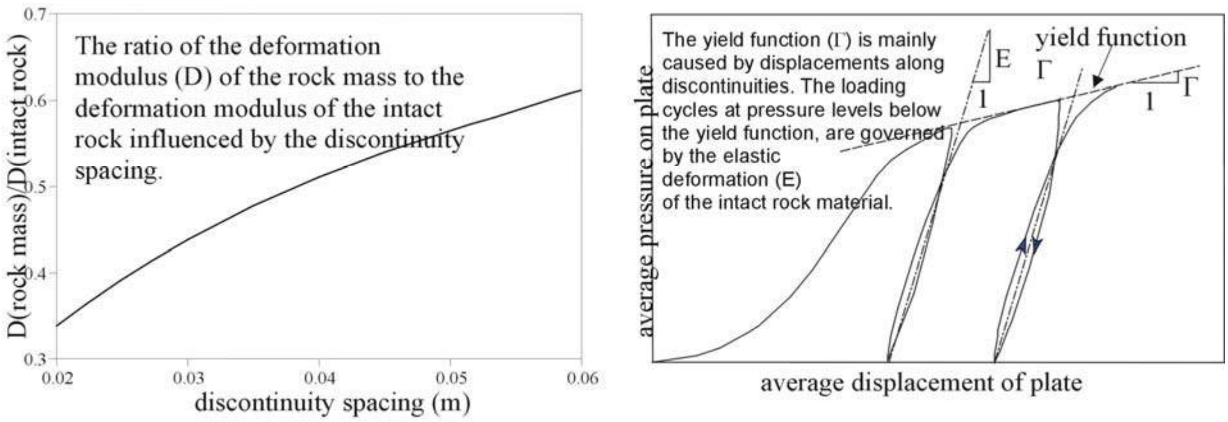
Fig. A-9. Stress distribution (bulbs of pressure lines of equal major principal stress) in a rock mass due to a vertically oriented plane load (after Gaziev et al., 1971)

Stress distribution in a rock mass

The stress distribution in a rock mass is strongly influenced by the presence of discontinuities. Fig. A-9 shows examples of a stress distribution in intact rock and in discontinuous rock masses. The figures clearly show the variation in the stress contours due to the presence and orientation of discontinuities.

Deformation

Deformation of intact rock is the change in volume or shape of intact rock under the influence of deforming loads. In general, the deformation of intact rock is partly elastic and partly plastic, and is time dependent (see also creep, below). Deformation of a rock mass is the change in volume or shape of the rock mass. The deformation is mainly caused by displacements of intact rock blocks along or perpendicular to discontinuities.



$D_{intact\ rock}/D_{max}$ vs. discontinuity spacing for plate diameter 8 cm on a model rock mass (after

Example of a cyclic plate-bearing test on fractured rock (after Schneider, 1967).

Fig. A-10 The influence of discontinuities on deformation of a rock mass

The discontinuities cause a dramatic change in deformation behavior of a rock mass in comparison to that of intact rock. The deformation in a rock mass is for a large part due to shear displacements along discontinuities or opening or closure of discontinuities. The shear deformations are non-elastic for larger displacements. The opening or closure of discontinuities is elastic or non-elastic depending on the infill material in the discontinuities and the discontinuity wall material, but usually the displacements are non-elastic (e.g. for a common infill material such as clay). Therefore, a rock mass shows mostly non-elastic deformation behavior. Fig. A-10 illustrates the influence of discontinuities on deformation of a rock mass and the resulting non-elastic deformation behavior of rock masses.

Rock mass failure

In general a rock mass does not fail and therefore failure of a rock mass is usually defined as the deformation of the rock mass larger than allowed for a particular engineering construction.

Compressive, tensile and shear strength of intact rock

Intact rock material has compressive[3], tensile and shear strength. Rock material consists of mineral grains completely or partially bonded together by cement or another bonding agency. If loaded to failure under a compressive, tensile or shear stress, intact rock material will break into smaller pieces of rock when the compressive, tensile or shear strength is reached ('the rock fails'). Intact rock strength behavior may be approximated with a 'Mohr-Coulomb failure criterion'[4]. This allows definition of the intact rock strength in terms of intact rock cohesion and intact rock friction.

Strength of a rock mass

The 'strength' of a rock mass, as often used in the literature or in day-to-day practice, is a confusing and false expression. A rock mass may be considered to have strength. However, due to the discontinuities in a rock mass, this strength is dependent on a variety of factors: the shape and size of the rock mass considered the environment (e.g. the engineering application, the confining stresses, etc.), the amount and the orientation of discontinuities and, although in many situations of minor importance, the intact rock strength. Consider the sketch in Fig. A-11. The rock mass (including the orientation of the discontinuity) and the stresses on the rock mass are in both cases the same. Only the volume of the rock mass is changed. It is easily seen that the rock mass in figure a) has a higher 'strength' than in figure b). In a), intact rock has to be broken, and in b) sliding along the discontinuity is sufficient for 'failure'.

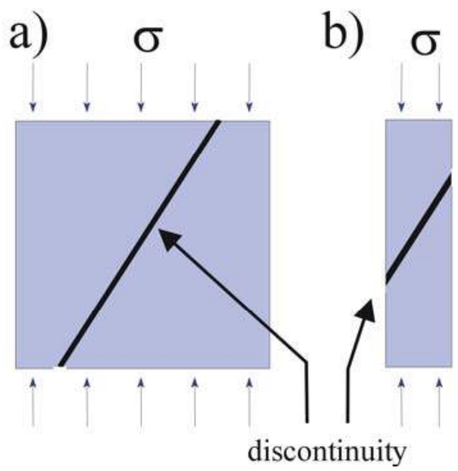


Fig. A-11.Rock mass under stress

Tensile strength of a rock mass

The bonding strength between the particles causes the tensile strength of intact rock. A rock mass with discontinuities has only a tensile strength if the discontinuities have a tensile strength or are filled, coated or cemented with a material that has a gluing or bonding effect between both sides of the discontinuity. This is not true for most rock masses at (near-) surface and for all practical purposes, most rock masses have a tensile strength equal to zero.

Compressive and shear strength of a rock mass

A rock mass consists of rock blocks bounded by discontinuities which have shear strength and may have some tensile strength. The rock mass could thus be considered as a large-scale rock material, rock blocks replacing mineral grains. In a rock mass with discontinuities that have a tensile strength, the bonding agent causing the tensile strength may be broken due to compressive or shear loading. This is comparable to the

[3] The compressive strength is dependent on the test method, see glossary.

[4] See glossary. Note: the Mohr-Coulomb failure criterion does not suit all rocks in all situations and different theoretical or empirical models for which the strength of intact rock have been defined. These will not be repeated here as these can be found in any standard textbook on rock mechanics (e.g. Goodman, 1989, Hoek et al., 1992).

failure of intact rock material and compressive and shear 'strength' may be defined, although these 'strengths' are likely anisotropic and may still depend on the environment. If the discontinuities do not have tensile strength the rock mass may be compared to not-cemented dense sand, where grains, being the intact rock blocks, fit closely together. The environment (confinement, etc.), the shear strength along the discontinuities, and the intact rock strength determine the maximum compressive and shear load that can be sustained[5]. Thus, 'failure' depends on the configuration of the rock mass and the orientation and variation of the stress fields. Generally, valid compressive and shear strength values can therefore not be defined[6]. In some situations where anisotropy is absent or not very important, it is, however, possible to approximate the strength behavior of a rock mass in models analogous to the methods used for intact rock, but with strongly reduced values for compressive and shear strength.

Weathering

Weathering is the chemical and physical change in time of intact rock and rock mass material under the influence of the atmosphere and hydrosphere. Two main processes are distinguished: physical and chemical weathering. Physical weathering results in the breakdown of rock material into progressively smaller fragments. The rock and rock mass break up due to temperature variations resulting in differential expansion and shrinkage of minerals, freezing and thawing of water, pressures of water in pores and discontinuities, (re-) crystallization pressures, hydration, and frequent swelling and shrinkage of clays due to water absorption, etc. Chemical weathering results in decomposition of minerals. Water and groundwater with dissolved chemical agents are of major importance as these react with rock and rock mass material. Normally biotic influences, induced by living organisms, plants, bacteria, worms, etc., are included and cause physical as well as chemical weathering. On or near to the surface the influence of these processes (due to larger temperature variations, influence of vegetation and rain, etc.) is more distinct than deeper below the surface. The effects of stress relief, intact rock creep, and rock mass creep are also included in the definition of weathering as proposed by Price (1995). Intact rock, and rock mass creep and stress relief can lead to new cracks in intact rock, develop integral discontinuities into mechanical discontinuities, and open existing discontinuities.

A distinction is made between 1) the degree (state) of weathering and 2) the susceptibility to weathering. The degree of weathering denotes the state of weathering of a particular rock mass or geotechnical unit at a certain moment. Susceptibility to weathering is the susceptibility of the rock mass to further weathering in the future.

The influence of weathering on intact rock and on discontinuities is as follows:

Weathering of intact rock - Weathering of intact rock discolors the material and decreases intact rock strength. Further progressive weathering of minerals and cement may lead to a decomposition of the intact rock ultimately resulting in a residual soil. New cracks may develop in blocks of intact rock.

Weathering of discontinuities –In general, the discontinuity wall material and the infill material are weakened, resulting in lower shear strength along the discontinuities. The material resulting from weathering of the discontinuity walls will often form an infill in the discontinuities. The discontinuity wall loses its asperities and becomes smoother. Integral discontinuities can develop into mechanical discontinuities. The discontinuities become visible and can therefore be measured (Price, 1993) resulting in lower values for discontinuity spacings.

Creep

Creep in rock mechanics is a confusing term. Various forms of plastic or time dependent deformation processes that are governed by very different physical or chemical processes are all described as creep. In these

[5] Comparing a rock mass to intact rock or to not cemented sand is only partly valid. The elements in a rock mass (rock blocks) fit together like dry masonry, whereas the grains in intact sedimentary rock or in sand do usually not fit together. The cement in a rock mass is in the discontinuities whereas in intact rock or in sediment the elements (grains) are bound together by cement filling the pores between the grains.

[6] An alternative way to understand rock mass 'strength' is as follows: If loaded to failure under a compressive or shear stress a piece of intact rock will break into smaller pieces of rock when the compressive or shear strength is reached ('the rock fails'). Effectively it then becomes a rock mass (intact pieces of rock with boundaries by fractures = discontinuities). Reversed this leads to the conclusion that a rock mass does not have a compressive or shear strength; it already consists of blocks with boundaries by discontinuities.

notes, the term creep is avoided as much as possible, but if used, the process responsible for the creep will be named. The following are examples of ‘creep’.

Creep in intact rock - Creep in intact rock usually means that the intact rock deforms with time under a constant load. The velocity of the deformation depends on the level of the load. Creep deformation takes place by solution and re-crystallization of minerals, or by the growth of micro cracks into larger cracks, sometimes leading to failure. Both require time and are dependent on stress levels.

Creep in a rock mass - In a rock mass all processes of creep in intact rock may occur, together with time and stress level dependent deformation along and perpendicular to discontinuities.

Slope creep - Slopes are said to creep if the surface layer of the slope moves downhill in a slow process under influence of gravity. Underlying mechanisms are deformation of intact rock and displacements along existing discontinuities. Processes such as weathering of the intact rock (growth of new mechanical discontinuities) and discontinuity infill material, and creep in intact rock and rock mass are normally also included. The process is facilitated by fluctuations in pore and discontinuity water pressures from water flowing over and through the surface of the slope.

Porosity

Porosity is defined as the pore space (space not occupied by rock material and filled by vapors or fluids) in intact rock or in a rock mass. Porosity is divided in primary and secondary porosity. Primary porosity is the porosity of intact rock and secondary porosity is the porosity of the rock mass due to discontinuities.

Permeability

Permeability is a property of the rock material or mass and describes the ease with which a fluid may move through it. Primary permeability is the permeability of intact rock whereas the secondary permeability is the permeability of the discontinuity system in a rock mass. The permeability of intact rock is usually lower than that of the rock mass.

B INTACT ROCK STRENGTH AND DEFORMATION

Intact rock strength and deformation is often of importance, but the importance is often less than expected. In strong rock, it is clear that failure trough intact rock will seldom happen, but also in weaker rocks, intact rock strength may be less important. Sometimes a rock mass with low intact rock strength (based on unconfined compressive strength - UCS tests) appears to have failed through intact rock failure, but, on closer examination, the low intact (UCS) strength is a consequence of a large number of (mechanical) discontinuities in the rock test specimen. Thus a shale may have a very low intact rock strength as determined by conventional UCS testing), but this is not caused by the low strength of the intact material but by the numerous closely spaced bedding planes. A similar reasoning can be followed for deformation of intact rock. In many cases it is, however, necessary to obtain the characteristic or mean values of strength and deformation for intact rock of a geotechnical unit in which the engineering structure is made or to be made.

B.1 Unconfined and confined compressive strength of intact rock

The compressive strength is the compressive stress at failure of a sample under a compressive stress (σ1). Compressive strength of rock or rock mass material can be tested under different stress configurations. Depending on the type of test done the compressive strength is denoted as Unconfined Compressive Strength (UCS), triaxial compressive strength or true triaxial compressive strength. Unconfined compressive and triaxial tests are normally done on cylindrical samples and a true triaxial test is done on a rectangular (mostly cubic) sample.

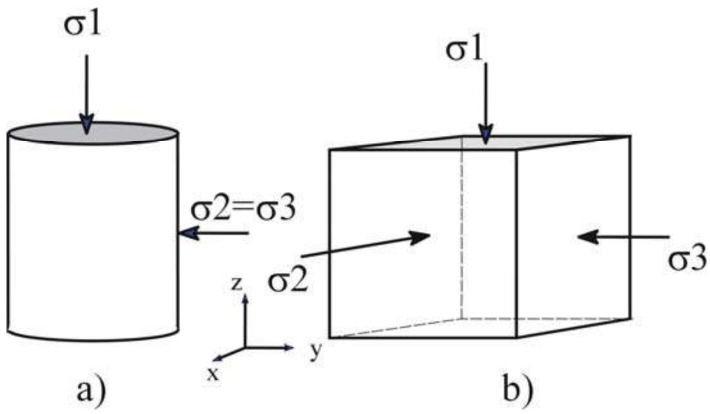


Fig. B-1. Compressive strength

Unconfined Compressive Strength (UCS)

The compressive stress (σ1) is measured at failure of the sample under the condition that the confining pressure is zero (σ2 = σ3 = 0) (Fig. B-1a). Typical values of UCS values are listed in **Error! Reference source not found.**.

Triaxial compressive strength

The compressive stress (σ1) is measured at failure of a sample that is under a confining pressure. The confining pressure is equal in x and y direction (σ2 = σ3) (Fig. B-1a). Typical values are listed in **Error! Reference source not found.**.

True triaxial compressive strength

The compressive stress (σ1) is measured at failure of a sample that is under confining pressure. The confining pressure is not equal in x and y direction (σ2 ≠ σ3) (Fig. B-1b).

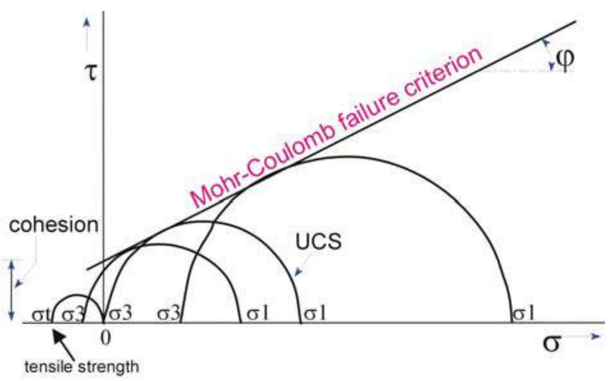


Fig. B-2. Mohr-Coulomb failure criterion

Interpretation of the test results can be done following the Mohr-Coulomb failure criterion (Fig. B-2). Many intact rocks, however, do not fit the Mohr-Coulomb failure criterion and other empirical relations between failure and stress configuration have been proposed. An example is shown in Fig. B-3. The relation shown in Fig. B-3 follows the empirical relation in eq. [B-1] (Bieniawski, 1974).

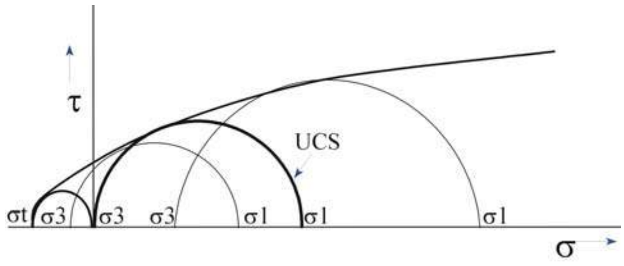


Fig. B-3. Empirical relation

$$\frac{\sigma_1}{UCS} = 1 + N\left(\frac{\sigma_3}{UCS}\right)^M \qquad \text{[B-1]}$$

$\sigma_1,\ \sigma_3$ = major,respectivily, minor principal stress

$N,\ M$ = material dependent coefficients

The compressive strength is an expression depending on the way of testing. In practical applications, the test and the compressive and confining pressures should be as much as possible similar to those that will occur in the application. Hence, the strength should be determined in a situation that resembles the real application. Obtaining samples for testing is a relatively costly affair and therefore various alternative methods to determine or estimate compressive strength have been developed.

B.2 Tensile strength of intact rock

Intact rock has a tensile strength. The tensile strength can be established by a direct unconfined tensile strength test (Fig. B-4a) or by an indirect tensile strength test Fig. B-4b. The tensile strength plots negative in the 'circles of Mohr-Coulomb' analyses (Fig. B-2 and Fig. B-3). The indirect tensile strength, also called 'Brazilian tensile strength', is established by compressing a disk of the material. The disk will fail by induced internal tensile stress.

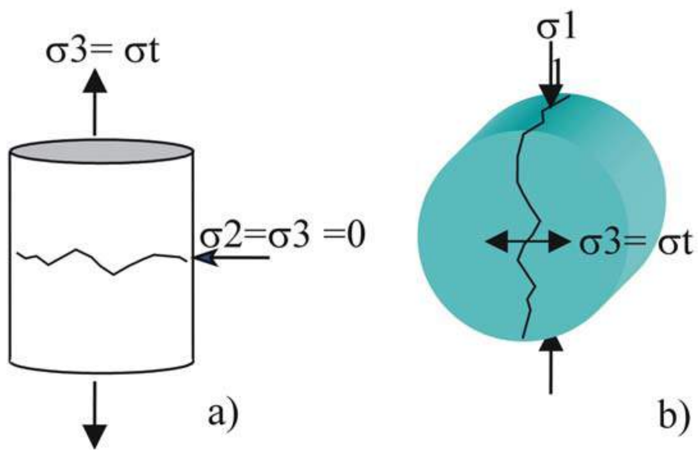


Fig. B-4. Tensile strength

Tensile strength of intact rock is generally highly unreliable. Small inhomogeneities or small cracks may decrease the tensile strength considerably. Additionally, failure in a tensile stress environment is a mechanism propelling itself. If failure starts, the tensile stress has to be taken by the remaining not failed part. This volume is overstressed even more and fails faster. Tensile failure is a (very) rapid process and happens normally without warning. For these reasons, the tensile strength of intact rock is generally not considered in design in rock mechanics and in calculations assumed to be nil. Typical values for intact tensile strength are listed in Table B-1.

Table B-1. Intact rock and rock mass strength

Rock name	Intact rock		Rock mass		
	UCS (MPa)	tensile strength (MPa)	friction (φ) (degrees)	cohesion (Si) (MPa)	range of confining pressure (MPa)
Bera sandstone (1)	74	1.17	28	27	0-200
Nvajo sandstone (1)	214	2.11			
Sioux quartzite (1)			48	71	0-203
Pottsville sandstone			45	15	0-69
Hackensack sandstone (1)	123	2.9			
Oneota dolomite (1)	87	4.4			
Flaming Gorge shale (1)	35	0.2			
Muddy shale			14	38	0-200
Edmonton bentonite shale (30% water) (1)			7.5	0.3	0.1-3.1
Nevada Test Site tuff (1)	11	1.1			
Falset granodiorite (unweathered) (2)	175		47	0.017	0-0.6
Falset granodiorite (slightly weathered) (2)	110		46	0.016	0-0.6
Falset granodiorite (moderately weathered) (2)	80		38	0.014	0-0.6
Falset granodiorite (highly weathered) (2)	3		17	0.008	0-0.6
Falset granodiorite (completely weathered) (2)	0.5		6	0.003	0-0.6
Falset Lower Muschelkalk Limestone (thick bedded) (2)	80		62	0.027	0-0.6
Falset Upper Muschelkalk Limestone (slightly weathered (thick laminated) (2)	70		18	0.007	0-0.6
Falset Buntsandstone (2)	25				
Data after: 1) Goodman, 1989 2) UCS and tensile strength (ITC, 1998); rock mass friction and cohesion of SSPC system (back analysis of slope stability, Hack, 1998).					

B.3 Shear strength of intact rock

Shear strength of intact rock is only seldom established by direct shear testing (direct shear box see C.8), but normally established by triaxial or true-triaxial testing. The reason is that the necessary stresses are such that testing equipment is expensive and the results are normally comparable to those obtained with triaxial compressive tests.

B.4 Impact methods to determine intact rock compressive strength

The Schmidt hammer determines the rebound of a piston activated by a spring (the Schmidt Hammer was originally developed for testing concrete quality). The rebound values measured on rock surfaces have been correlated to intact rock strength. Schmidt hammer values, however, are influenced by the material to a fairly large depth behind the surface. If a discontinuity lies within the influence sphere, the Schmidt hammer values will be affected. The Schmidt hammer is thus not considered suitable to measure rock material strength in the field. The same applies to any other impact/rebound devices whose released energy per surface unit area is of the same order of magnitude as the Schmidt hammer of L or N design). Equotip or other rebound impact devices might be suitable, but as these devices are only recently applied to rock mechanics it is not yet certain whether the relationships between rebound values and intact rock strength are correct.

B.4.1 'Simple means' intact rock strength field estimates

'Simple means' field tests that make use of hand pressure, geological hammer, etc. (Burnett, 1975), are used to determine the intact rock strength classes following the British Standard (BS 5930, 1981) (the test classes are listed in Table B-2). The 'simple means' field tests to

Table B-2. Estimation of intact rock strength

intact rock strength	'simple means' test (standard geological hammer of about 1 kg)
< 1.25 MPa	Crumbles in hand
1.25 – 5 MPa	Thin slabs break easily in hand
5 - 12.5 MPa	Thin slabs break by heavy hand pressure
12.5 - 50 MPa	Lumps broken by light hammer blows
50 – 100 MPa	Lumps broken by heavy hammer blows
100 - 200 MPa	Lumps only chip by heavy hammer blows
> 200 MPa	Rocks ring on hammer blows. Sparks fly.

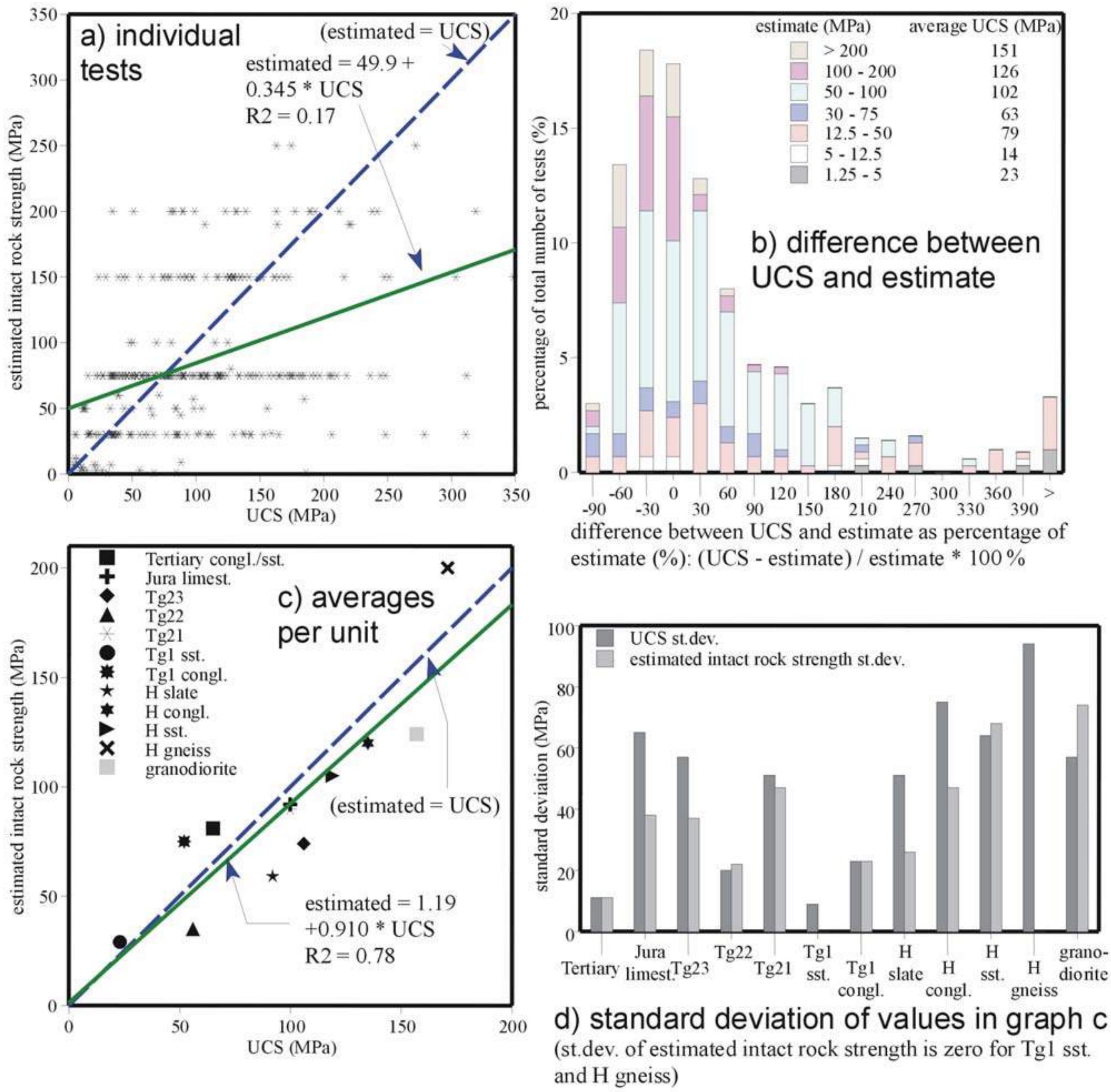


Fig. B-5. Estimated intact rock strength vs. strength values determined by UCS tests. (The dashed lines in A and C indicate the relation if estimated strength equals UCS strength.) (Number of UCS tests: 941) (Hack, 1998)

estimate intact rock strength following Table B-2 have been extensively tested against large amounts of unconfined compressive strength (UCS) tests in the same geotechnical units and in the same exposures. If possible, estimates and UCS tests were done both perpendicular and parallel to the bedding or cleavage.

The extensive quantity of tests allowed a thorough analysis of the accuracy and reliability of the 'simple means' field tests for estimating the intact rock strength. This analysis is presented in the following chapters. The estimated strength values in the graphs in this chapter are plotted as the mid values of the ranges of Table B-2. If the strength was estimated to be on the boundary between two classes, the boundary value is used.

B.4.2 Intact rock strength field estimates versus UCS tests

In Fig. B-5a, the estimated values of intact rock strength by 'simple means' field tests are plotted versus UCS test values for all locations for which both were available. In Fig. B-5b, the differences between the UCS test values and the estimated values as percentage of the estimated values are plotted, and in Fig. B-5c the averages of estimated and UCS values per unit. In Fig. B-5 no differentiation is made for the direction of the measurements. Fig. B-5a shows that the scatter is wide and consequently only low or no correlation can be seen. In Fig. B-5b, is clearly visible that the differences between UCS and estimated values do not show a normal distribution for lower strength values. The distribution is skewed to higher values, e.g. the UCS values are higher than the estimated values. For high strength values, the distribution of the differences is more normal but the average values of the UCS tests per estimated strength class are lower than the averages of the estimated values. A quite good correlation is found for the averages per unit (Fig. B-5c). The standard deviation of the UCS values per unit is for most units considerably higher than the standard deviation for the estimated strength value per unit (Fig. B-5d).

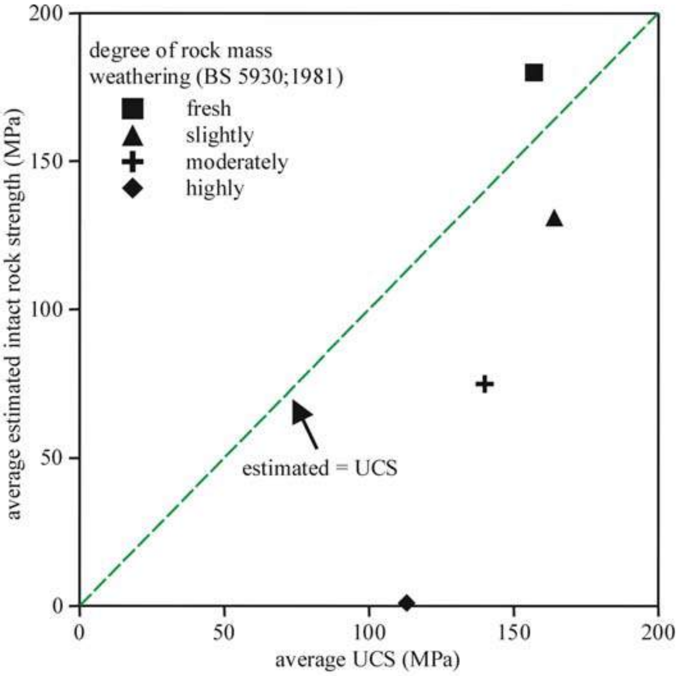
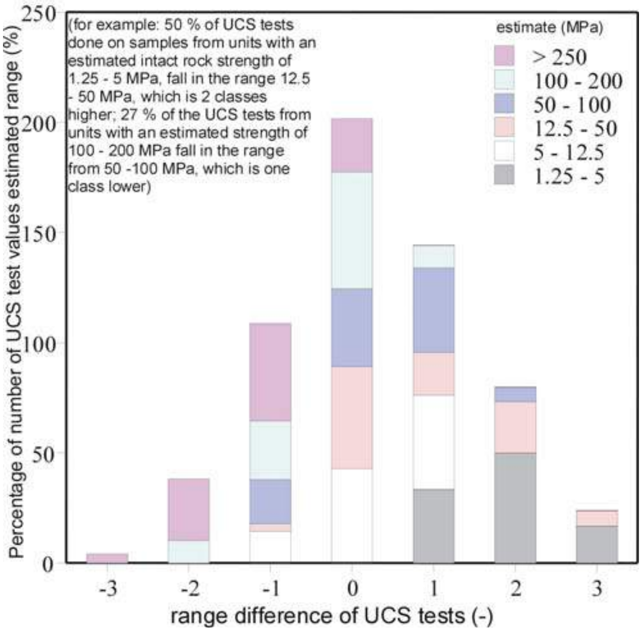


Fig. B-6. Average estimated intact rock strength vs. average UCS for granodiorite units with various degrees of rock mass weathering. (after Hack, 1998)

If is assumed that a unit has a characteristic strength distribution with a characteristic mean strength value, then the estimated value will be nearer the mean value of the distribution because it is an average of more tests. The UCS test value is, however, only a single value or the average of few test values (normally less than three or four) and is likely to differ more from the mean value. This leads to the conclusion, as expected, that the characteristic mean strength value of a unit is better determined by a large quantity of estimated values than by few UCS tests. The skew of the distribution of the differences between UCS and estimated values for low strength (Fig. B-5b) is probably caused by the fact that samples are not taken randomly. Samples are very seldom taken from the worst parts of a rock exposure.

Bias in sample taking for testing

A similar bias effect is observed by an analysis of the results of intact rock strength estimation and UCS tests for granodiorite with various degrees of rock mass weathering in the same exposure. In Fig. B-6, UCS values are considerably higher than the estimates of intact rock strength for the higher degrees of weathering of the rock mass. The granodiorite has weathered starting from the discontinuities and often a complete sequence of weathering is found. The weathered material and certainly the highly weathered parts will break from the sample during transport and sawing of the sample. Thus, the UCS test is done on pieces of rock material less weathered than the average degree of weathering in the unit and, therefore, results in a too high strength value.



The difference between UCS test values and estimated values for high intact rock strength might be due to a similar, but reversed effect. For high intact rock strength (> 100 MPa), it is often difficult to get sample blocks out of an exposure without equipment (saw, blasting, etc.) and a tendency exists to do tests on loose blocks that are more easily obtained. These, however, may have a lower strength. This effect is also observed in the granodiorite for which the estimated strength of the fresh exposures is higher

Fig. B-7. Percentage of UCS test values falling in a range different from the estimated value (after Hack, 1998) than the UCS strength values. The same effects, but for all rock units, are obvious in Fig. B-7, which shows the percentages of UCS tests falling in the ranges for the estimate of intact rock strength different from the estimated range value. For lower intact rock strength values, the UCS values are higher than the estimated values while for the higher intact rock strength values the UCS value is lower than the estimated value.

B.4.3 Repeatability of intact rock strength estimates

The repeatability of estimating the intact rock strength is fairly good. In the field, intact rock strength has been estimated by different students and staff members in the same exposure and in the same geotechnical unit. The results show that the majority estimates the strength to be in the same class and a minority estimates the strength to be in a class one lower or higher. Strength estimates more than one class different from the class estimated by the majority were rare and could often be attributed to real variability in intact rock strength within a unit. An argument against estimating intact rock strength by classifying following Table B-2, is that it would be dependent on the person who does the estimation, e.g. a large or physically strong person estimates the strength lower than a small or fragile person. This has only rarely been observed. The class ranges are obviously large enough to accommodate for most physical strength differences.

B.5 Influence of degree of water saturation on intact rock strength

Some porous rocks exhibit a difference in intact rock strength depending on the degree of water saturation when tested by UCS tests (Goodman, 1989, Bekendam et al., 1993). This depends on the porosity and the rock has to have a certain permeability to allow the water to fill the pores. If the porosity is low, the effect is also small. Whether impact tests establish the same effect of water as UCS tests for intact rock with a high porosity is not known.

B.6 Strength anisotropy

The correlation of the estimated value of intact rock strength with the UCS tested in a particular direction is not proven. Only in strongly anisotropic rocks (e.g. slate), the estimate is in agreement with the results from UCS tests. The highest strength is expected perpendicular to the cleavage direction. For the other rocks, the estimation of intact rock strength results in higher values parallel to the bedding direction. Fig. B-8 shows, per unit, the ratios of the strength perpendicular over the strength parallel for average UCS test values and for average field estimated values.

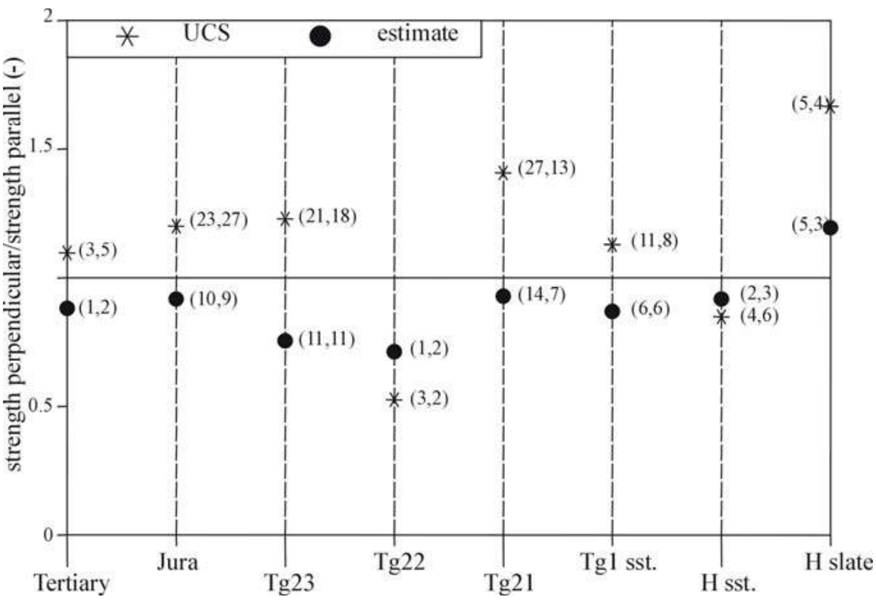


Fig. B-8. Ratio of average intact rock strength perpendicular over average intact rock strength parallel for UCS and field intact rock strength estimate per unit (values in brackets are the number of UCS tests respectively estimate) (after Hack, 1998)

Although this effect is not studied in detail, a possible (and tentative) explanation could be as follows. All rocks included in Fig. B-8 have intact rock strengths that are in 'intact rock strength estimate' classes established by hammer blows (≥ 12.5 MPa). The field estimate by hammer blows is a form of impact (dynamic) testing by which the rock breaks due to the impact energy (e.g. hammer blow). The impact energy is a limited quantity of energy induced into the rock in a small amount of time. Energy induced per time unit is thus high. The UCS test is a static test by which an unlimited amount of energy is induced into the rock until failure in a relatively large time span. The energy induced per time unit is low.

Deformation of rock is a time dependent phenomenon. It requires a certain amount of time before a stress is converted into a deformation and vice versa. Stress and deformation are linked and it requires time to transfer stress and deformation throughout a test specimen. In an impact test, part of the energy dissipates due to crack forming directly at the impact point. The remaining energy travels through the rock as a stress/deformation wave (e.g. shock or seismic wave). This wave is reflected at layer boundaries and at the end of the sample. When the incident and reflected waves are at the same location and have the same phase, the stresses (and deformations) enhance and may cause the rock layer to break. In a layered sample, the distance between layers is smaller than the length of the sample. The wave will loose energy (due to spherical dispersion, non-elastic deformation, absorption, etc.) during traveling through the rock. A wave reflected against the end of the sample with a longer travel distance, has thus less energy than a wave reflected against a layer boundary. The concentration of energy at a certain point due to the coincidence of direct and reflected waves will also be less.

This may be the explanation that a rock sample when tested (by hammer blows) breaks more easily perpendicular than parallel to the layering and thus that the strength estimate for a sample tested perpendicular is lower than tested parallel. It is likely that this mechanism is less (or does not occur) in very thin spaced layered material (e.g. slate) because the rock at the impact point is easily fractured and broken whatever the orientation.

The induction of energy in the sample in a UCS test is so slow that a stress/deformation wave will not occur. The whole sample will be stressed and deformed. The tensile strength perpendicular to the layer boundary planes in a layered material is normally less than the tensile strength of the material. In a UCS test of layered material tested parallel to the layering, failure will occur due to bending and separation of the individual layers, resulting in breaking of layers (starting with the layers at the rim of the sample). Perpendicular to the layering failure occurs due to stress concentrations in the intact rock of individual layers. Bending of the layers and consequent cracking/failure requires mostly less stress/deformation than breaking the rock due to stress concentrations and thus is the measured strength perpendicular larger than parallel to the layering.

B.7 Stress-strain

B.7.1 Ideal elastic, homogene and isotropic intact rock

If a stress is applied on a body of intact rock, the rock will deform. Stress and strain are coupled properties; there is no strain without stress and vice versa. The deformation per length unit is called the strain. The relation between stress and strain for an ideal elastic, homogene, and isotropic intact rock is (Fig. B-9):

$$\varepsilon_l = \frac{\Delta l}{l} \qquad E = \frac{\sigma}{\varepsilon_l} \qquad \text{[B-2]}$$

l = length of body

Δl = deformation in length direction

ε_l = strain σ = stress

E = modulus of elastic deformation

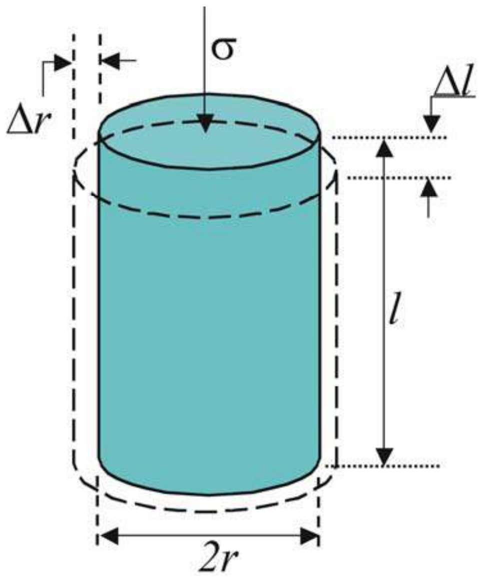


Fig. B-9. Stress-strain

Under influence of the stress in the length direction, the width of the body will become larger. The ratio between the strain in width direction to the strain in the length direction is the Poisson's ratio:

$$\nu = \frac{\varepsilon_r}{\varepsilon_l} \qquad \text{[B-3]}$$

ε_l = strain in l direction ε_r = strain in r direction

ν = Poisson's ratio

B.7.2 Non-elastic intact rock

Most intact rock is not linear elastic and the deformation modulus is mostly not elastic or only partially elastic (Fig. B-10). Rocks may deform plastically or a combination of plastic, elastic, and brittleness. Brittleness means that the intact rock fails. It is therefore better to use “D” for the ration between stress and strain instead of “E”.

B.7.3 Anisotropy

Intact rock is often not isotropic, for example, alignment of flaky minerals causes anisotropy. An orthotropic symmetric (= three perpendicular symmetric directions) anisotropic, homogene, and ideal elastic rock is described as follows:

$$\begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{bmatrix} = \begin{pmatrix} \frac{1}{E_x} & -\frac{\nu_{yx}}{E_y} & -\frac{\nu_{zx}}{E_z} & 0 & 0 & 0 \\ -\frac{\nu_{yx}}{E_x} & \frac{1}{E_y} & -\frac{\nu_{zy}}{E_z} & 0 & 0 & 0 \\ -\frac{\nu_{zx}}{E_x} & -\frac{\nu_{zy}}{E_y} & \frac{1}{E_z} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{xy}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{yz}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{zx}} \end{pmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{bmatrix} \qquad [B-4]$$

Complicated forms of anisotropy may be very difficult or impossible to describe fully mathematical and can only be handled with numerical calculation methods. (Note that a rock mass is virtually always anisotropic due to the presence of discontinuities.)

B.7.4 Rock mass

A rock mass is not homogene, is not isotropic, and does not behave as an ideal elastic material. This applies to most intact rock but is certainly applicable to rock masses because of the presence of discontinuities. This makes that analytical formulations of stress-strain relations, strength criteria, and calculations of rock masses are mostly impossible. Examples of deformation of a rock mass are given in ch. A.2.4.

B.8 Time effects and creep

Deformation of intact rock (and rock masses) is time-dependent. There are various reasons that cause this time-dependency:

1. No strain can be instant. Instant strain would require an infinite velocity of the material particles, which is impossible. The particles have a certain mass and displacement requires a certain time span.

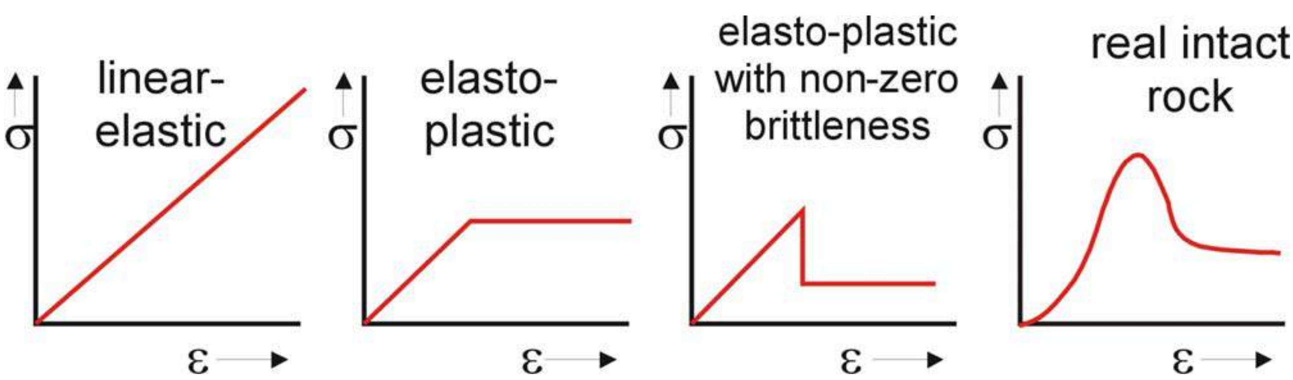
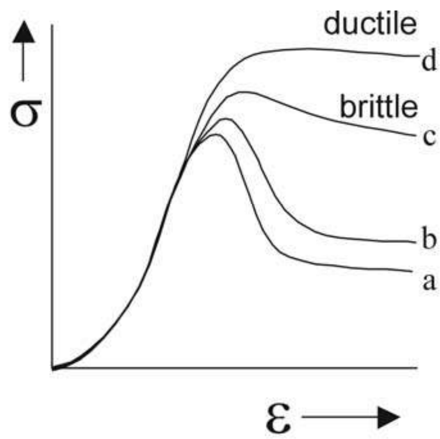


Fig. B-10. Stress versus strain for various materials

2. If stress is applied, shock waves of stress-strain will travel trough the rock and rock mass. It will take some time before stress and strain will be in equilibrium throughout the rock and rock mass. Under slow application of stress, the shock wave effect may be minimal but it will still take some time before equilibrium between stress and strain throughout a rock and rock mass is established.
3. All rocks show some (for some rocks limited) effects of time dependent deformation; strain increases with time under constant stress. This is called creep. Creep is responsible for the delay in, for example, collapses of underground excavations. The rock mass is loaded with a new stress environment due to the excavation of the tunnel. If stress levels do not exceed immediately the strength of the rock mass, the excavation will not fail immediately. However, small micro cracks will develop in the rock if the stresses at the wall are more than half the UCS strength (rule of the thumb). The number of cracks will increase over time and after some time the rock will fail.
4. All rocks show a long-term creep effect. The rock deforms and after some time the rock will fail even if the rock is stressed well below half of the UCS strength. The mechanisms for this effect are largely unknown, but it is thought that re-crystallization under stress may play a role. Long-term creep is likely responsible for some collapses of excavations after very long time spans, sometimes up to 2000 years.

B.9 Brittle – ductile behavior of intact rock

The process of failing is within a short time span for most intact rock under surface temperature and confining pressure conditions. These rocks show a 'brittle' behavior'. Some rocks noticeable, salts; show a failure gradually in time; this is denoted as 'ductile' behavior. All rocks show a more ductile behavior with increasing temperature and confining pressure (Fig. B-11). Depending on temperature and confining pressure, a so-called brittle-ductile transition can be indicated for all rocks.



stress-strain tests on the same material with incresing confining pressure and/or temperature (a<b<c<d)

Fig. B-11. Brittle – ductile behavior

C SHEAR STRENGTH ALONG A DISCONTINUITY

The orientation of discontinuities in combination with the shear strength along discontinuities determines the possibility of movement along discontinuities. The influence of discontinuities on various engineering and mining structures and on slope stability is extensively described in the literature (Barton et al., 1990a, Goodman, 1989, Hoek et al., 1980, 1981, etc.). A basic problem is that shear strength along discontinuities is not fully understood. Some deterministic and empirical models do exist to calculate shear strength from discontinuity characteristics (form of discontinuity, type of infill material, etc.), however, most of these methods are not without criticism and do not always work in all circumstances. The literature describing shear strength of discontinuities is extensive and often contradictory.

C.1 Persistence

Persistence[7] determines the possibilities of relative movement along a discontinuity. Discontinuities are usually differentiated in (Fig. C-1) (ISRM, 1978b, 1981a):

1) **Persistent discontinuities**; the discontinuity is a continuous plane in the geotechnical unit. Shear displacement takes place if the shear stress along the discontinuity plane exceeds the shear strength of the discontinuity plane. If unfavorable orientated it is often a sliding plane in slopes.

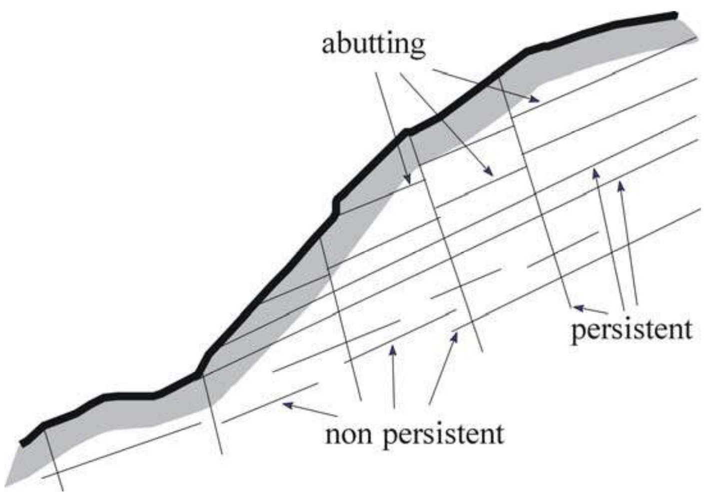


Fig. C-1. Persistent, non-persistent, and abutting discontinuities

2) **Abutting discontinuities** abut against other discontinuities. Abutting discontinuities might continue at the other side of the intersecting discontinuity, however, with a displacement to give so-called ‘stepped planes’. Shear displacement along the discontinuity can take place if a) the shear strength along the discontinuity plane is exceeded and b) the blocks of rock against which the discontinuity abuts can move.

3) **Non-persistent discontinuities** end in intact rock. Before movement of the blocks on both sides of a non-persistent discontinuity is possible, the discontinuity has to extend and break through intact rock material. As intact rock material has virtually always far higher shear strength than the discontinuity, a non-persistent discontinuity will have larger shear strength than a persistent discontinuity.

The above definitions do not consider differences in persistence in different directions. It is assumed that the discontinuity is persistent in any direction for the same length. This is not necessarily true. A discontinuity might be persistent in dip direction but not persistent perpendicular to the dip direction or vice versa (ISRM, 1978b, 1981a). In the literature, different authors treat persistence in different ways. Some (Barton et al., 1974, 1976a, 1988) treat persistence combined with roughness of the discontinuity walls while Selby (1980, 1982) combines persistence with the classification of infill material. In his classification, Laubscher (1990) includes only those discontinuities that are larger than visible, thus those extending for a length larger than

[7] Persistence is treated as a discontinuity property because in most forms of rock mechanical calculations persistence is taken as a discontinuity property (for example, in the classification systems of Barton, 1974, 1976a, 1988, and Selby, 1980, 1982).

the exposure or tunnel, or those abutting against another discontinuity. Further quantitative descriptions of persistence are few and probably not fully satisfactory (Bandis, 1990).

C.2 Discontinuity roughness

The contribution of discontinuity roughness to the shear strength of a discontinuity can directly be measured with, for example, a shearbox test, but only for relatively small surfaces. In theory, the contribution of roughness to the shear strength of a large surface can be determined from other easily determined discontinuity parameters, such as the friction of the material and the measurement of roughness profiles (Patton, 1966) following the 'bi-linear shear criterion'.

C.2.1 Bi-linear shear criterion

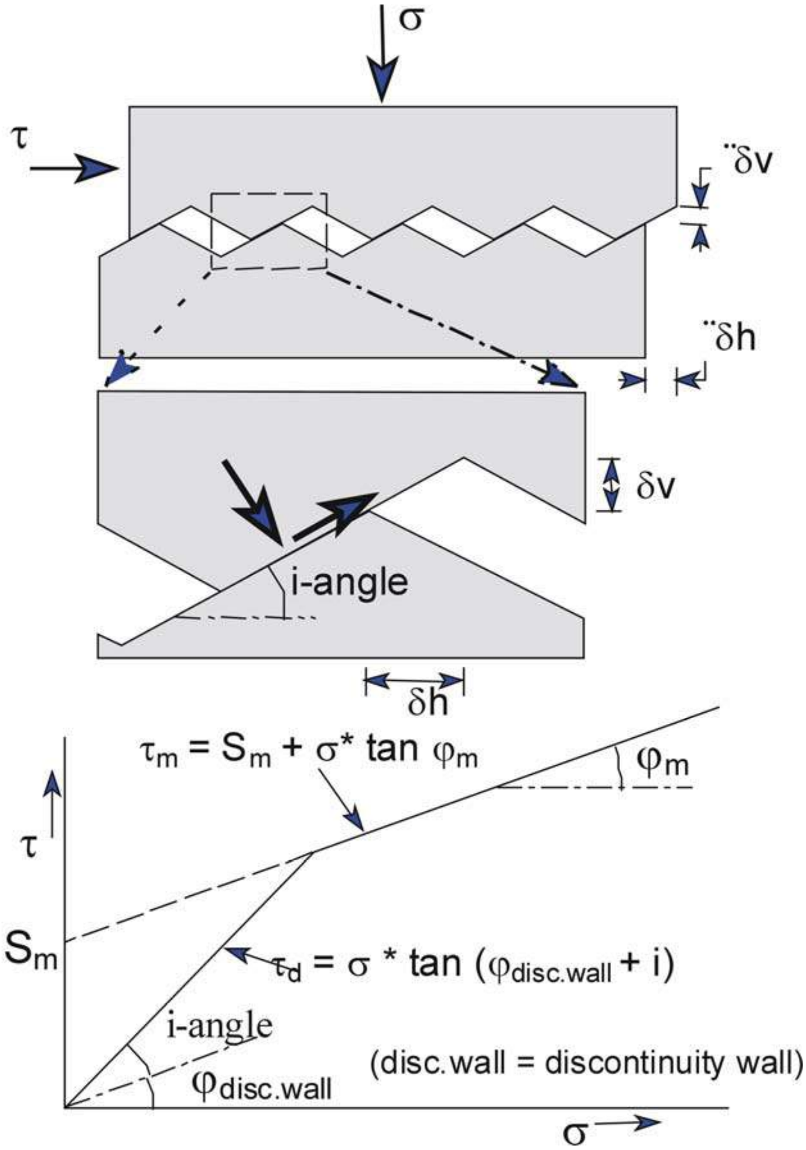


Fig. C-2. 'Bi-linear shear criterion' for a discontinuity with a regular set of triangular shaped asperities (modified after Patton, 1966)

Terminology of shear strength along a discontinuity is easiest explained with the 'bi-linear shear criterion' (Patton, 1966) (for more sophisticated relations for shear strength along discontinuities is referred to the appropriate literature). Patton formulates the shear strength along a discontinuity in the 'bi-linear shear criterion' for a discontinuity with a regular set of triangular shaped asperities (Fig. C-2). The angle of friction along the discontinuity wall ($\varphi_{disc.wall}$)[8] is a material constant depending on the structure, texture, type of material, roughness, and degree of interlocking of the discontinuity surfaces. The roughness included in $\varphi_{disc.wall}$ should not cause dilatancy of the discontinuity (opening in the direction perpendicular to the shear plane, δv in Fig. C-2. The roughness that causes dilatancy of the discontinuity is described by the angle of roughness (*i-angle* = arctan $\delta v/\delta h$). In Fig. C-2 the roughness are the triangular asperities. Depending on the steepness of the asperities and the normal stress across the discontinuity, the asperities will break rather than that these are overridden. The shear strength is then described by the rock material parameters cohesion S_m and friction φ_m. If there is no gluing or bonding agent (for example, cement) between the walls of the discontinuities, the cohesion is described as 'apparent cohesion'. The cohesion, or a part of it, may be 'real cohesion' if a gluing or bonding agent is present. Note that the parameters cohesion S_m and angle of friction φ_m are for a discontinuity surface with sheared asperities or with non-fitting asperities. These parameters are normally not the same as the cohesion and angle of (internal) friction (φ) of the intact rock material as defined by the 'Mohr-Coulomb failure criterion'.

[8] In the literature, the friction angle along the discontinuity plane is often described as *φ-basic*. This denotation is confusing. It seems to denote a material constant depending on the type of rock material only. The discontinuity wall, however, may have been altered (for example, by weathering) and the angle of friction along the wall may depend on the alteration.

C.2.2 Problems with the 'bi-linear shear criterion'

The 'bi-linear shear criterion' is, too simple for natural irregular discontinuity surfaces. More complicated theories about roughness profiles, methods to characterize roughness profiles, and relations between roughness profiles and shear strength are to be found in the literature (Bandis et al., 1981, Barton et al., 1977, Fecker et al., 1971, Grima, 1994, Hsein et al., 1993, ISRM, 1981a, Rengers, 1970, 1971, etc.). However, many of these relations between roughness and shear strength are hampered by scale effects (Cunha, 1990, 1993) or do not consider all discontinuity properties that are important. In fact, the determination of the contribution of roughness to the shear strength is so complicated that exact methods for large planes can probably not exist other than by full scale shear tests. Variation of roughness properties throughout a rock mass and the impossibility to establish the roughness properties for discontinuity surfaces that are not exposed, complicate the matter even further. Obtaining the properties in the required detail to make it worthwhile to apply a sophisticated methodology is therefore mostly impossible or impractical.

C.2.3 Roughness parameters

The importance of the roughness of a discontinuity partly depends upon the stress configuration on the discontinuity plane in relation with the strength and deformation characteristics of the discontinuity wall material and asperities. To understand clearly the mechanisms involved, the three following theoretical situations are distinguished. These situations apply to a discontinuity without infill

1. Overriding of asperities - the rock blocks on both sides of the discontinuity are not confined[9] and no shearing through asperities occurs.
2. Deformation of asperities - the rock blocks on both sides of the discontinuity are confined[9] and no shearing through asperities occurs.
3. Shearing through asperities - the rock blocks on both sides of the discontinuity can be confined[9] or not be confined, but shearing through asperities occurs.

1) Overriding of asperities

For a plane sliding situation the normal stress (= the weight of the block under gravity) on the shear plane is constant in time (influences that can change the stress, such as snow, water, etc. are not considered for this theoretical situation). If is assumed that no asperities can be sheared off, because, for example, the strength is too high, the asperities have to be overridden for movement along the discontinuity to be possible. Then the most important roughness parameters are the friction of the discontinuity wall material ($\varphi_{disc.wall}$) and the maximum roughness angle (i_{max}) from the datum reference plane (Fig. C-3 left). The deformation characteristics of the rock material adjacent to the discontinuity

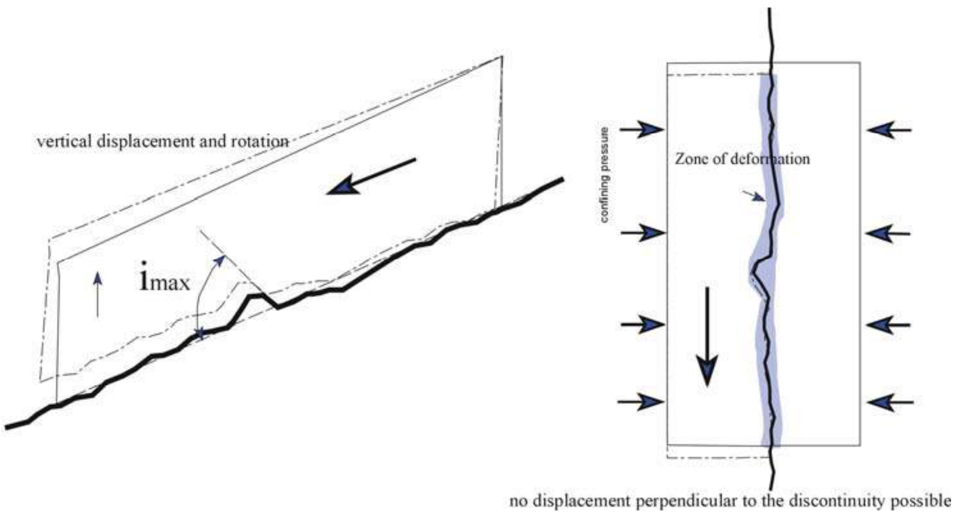


Fig. C-3. Influence of roughness on displacement without shearing through asperities (left figure: unconfined; right figure: confined)

[9] Confined denotes here that the rock blocks on both sides of the discontinuity are not free to move in the direction perpendicular to the discontinuity.

and the geometry of the asperities at other locations along the shear plane are of no or minor importance. Movement is impossible if $\varphi_{disc.wall} + i_{max} \geq 90°$.

2) Deformation of asperities

If a discontinuity is confined and no shearing through asperities can occur, then the angle of the roughness is less important but the geometry (in particular the maximum height) of the asperities, the amount of asperities and the deformation characteristics will mainly determine the shear strength (Fig. C-3 right, deformation is hatched).

3) Shearing through asperities

If shearing through asperities can take place then all parameters are of importance, e.g. the strength of the asperity material, the geometry, and the deformation characteristics (Fig. C-4). Not only all parameters are of importance but also all variations of these parameters everywhere along the plane where contact between the walls will occur during displacement.

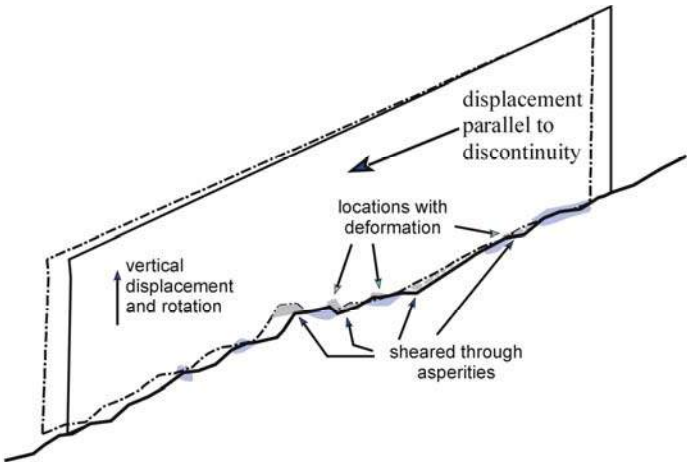


Fig. C-4. Displacement of block (shearing through asperities and deformation)

A complicating factor is that a piece of intact rock will often break under stress. Where and when a block of rock breaks is virtually impossible to establish by analytical calculations and highly complicated in a numerical analysis (Baardman, 1993). Situation 3) is the common situation and nearly all shear displacement along discontinuities is governed by a combination of overriding of asperities, deformation of asperities and shearing through asperities.

C.2.4 Measuring roughness

Measuring a roughness profile on an exposed plane is theoretically simple. All that is necessary is to measure the height of the surface above and below a certain datum plane at regular intervals. There are, however, practical problems with regard to the datum plane, the measuring interval, and the three-dimensionality of roughness.

Datum plane

Fig. C-5 (left) shows a single block on a slope with the datum plane for this particular block. The datum plane is established by a least squares regression analysis of the profile. The roughness profile can be determined by sampling at a regular interval, measuring the distance below and above the datum plane. Fig. C-5 (right) shows the same block but the block contains a (vertical) not-cemented mechanical discontinuity across which no tensile stresses can be transmitted. Thus, block B can move while block C remains stable, then datum planes have to be established for both blocks and do not have the same orientation. In a discontinuous rock mass, each independent block of rock material has therefore its own datum plane.

Measuring interval

The friction along the discontinuity plane is determined by the roughness of the discontinuity surfaces together with the friction properties of the material. 'Roughness' may range from the scale of atoms (e.g. irregularities in crystal structures) up to large-scale roughness of the order of meters. A uniform measuring interval is therefore not practical and roughness measurements have to be confined to certain ranges. The measured roughness (*i-angle's*) depends, however, on the measuring interval and consequently the shear strength calculated from this roughness[10].

[10] Fractal representation of roughness is proposed as a solution for this problem (Carr, 1989, Lee et al., 1990, etc.). Research showed, however, that the results published might be accidental. Fractal representation is therefore not suitable without further research and a proper definition of the used methodology (Den Outer et al., 1995).

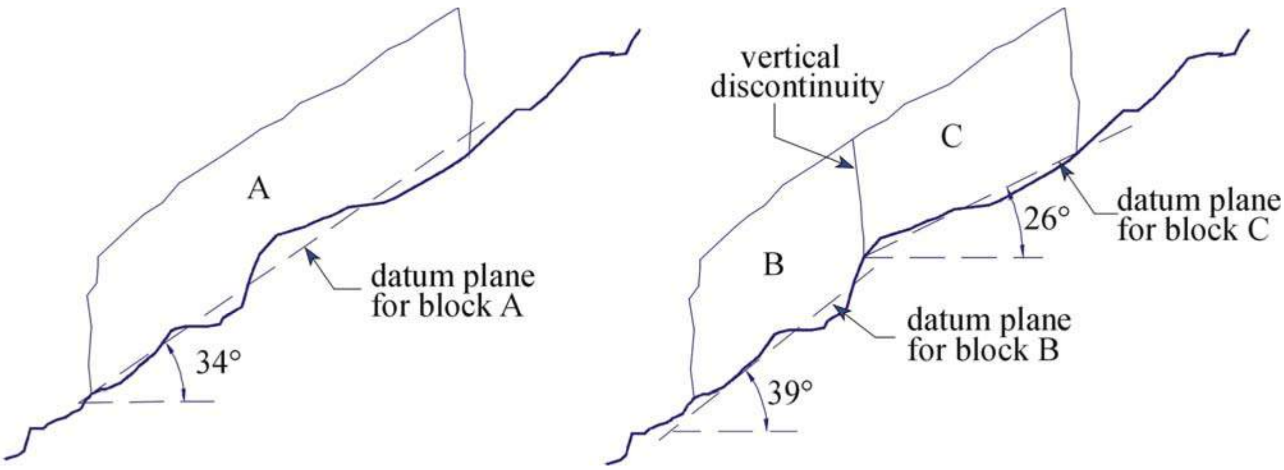


Fig. C-5. Roughness datum plane for single block (left) and same block intersected by vertical discontinuity (right)

Three or two dimensions

Most empirical shear strength relations or roughness profiles (e.g. Barton et al., 1977, ISRM, 1978b, 1981a, Laubscher, 1990, etc.) that include discontinuity roughness are based on two-dimensionality whereas the reality is three-dimensional. Discontinuity surfaces can be highly irregular in three dimensions. Fig. C-6 shows a series of parallel roughness profiles measured with a laser roughness meter on one discontinuity plane (Baardman, 1993). It is clear that the profiles are considerably different and that a shear strength calculation based on one profile will be different from those calculated on the other profiles. A complicating factor is that during displacement the contact points between two irregular surfaces can be anywhere. For these reasons, measurement of roughness should be done in three-dimensions.

C.2.5 *I-angle* measurements

Fecker and Rengers (Fecker et al., 1971, Rengers, 1971) developed a methodology to measure the roughness *i-angle* at different scales on a three-dimensional surface. Circular plates are positioned on an exposed discontinuity surface. The plates vary in diameter. Of each plate, the dip-direction and dip are measured and the results plotted in a stereogram (Fig. C-7). Note that the *i-angle* decreases until zero with increasing plate diameter.

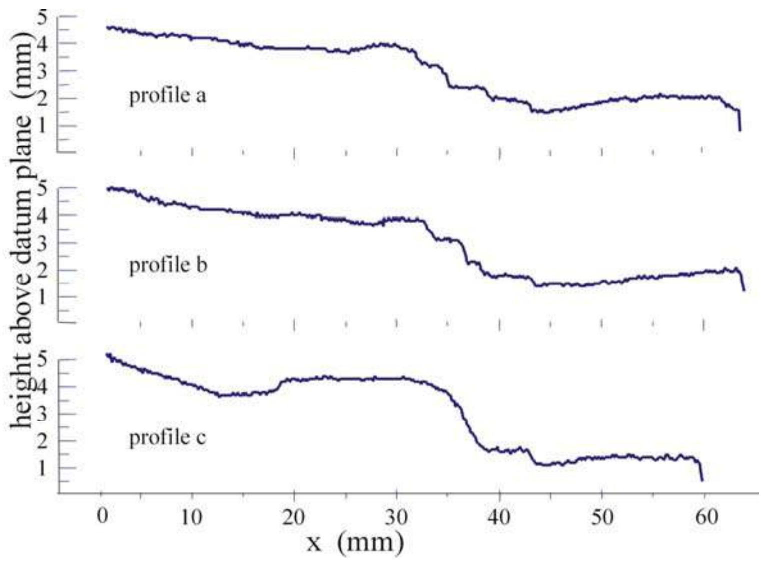


Fig. C-6. Parallel roughness profiles of one discontinuity plane. Spacing between profiles ≈ 1.5 cm (after Baardman, 1993)

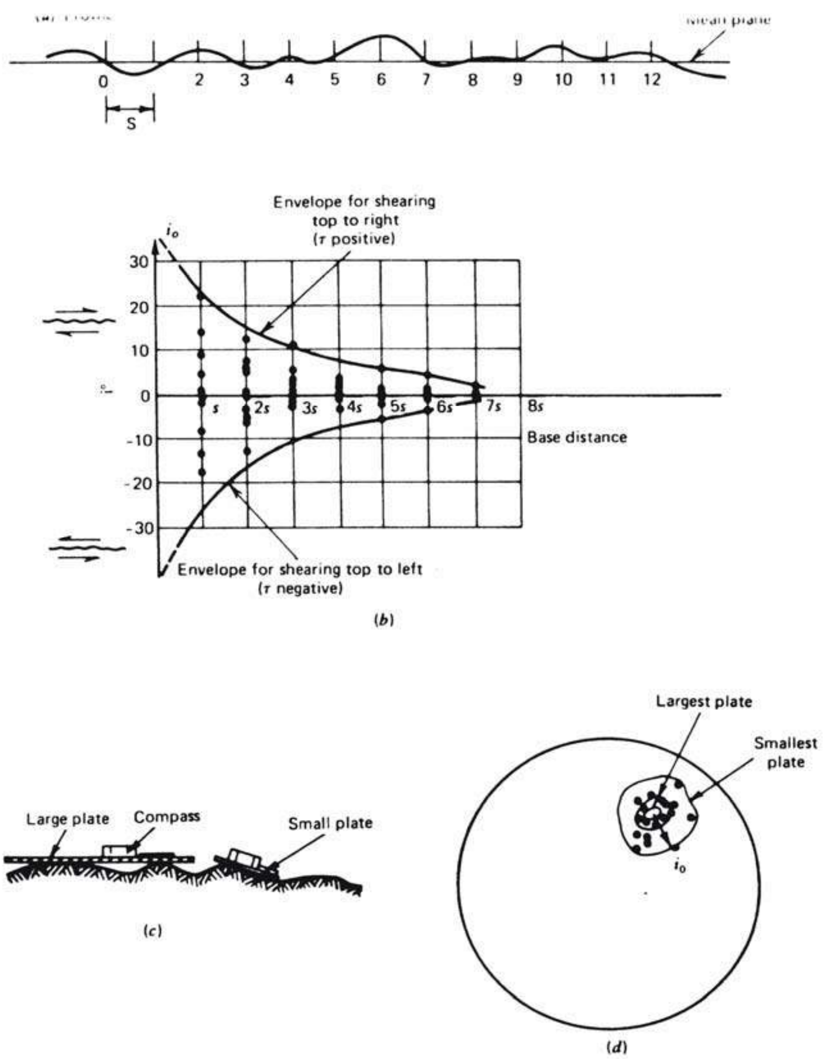


Fig. C-7. Rengers' analysis of roughness. (a) A rough surface. (b) Envelopes to roughness angles as a function of base length. (c) Dip and dip direction measurement. (d) Plot of (c) in stereogram. (after Fecker and Rengers, 1971)

C.2.6 Fitting - non-fitting surfaces

In the forgoing chapters is assumed that both discontinuity walls fit, e.g. are complementary and the asperities from one discontinuity wall fit the cavities in the other wall and vice versa. If discontinuity walls do not fit, the *i-angle* is reduced. This is shown by the Rengers' method (Fig. C-7b). The larger the diameter of the plate, hence, a lower fit, the lower the *i-angle*.

C.2.7 Estimating roughness and roughness profiles

The foregoing show that a simple theory of shear strength based on material friction and measured roughness angles is not satisfactory for natural discontinuities. The theory does not consider all parameters (e.g. deformation, etc.) and measuring of roughness of a natural discontinuity plane is not realistic on a large scale. Estimating the contributions of roughness to the shear strength of a discontinuity is an alternative approach. This is easiest done if it is divided in ranges. A simple practical division can be established by the naked eye to give (Fig. C-13): 1) roughness that cannot be seen and 2) visible roughness that can be estimated by visual comparison with standard roughness profiles[11]. A large advantage of this method is that it does not need an

[11] Visible roughness is that which can actually be seen. Light reflection characteristics (luster) partly depend on roughness but on a far smaller scale, and are not included in visible roughness. Measuring of roughness can be done by means of a laser-profile meter,

extensively exposed discontinuity plane. It is often enough to see traces of the discontinuity in different directions.

C.2.7.1 Barton JRC concept

The standard roughness profiles (Fig. C-8) and an empirical relation that relates the profiles to shear strength values is developed by Barton et al. (1977). Barton introduced the JRC standard roughness profiles as a means to be able to describe roughness profiles, which are also related to shear strength:

$$\frac{\tau_{peak}}{\sigma'_n} = \tan\left[JRC * \log_{10}\left(\frac{JCS}{\sigma'_n}\right) + \varphi_r\right] \qquad \text{[C-1]}$$

τ_{peak} = peak shear strength σ'_n = effective normal stress on discontinuity plane
JCS = discontinuity wall compression strength JRC = discontinuity roughness coefficient
φ_r = residual friction angle

A problem with the JRC roughness profiles is that they do not include stepped surfaces and require measurement of the residual friction angle. In addition, in the author's experience it is often very difficult to establish the proper JRC number visually.

by photogrammetry or, for larger scale roughness with rulers, theodolites, etc.. The range for visible roughness is normally limited to a maximum. Roughness on a larger scale than the maximum, for example, large waviness in bedded rocks, implies a change in dip and hence the unit should be split in more geotechnical units, so that the dip is uniform within each unit.

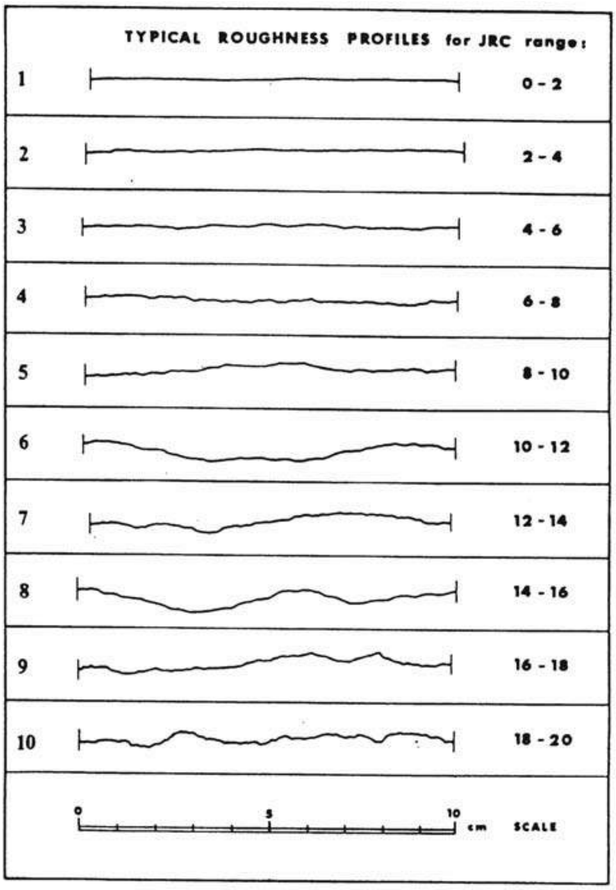


Fig. C-8. JRC graphs (after Barton, 1977)

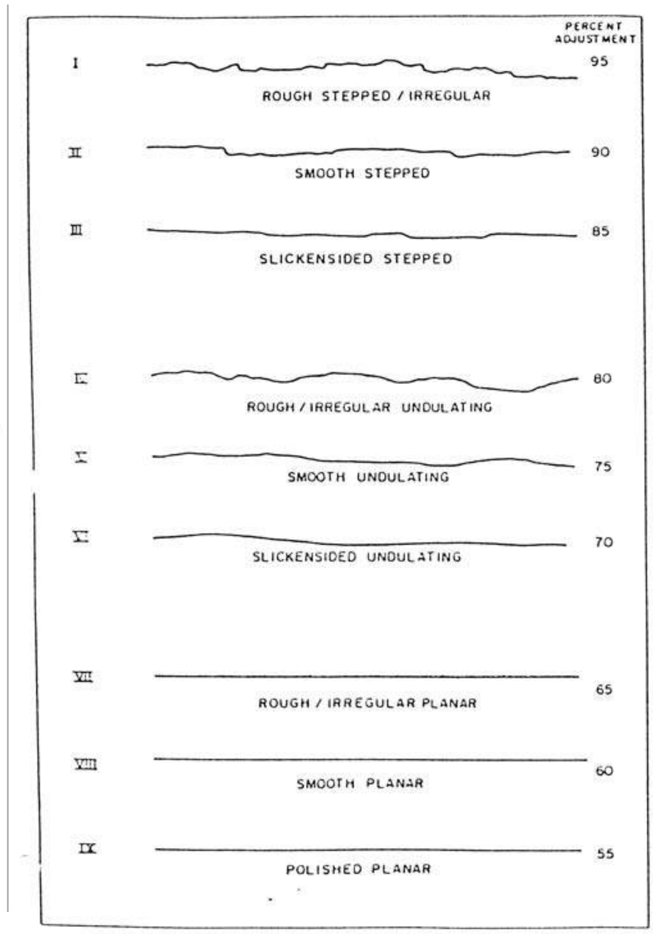


Fig. C-9. Laubscher's roughness graphs (after Laubscher, 1990)

C.2.7.2 Laubscher's roughness curves

Laubscher (1990) developed a thorough set of descriptive terms for roughness of discontinuities with factors rating the influence on the stability of underground excavations. The descriptions used by Laubscher are partly based on the profiles published by ISRM (1978b, 1981a). The roughness is divided in roughness that cannot be seen, but can be felt by using fingers (tactile roughness), and roughness that can be seen, which is described visually at different scales (Fig. C-9). This set of descriptions is used in Laubscher's classification system for underground excavations (ch. N.4.3).

C.2.7.3 SSPC roughness determination

An empirical relation between tactile and visible roughness based on the ISRM (1978b, 1981a) profiles, and the friction along a discontinuity plane resulting from roughness, has been developed for the SSPC classification system for slopes

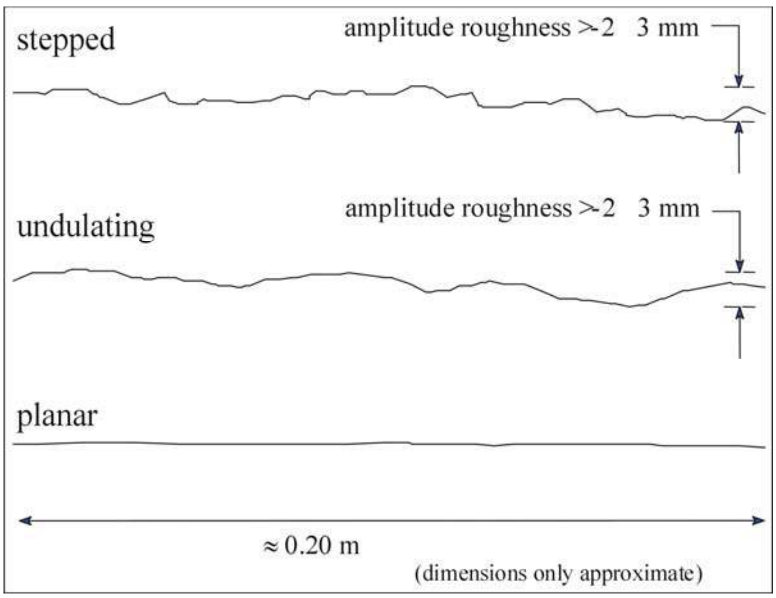


Fig. C-10. Small-scale roughness profiles for SSPC (after Hack, 1998)

(Hack, 1998). The roughness profiles of ISRM (and Laubscher) have been modified. Tactile roughness is to be distinguished by feeling with fingers and described in three classes: rough, smooth, and polished. The small-scale roughness visible on an area of 20 x 20 cm^2 of the discontinuity surface is described in three classes: stepped, undulating, and planar. Representative example profiles including scales are provided in Fig. C-10 (small-scale) and Fig. C-11 (large-scale). The large scale roughness determined on an area larger than 20 x 20 cm2 but smaller than 1 x 1 m2, is described in five classes: wavy, slightly wavy, curved, slightly curved, and straight.

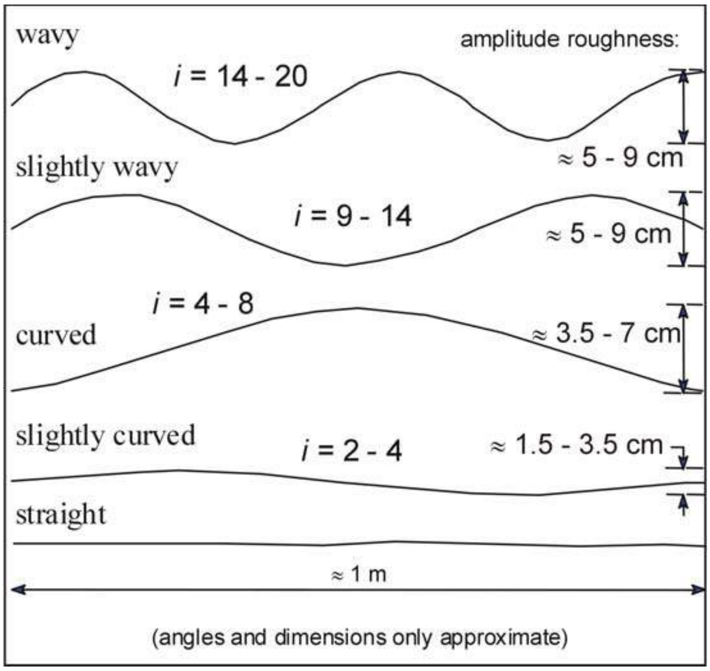


Fig. C-11. Large-scale roughness profiles for SSPC (after Hack, 1998)

C.2.8 Stepped roughness planes

Stepped roughness planes are planes on which asperities with sides occur for which applies that φ + *i-angle* $\geq 90°$. These asperities are normally denoted as steps on the discontinuity plane, although the *i-angle* does not have to be 90°. If a step is present perpendicular to the direction of sliding then either the step has to be sheared off before the block can slide or so much dilatancy deformation has to be possible that the block can slide over the asperity. Steps on surfaces often prohibit sliding. Many of the empirical relations do not consider this, however the standard profiles by ISRM (1978b, 1981a), Laubscher (1990) and SSPC do.

C.2.9 Anisotropic roughness

Roughness of a surface can be anisotropic (e.g. ripple marks, Fig. C-12, striation, etc.), and thus the shear strength will be direction dependent. Theoretically, the roughness should therefore be measured in different directions. The number of different directions that should be measured depends on the type of the roughness. For example, it is sufficient to measure the roughness in one direction only for a regular striation; perpendicular to the striation the contribution to the shear strength of the roughness due to the striation is maximum while parallel to the striation no influence of the striation is present. For less regular surfaces the number of directions in which the roughness has to be measured increases, but roughness in all directions will be again about equal for a fully irregular surface and one measurement will be sufficient. Alternatively, the roughness can be measured only in the direction in which shear displacement over the discontinuity is expected.

Fig. C-12. Ripple marks on sand dunes. Fossilized these will give an anisotropic roughness on a discontinuity plane

In practice, it will mostly be sufficient to determine the roughness in one direction or in two perpendicular directions only; parallel and perpendicular to the maximum roughness. The accuracy of roughness determination and subsequent translation into friction angles is, in general, not high enough to justify the determination of roughness in more than two directions.

C.2.10 Discontinuity history

The history and origin of a discontinuity have an influence on the shearing characteristics of the discontinuity. If movement along a discontinuity has taken place in the past then two situations are possible:

1. Due to the movement, asperities have sheared off completely and the roughness of the discontinuity is nil. The roughness of the discontinuity is determined as found and thus the history is included in the assessment of the roughness.
2. The movement happened without shearing off the asperities or the asperities are only partially sheared off. The resulting discontinuity has then a non-fitting roughness profile and the dilatancy necessary to allow further displacement is lower (ch. C.2.5, Rengers, 1971). In this situation, testing might help to guess an accurate value of the shear strength or an estimate can be made by which amount the necessary dilatancy (or *i-angle*) is reduced due to the displacement. For example, the material friction only governs the shear strength of a discontinuity that is not fitting at all.

Both breaking of asperities and non-fitting of asperities reduce the friction due to the roughness considerable and subsequent the shear strength along a discontinuity. Hence, it is extremely important to determine whether any displacement along a discontinuity has taken place previously, for example, due to tectonic movements. The shear strength along the discontinuity will then be considerably lower. For the same reason it is very important in building on or in a rock mass to keep the movements in the rock mass to the minimum possible. Any displacement allowed will result in less perfect fitting asperities and/or may result in breaking of asperities. In both cases, the shear strength will be reduced strongly.

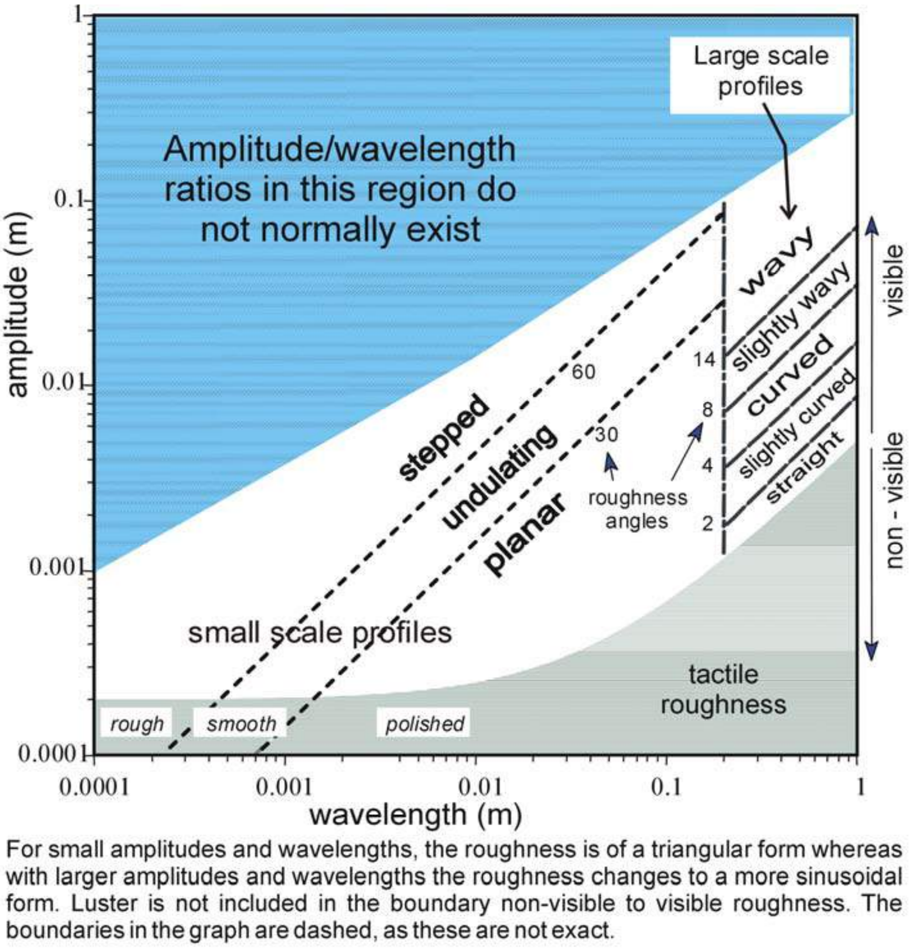


Fig. C-13. Interpretation of regular forms of roughness as function of scale and angle (after Hack, 1998)

C.2.11 Conclusions

A summary of the different ranges for roughness with wavelengths and amplitudes for regular forms of roughness is shown in Fig. C-13. The boundary lines are dashed, as these are not exact. The wavelengths and amplitudes for the roughness profiles are an indication only. The figure is an attempt to combine normally occurring different types, scales, and measuring methods of roughness and is not expected to cover all forms of roughness[12].

If the roughness is direction-dependent, the roughness should be assessed in two perpendicular directions. If movement along a discontinuity has taken place, in the past then the influence of this movement on the shear strength along the discontinuity should be quantified by estimating the remaining *i-angle* or the discontinuity has to be tested.

[12] For example, stylolites in limestones or very coarse-grained rocks (e.g. porphyritic granites) could plot in the region that is indicated as 'do not normally exist'.

C.3 Alteration of discontinuity wall

The discontinuity wall is the rock material directly adjacent to the discontinuity. The material if in contact, will determine the shear strength along the discontinuity. Determining the shear strength characteristics of discontinuities requires that the joint wall condition or joint wall strength should be established. Various authors have commented on the influence of the strength of the discontinuity wall on shear strength (Bandis, 1990, Barton et al., 1973a, 1973b, 1976b, 1977, 1985, Laubscher, 1990, Fishman, 1990, Rengers, 1970, 1971, Rode et al., 1990). Often the 'quality' (strength) of the discontinuity wall is lower than the intact rock strength. The decrease in strength may have been caused by weathering features, brought about by chemically charged water percolating through discontinuities that reacted with the wall, etc. The thickness of the layer having a lower strength may range from microscopic thickness up to many centimeters or more. In shearbox tests the discontinuity wall strength is incorporated in the results, however, shearbox tests can only be done on samples of limited size.

C.3.1 Rebound tests

Rebound tests are a method that may be suitable to assess discontinuity wall strength. The best-known rebound test is the Schmidt hammer[13] (ISRM, 1978a, 1981a, Rode et al., 1990, Stimpson, 1965). Other rebound measurements are based on a hammer, ball, or piston that drops from a certain height on to the surface to be measured (Equotip, 1977, Hack et al., 1993a, ISRM, 1978a, 1981a, Pool, 1981, Price et al., 1978, Stimpson, 1965, Verwaal et al., 1993). The rebound of the piston, hammer, or ball after hitting the surface is dependent on the elastic parameters of the tested material and on the strength of the material at the surface of the discontinuity. The crushing of surface asperities and surface material, which dissipates energy, causes this latter effect. Most of the rebound tests reported in the literature are not developed to measure the discontinuity wall strength but to measure the intact rock strength.

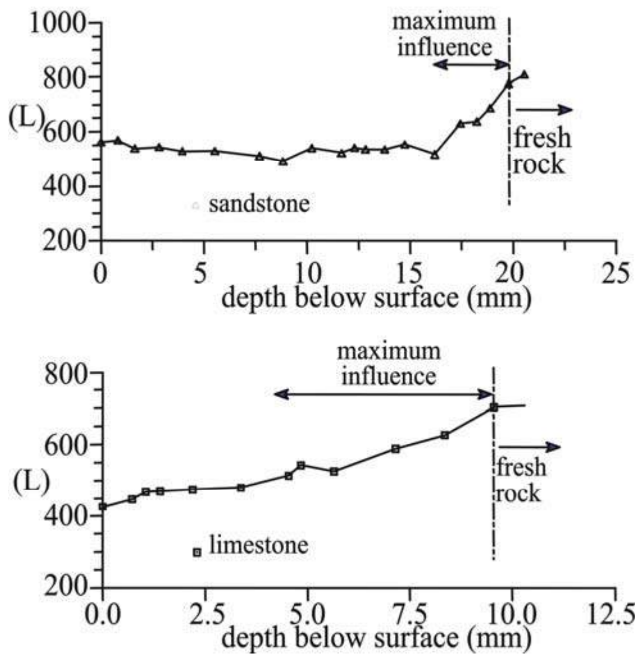


Fig. C-14. Equotip rebound values on weathered discontinuity walls progressively ground down to fresh rock (after Hack et al., 1993a)

The standard form of the Schmidt hammer releases so much energy over such a large area that in most rocks a layer of up to centimeters depth influences the measurement. The ball rebound device (Pool, 1981, Price et al., 1978) and the Equotip device (Equotip, 1977, Hack et al., 1993a, Verwaal et al., 1993) release considerably less energy and may therefore be better methods of establishing discontinuity wall parameters. Fig. C-14 shows that the depth of the material influencing the Equotip measurement is maximum about 5 mm and Fig. C-15 shows the correlation between Equotip rebound and unconfined compressive strength (UCS). This correlation suffers, however, from scale effects, the Equotip testing a much smaller volume of rock than the UCS. Thus, exact correlation is unlikely to be achieved.

C.4 Discontinuity infill material

The importance of discontinuity infill was recognized in nearly all rock geotechnical disciplines and many different description systems for discontinuity infill have been proposed, often as part of a rock mass classification system (Barton et al., 1974, 1980, Bieniawski, 1973, 1976, 1989, Holtz et al., 1961, Lama, 1978, Laubscher, 1990, Tulinov et al., 1971, etc.). The type of infill material and whether the walls of the discontinuity will be in contact or not during shearing have a very strong influence on the shear strength characteris-

[13] Different designs of Schmidt hammers for different impact energies exist. 'L' and 'N' design are most commonly used in the field.

tics. Types of infill materials may consider being cemented, non-softening or softening under influence of water, deformation, or shear displacement. The material itself is reported to be generally of minor influence (Barton et al., 1974, 1980, Tulinov et al., 1971). Testing the shear strength of discontinuities that include infill material is very difficult. Proposals for testing discontinuities with disturbed infill material have been made by Goodman (1989), but only in-situ tests are a reliable means to test shear strength of undisturbed filled discontinuities. In most situations, estimating the shear strength is therefore the only option. Because the infill material itself is of minor importance, the number of classes necessary to describe a filled discontinuity and to estimate the shear strength parameters can be done with a relatively simple and limited set of classes.

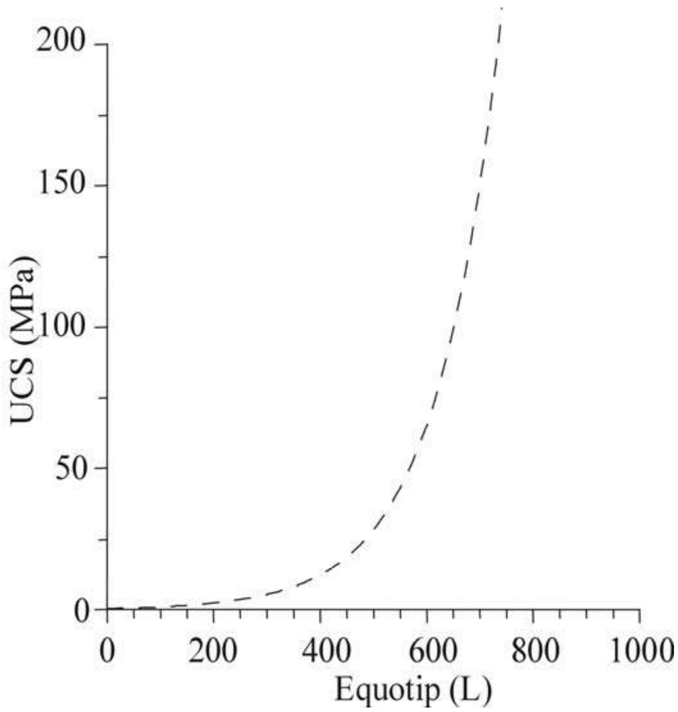


Fig. C-15. UCS vs. Equotip (after Verwaal et al., 1993)

C.4.1 Aperture or width of discontinuity

A parameter often used in the description of discontinuities in relation to infill, is aperture or width of the discontinuity (Bieniawski, 1973, 1976, 1989, Brekke et al., 1972, ISRM, 1981a, etc.). In these descriptions an aperture or width of a discontinuity larger than zero, implies that the discontinuity is filled with material over a certain distance with a continuous layer (band) of infill material with more or less the same thickness, or is completely open without any contact between the walls of the discontinuity. The latter, completely open discontinuities, can occasionally be observed at surface, and are usually vertical. Completely open discontinuities are obviously not common in other situations, as the stresses working on the discontinuities will cause closure. Although aperture is included in some descriptions, other research has shown that aperture itself is often of minor importance for the shear strength characteristics. The shear strength characteristics of a smooth, planar discontinuity can be roughly divided in three ranges (Phien-wej et al., 1990):

1. If there are points of contact between the discontinuity walls, the shear strength is mainly determined by the properties of the discontinuity walls.
2. If the infill thickness in the discontinuity is less than about the grain size of the intact rock grains or minerals in the discontinuity walls or of the grain size of the infill material, the shear strength of the discontinuity is that of the infill but influenced by the discontinuity wall material.
3. If the infill thickness is larger than the grain size of the discontinuity wall and the grain size of the infill, the shear strength is that of the infill material.

Aperture for irregular discontinuity walls is therefore meaningful only if the aperture of a discontinuity is related to the amplitude of the roughness of the discontinuity walls or is related to the grain or mineral size of the infill or rock material. The above does not apply to non-filled discontinuities (such as karstic discontinuities) for which aperture can be important.

C.4.2 Origin of a discontinuity or origin of infill material

Origin of discontinuity

Some classification systems describe discontinuity infill material based on the origin of the discontinuity (Brekke et al., 1972) because the origin of a discontinuity can have a relation with the shear strength characteristics of the discontinuity. For example: bedding planes will often be a potential discontinuity because the plane is formed by more softer or easier weathered materials (e.g. clay) than the rest of the rock mass, whereas tectonic joints will normally have an infill material consisting of weathered intact rock material. This method of description implies the risk of completely wrong assessments. The author has often observed bedding planes that did not contain any clay infill material and observed tectonic joints filled with clay material that was not weathered intact rock. It could even have been that the clay material of the bedding plane had been washed out of the bedding plane and accumulated in the joints. In this situation, shear strength parameters determined based on the origin of the discontinuity would be erroneous.

Origin of infill material

Origin of infill material can be used to determine the shear strength of the discontinuity infill material, but requires mostly a detailed analysis of the infill material (Welsh, 1994). The practical advantage is therefore limited and mostly it is more efficient to relate the infill material to shear characteristics directly than via establishing the origin of the infill.

C.4.2.1 Infill description for estimating shear strength of discontinuities

A reasonable accurate description is given below. The infill and thickness of the infill is described based on its shear strength characteristics. The classes roughly follow those established by Laubscher (1990). The system is relatively simple and no expert knowledge of geology is necessary.

No infill, cemented or not cemented

The first distinction to be made is between no infill, cemented, cemented infill, or non-cemented infill.

No infill

'No infill' describes a discontinuity that may have coated walls but no other infill. For most discontinuity surfaces friction is virtually independent of the minerals of the intact rock. This has been established by many researchers doing tests on smooth, planar surfaces to obtain φbasic (Hack et al., 1995). Apparent cohesion of the discontinuity walls does depend on the type of mineral but at low levels of low normal stress, apparent cohesion is less important. For mineral coatings on discontinuity walls, the same applies (Welsh, 1994, Hack et al., 1995). Therefore, one class describing the shear strength of a non-cemented, non-filled discontinuity is sufficient.

Cemented discontinuity or cemented infill

A cemented discontinuity or a discontinuity with cemented infill has higher shear strength than a non-cemented discontinuity if the cement or cemented infill is bonded to both discontinuity walls. If there is no cement bond between the discontinuity walls or between the cemented infill and one or both discontinuity walls, the discontinuity behaves as a non-cemented, non-filled discontinuity. Two classes should be distinguished for discontinuities with a cement bond or with cemented infill bonded to both discontinuity walls:

1. The cement or cemented infill and bonding to both discontinuity walls are stronger than the surrounding intact rock (failure will be in intact rock), and
2. The cement or cemented infill and bonding are weaker than the surrounding rock but still stronger than a non-filled discontinuity.

Those that are stronger than the surrounding rock do not need to be considered as a discontinuity, those weaker are described with the class 'cemented/cemented infill'.

Infill

Non-softening and softening infill

A major distinction should be made between non-softening and softening material for discontinuities without cement but with infill material (Barton, 1974, 1980, Laubscher, 1990, Tulinov et al., 1971).

Non-softening

Non-softening infill material is material that does not change in shear characteristics under the influence of water nor under the influence of shear displacement. The material may break but no greasing effect will occur. The material particles can roll but this is considered to be of minor influence because, after small displacements, the material particles generally will still be very angular.

Softening

Softening infill material will under the influence of water or displacements, attain in lower shear strength and will act as a lubricating agent.

Both classes of softening and non-softening infill material can be further sub-divided in classes according to the size of the grains in the infill material or the size of the grains or minerals in the discontinuity wall. The larger of the two should be used for the description (Tulinov et al., 1971, Laubscher, 1990).

Gouge

The so-called 'gouge[14] filled' discontinuities are a special case. Gouge filled discontinuities are often the larger discontinuities in a rock mass such as faults. Gouge layers are relatively thick and continuous layers of infill material, mainly consisting of clay but often also containing rock fragments. The common feature of gouge is the presence of clay material that surrounds the rock fragments in the clay completely or partly, so that these are not in contact with both discontinuity walls. The initial shear strength of such a discontinuity will be that of the clay. If the gouge is thicker than the amplitude of the roughness of the discontinuity walls, the clay material governs the shear movement. If the thickness is less than the amplitude of the roughness, the shear strength will be influenced by the wall material and the discontinuity walls will be in contact after a certain displacement. For further displacement, the friction along the discontinuity walls in combination with the clay infill and the friction of the rock fragments in the gouge govern the shear strength.

Flowing material

Very weak and not compacted infill in discontinuities flows out of the discontinuities under its own weight or as a consequence of a very small trigger force (such as water pressure, vibrations due to traffic or the excavation process, etc.).

C.5 Weathered discontinuities

In most rock materials weathering of a discontinuity results in weakening of the discontinuity wall and in the formation of infill material in the discontinuity. The shear strength of such a weathered discontinuity is determined more by the presence of infill material than by the reduction of the shear strength due to the weakening of the discontinuity walls. Reduction of the shear strength of the discontinuity walls becomes important only if the weathered material is flushed out of the discontinuity completely. However, usually a thin layer or coating of weathered material stays behind in the discontinuity. For example, in carbonate rock masses containing some clay, it is often found that the discontinuity walls are slightly weathered and that a thin clay infill is found in the discontinuities, this being all that remains of the weathered rock material. The remaining infill significantly determines the shear strength of the discontinuity. Weathering of a rock mass including discontinuities is further discussed in ch. E.1.

C.6 Discontinuity karst features

Karst features have been found to be of importance in rock engineering. The open holes considerably weaken the rock mass. Whether these have an influence on the shear strength of discontinuities depends on the type of the discontinuity. If the shear strength along the discontinuity is partly or completely due to cohesion, the shear strength is reduced because the contact surface is reduced. If the shear strength is due to friction, the reduction in surface area does not matter as the normal stress and consequently the shear strength will increase equivalently.

C.7 Effect of water pressure in discontinuities

Underground excavations

Water pressures in discontinuities reduce the shear strength of the discontinuities, which is the reason that many classification systems for underground excavations include a separate parameter quantifying this influence. In addition, in numerical (or analytical) calculations the presences of water and water pressures have to be included in the calculations.

[14] 'Gouge' is an ancient mining term that implies soft, easily extracted material (see glossary, page 223).

A difference should be made whether the water is allowed to flow in the direction of the excavation (draining to the excavation) or whether the excavation is made impermeable (by for example grouting or shotcrete) for water flowing out of the rock mass. In the first case water is a definite hazard as the flowing water causes additional pressures on the rock blocks in the direction of the excavation, secondly water pressures reduce the effective rock stress on the discontinuities shear plane and, hence, reduce the shear strength along the discontinuities. If drainage towards the excavation is prohibited, the water is static and water pressures should be taken into account only for the reduction in effective stress and consequent reduction of the shear strength.

Slope stability

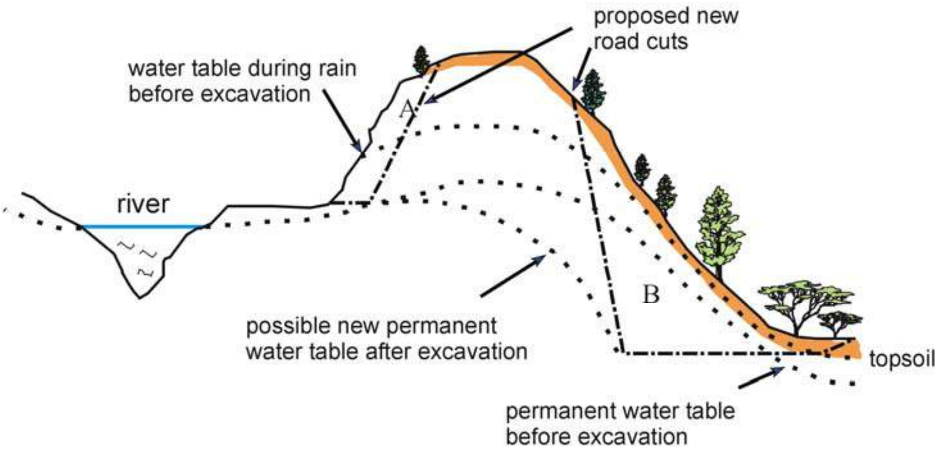


Fig. C-16. New slopes with different conditions with water table

The influence of water pressures on slope stability may be less important than often assumed. Consider the two new slopes in Fig. C-16, cuts A and B. For both slopes, the water table will adjust due to draining so that no intersection of the water table and the slope will occur. If complete draining (for example, due to a high hill with large quantity of water supply) is not possible, the new slope cut would intersect a permanent water table. Clearly, such a design of a slope would be considered bad engineering and drainage measures would be taken to prohibit this situation. The only situation that would occur and will require water pressures to be taken into account is if the water table is raised temporarily during rain (cut A). For this situation, it should be recognized that infiltration in the rock mass is often more difficult than draining. This is because of two reasons. Often the hill at the top of the slope is covered with a layer of topsoil. The topsoil will normally have permeability lower than the permeability of the rock mass. Secondly, the discontinuities exposed in the slope cut are more open due to stress relief than the discontinuities deeper in the rock mass. Both effects reduce and often prohibit the built-up of water pressures. In analytical and numerical calculations, the presence of water and water pressures should be included, however, disputable is whether the water pressures should be taken very high. In classification systems, it is questionable whether water has to be included. Some of the classification systems do not include a (separate) parameter for water but some do (see classification systems).

C.8 Discontinuity shear strength tests

Testing the shear strength of a discontinuity can be done by field and laboratory tests. In practice, the various tests contain serious shortcomings and only give crude estimates of reality. All non-in-situ field and laboratory tests on discontinuities are hampered by difficulties in sampling and executing of the tests. The problems involved in testing of shear strength have been commented on by many authors (Goodman, 1989, Cunha, 1990, 1993, Hack, 1998).

C.8.1 Laboratory direct shear box test

A sample including a discontinuity is fitted in two metal half boxes with the discontinuity plane parallel to the top of the boxes (Fig. C-17). Normal stress and shear stress are applied and the horizontal and vertical displacements between the two boxes are measured. The maximum size of the discontinuity surface is normally in the order of 7 x 5 cm^2, which implies that only material friction and part of the small-scale roughness can be tested. Larger size testing equipment is used, however, very costly to make and maintain.

Obtaining undisturbed samples is mostly an illusion. Infill material is nearly always disturbed and due to vibrations of, for example, sawing equipment and due to transport, often-small asperities are damaged. The difficulties with obtaining suitable and undisturbed samples increase dramatically if larger samples are wanted.

Fig. C-18 shows a sample prepared for a shear box test. A roughness profile of this sample is shown in Fig. C-19 and the shear strength versus shear displacement is shown in Fig. C-20. Note the steps in the roughness profile and the sharp decrease in shear strength after 1 mm displacement.

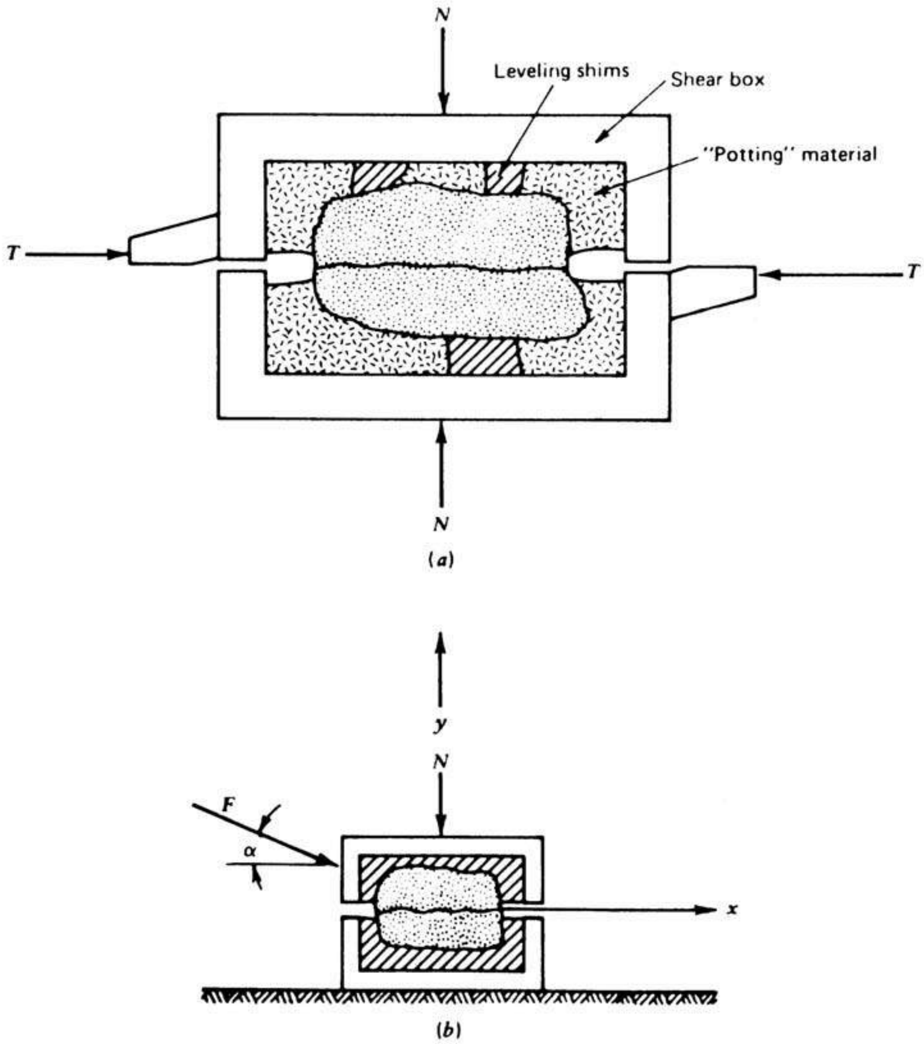


Fig. C-17. Direct shear testing (a) The arrangement of the specimen in a shear box (b) A system for testing with inclined shear force to avoid moments (after Goodman, 1989)

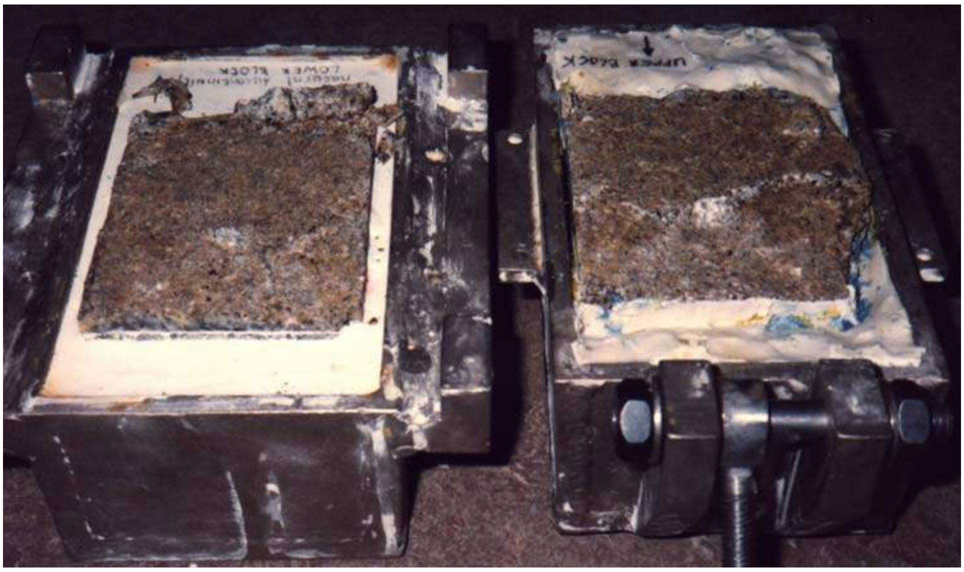


Fig. C-18. Prepared sample for shear box test. The sample halves are imbedded in cement. Test direction is in the length direction of the sample. (after Baardman, 1993)

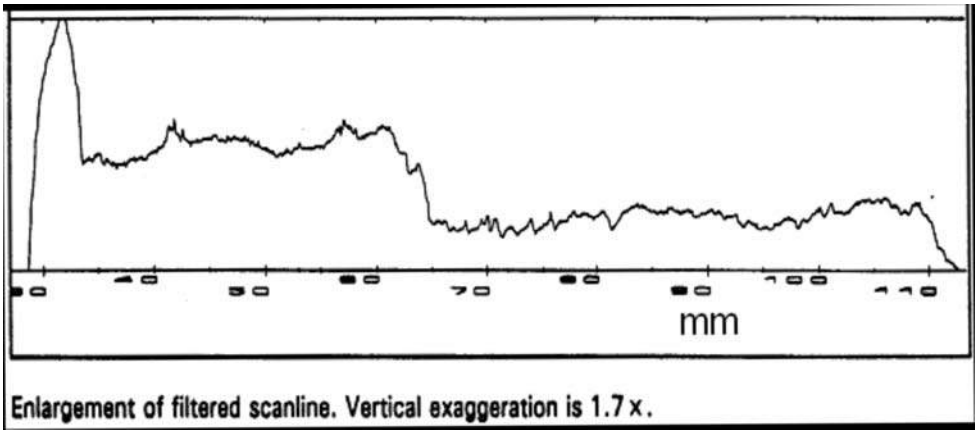


Fig. C-19. Laser roughness profile of sample in photo Fig. C-18. Profile measured in length direction of sample. Note the steps at 20 and 60 mm in the roughness. (after Baardman, 1993)

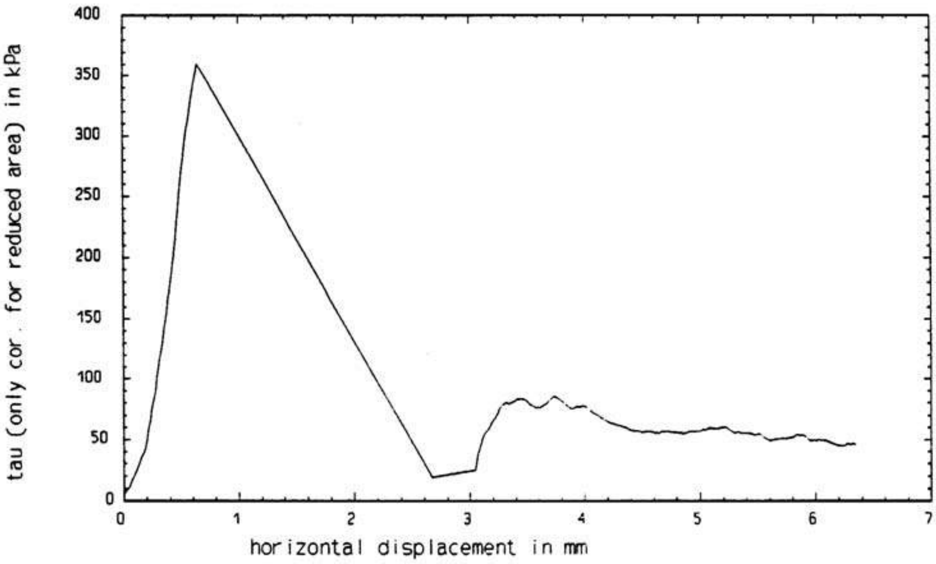


Fig. C-20. Shear strength (tau) vs. shear (horizontal) displacement of the sample in Fig. C-18 (after Baardman, 1993)

C.8.2 Large scale field direct shear test

In the field or in a tunnel a test similar to the laboratory shearbox tests can be set up on in-situ discontinuity planes (Fig. C-21). The advantages are that the problems with obtaining undisturbed samples are avoided and that (far) larger discontinuity surfaces can be tested. Tests up to about 0.6 x 0.6 m^2 are often done. The disadvantage is that the tests are very costly.

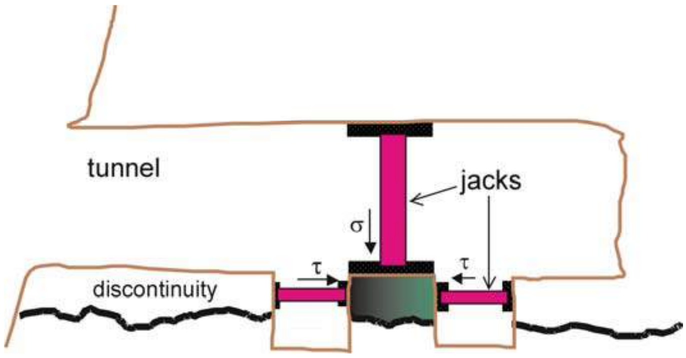


Fig. C-21. Large-scale direct shear test

C.8.3 Tilt test

A simple test to estimate the shear strength parameters of a discontinuity is the 'tilt test'. Two pieces of rock containing a discontinuity are held in the hand with the discontinuity horizontal. The sample is slowly tilted until the top block moves (Fig. C-22). The angle with the horizontal at movement is the so-called 'tilt angle'. The 'tilt angle' is not directly comparable to the parameters measured in a direct shear test. Only if no cohesion (apparent or real) is present, the asperities do not break, and the two blocks contain a fitting discontinuity the 'tilt angle' equals $\varphi_{disc.wall}$ + *i-angle*. The measured 'tilt angle' will equal $\varphi_{disc.wall}$ if the discontinuity in the sample is completely non-fitting. In other cases, the tilt angle will be a combination of the discontinuity roughness properties.

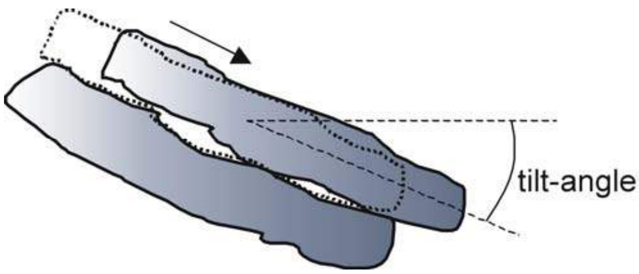


Fig. C-22. Tilt test

C.9 Sliding criterion

Based on back analyses of slope stability a sliding criterion has been developed to estimate easily the shear strength of a discontinuity (Hack et al., 1995, 1998). The discontinuity is characterized following Table C-1. The roughness is characterized by visually estimating large-scale roughness following Fig. C-11, small-scale roughness by tactile roughness (rough, smooth, and polished) and by visual estimation following Fig. C-10, infill material, and presence of karst. The different factors for the different characteristics are multiplied and this results in a so-called 'sliding angle':

$$\varphi_{sliding\ angle} = \frac{Rl*Rs*Im*Ka}{0.0113} \qquad [C\text{-}2]$$

The 'sliding angle' is comparable to the tilt test idea only the scale is larger and infill and karst are considered. The sliding angle gives the maximum angle under which a block on a slope is stable. The 'sliding criterion' has been developed on slopes between 2 and 25 m. The 'sliding criterion' applies for stresses that would occur in such slopes, hence, in the order of maximum 0.6 MPa.

CONDITION OF DISCONTINUITY			factor
Roughness large scale (Rl) (visual area > 0.2 x 0.2 and < 1 x 1 m2)	wavy		1.00
	slightly wavy		0.95
	curved		0.85
	slightly curved		0.80
	straight		0.75
Roughness small scale (Rs) (tactile and visual on an area of 20 x 20 cm2)	rough stepped/irregular		0.95
	smooth stepped		0.90
	polished stepped		0.85
	rough undulating		0.80
	smooth undulating		0.75
	polished undulating		0.70
	rough planar		0.65
	smooth planar		0.60
	polished planar		0.55
Infill material (Im)	cemented/cemented infill		1.07
	no infill - surface staining		1.00
	non softening & sheared material, e.g. free of clay, talc, etc.	coarse	0.95
		medium	0.90
		fine	0.85
	soft sheared material, e.g. clay, talc, etc.	coarse	0.75
		medium	0.65
		fine	0.55
	gouge < irregularities		0.42
	gouge > irregularities		0.17
	flowing material		0.05
Karst (Ka)	none		1.00
	karst		0.92

Table C-1. Discontinuity characterization for 'sliding criterion' (after Hack et al., 1995, 1998)

D SETS OF DISCONTINUITIES VERSUS SINGLE DISCONTINUITIES, CONCEPT OF DISCONTINUITY SPACING

To be able to describe discontinuity properties it is necessary to define whether discontinuities can be grouped in a 'set' or should be treated as a single phenomenon. Determining the parameters for a 'set' of discontinuities requires a form of averaging of the parameters of individual discontinuities. This can be done by various methodologies. These are briefly described, with their advantages and disadvantages. All parameters are determined separately for each geotechnical unit.

D.1 Discontinuity sets

A description of each single discontinuity in a rock mass would lead to an unreasonable quantity of work; calculations with hundreds or thousands of discontinuities are very time-consuming. However, discontinuities occur often as a regular feature, e.g. bedding planes, cleavage, regular sets of joints or fractures, etc. Therefore, a normal procedure in discontinuous rock mechanics is to group discontinuities in sets (or families). All discontinuities in a set are then considered to have broadly the same characteristics such as orientation, spacing, roughness, etc.. Shear zones or faults may also occur in a set but occur usually as single phenomena on the scale of engineering works.

D.2 Sets of very widely spaced discontinuities or single discontinuities

If the discontinuities in a set are very widely spaced (for example, if the spacing is considerably more than the dimensions of the tunnel, slope, borehole, etc., or if the discontinuities are very widely spaced compared to the dimensions of the geotechnical unit) then each discontinuity of the set may be treated as a single discontinuity in subsequent analyses. For each single occurring discontinuity, all characteristics should be described and measured so that in further calculations these can be dealt with individually.

D.3 Grouping discontinuities and determining characteristic discontinuity properties and parameters

Grouping discontinuities in sets based on their properties and finding the characteristic properties and orientation of a discontinuity set can be done with various methods.

D.3.1 Geological and structural analyses

A geological and structural geological analysis consists of the determination of the discontinuity properties in various exposures, the relations between the different discontinuities and discontinuity sets, and the origin and history of the discontinuity sets. These are plotted on plans and sections representing the volume of the rock mass or directly into a three-dimensional GIS. Stereographic representations can be used (Maurenbrecher et al., 1990, 1995, Phillips, 1972). A geological and structural geological analysis will allow in many situations for prediction of the properties of discontinuity sets in-between the exposures. Fig. D-1 shows a simple example where the bedding is estimated to be about horizontal at the location of the new slope. A complete discussion

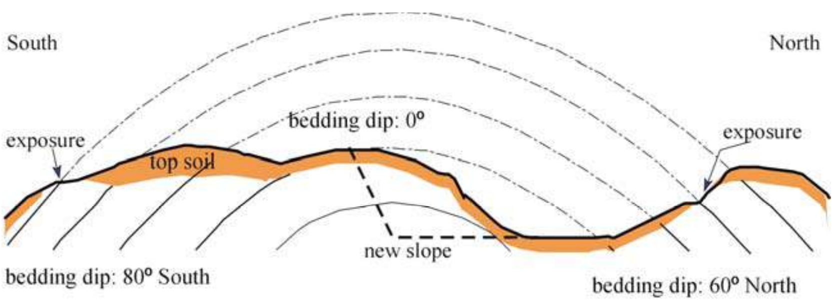


Fig. D-1. Geological and structural geological analyses to obtain discontinuity properties on how to determine discontinuity properties from structural and geological analyses is outside the scope of these notes and can be found in books on geology and structural geology.

D.3.2 Scanline method

An often-applied practice to determine characteristic discontinuity properties in an exposure is by a form of averaging of the properties of all individual discontinuities crossing a scanline. Scanlines are normally positioned at an easily accessible location on the exposure, mostly about 1.5 m above the ground and horizontally oriented. This implies that: 1) depending on the height of the exposure only a part (and often very small part) is represented in the measurements, 2) discontinuity sets with a (very) large spacing may be missed, and 3) all discontinuities with a trace (near-) parallel to the scanline are under-sampled. The last-mentioned is, in particular for slope stability, a large shortcoming, as these discontinuities may be the planes of instability for plane sliding.

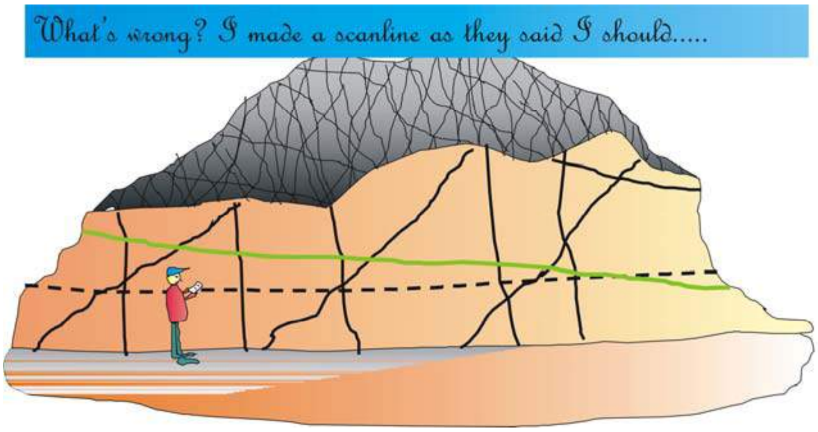


Fig. D-2. Scanline

D.3.3 Exposure - measuring and averaging discontinuity properties and parameters

If discontinuities occur as a set, the average orientation can be found mathematically or by stereo projection methods and subsequent contouring (Davis, 1986, Hoek et al., 1981, Mardia, 1972, R.D. Terzaghi, 1965). The characteristic properties of each discontinuity set are the average of the properties of each measured discontinuity belonging to that set. A disadvantage of contouring is that it may be difficult to distinguish between the different discontinuity sets in a stereo plot. Furthermore, an important discontinuity set may be missed out or underrated in importance because the discontinuity spacing is large. In a stereo plot, such a discontinuity set may be masked by a less important but far more often measured discontinuity set. A single discontinuity with an unfavorable dip and dip-direction for the slope can well determine slope stability while the mean or average direction and dip of the complete set do not indicate an unstable situation. This is no problem if all discontinuities in a rock mass are measured and plotted because the stereo scatter plot will show this critical discontinuity. Often, however, only the discontinuities along a scanline or in a particular (accessible) area of the exposure are measured. Then the most unfavorable discontinuity may well have been missed if it happened not to cross the selected scanline, not to be present in the area of the exposure where the measurements are done, or unfavorably oriented with respect to the exposure, e.g. discontinuities near parallel to the face of the exposure. The errors, which may affect the results of stereographic projection methods to determine discontinuity sets and orientations, are discussed, in extenso, by R.D. Terzaghi (1965).

Making visually an inventory of the different discontinuity sets (based on orientation, spacing and the character of the discontinuity, e.g. infill, roughness, etc.) is a more proper method. A mean orientation value for a discontinuity set can then be calculated by using only those discontinuities that belong to the discontinuity set in a stereo plot or by a vector analysis. For this method, separate scanlines for each discontinuity set can be oriented in order to cross a maximum number of the discontinuities of the set being measured. Alternatively, all discontinuities belonging to a set in the whole exposure or in part of the exposure can be measured and analyzed. Experience shows, however, that scanlines or measuring large quantities of discontinuities in a part of the exposure are still likely to be done only on easily accessible parts of the exposure.

D.3.4 Exposure - studied assessment and interpreted properties and parameters

In a studied assessment to determine discontinuity properties in an exposure, those discontinuities that are most unfavorable for the engineering structure or if that is not a priori known, the discontinuities that are representative for the set are visually selected. In this selection is incorporated the whole exposed area (as this selection is done visually it does not matter whether the discontinuity is accessible or not) and the character of the discontinuity (infill, roughness, etc.). After the selection, the properties of the selected discontinuities are measured in detail in pre-selected locations[15]. In the opinion of the author based on experience this method gives an equal or better result than the results of extensive measurements of discontinuities for a statistical analysis. If extensive amounts of measurements of discontinuity properties and parameters have to be done, they are always done on a part of the exposure that is (easily) accessible whether representative for the rock mass or not.

material	unit	block size (length x width x height) (m)	block form
limestone/ dolomite	Falset lower Muschelkalk (Tg21), fresh	1 x 1 x 0.6	cubic
	Falset lower Muschelkalk (Tg21), moderately weathered	0.3 x 0.3 x 0.05	tabular
	Falset upper Muschelkalk (Tg23), fresh	0.3 x 0.3 x 0.2	cubic
	Falset upper Muschelkalk (Tg23), moderately weathered	0.1 x 0.1 x 0.02	tabular
	Falset Jura	2 x 2 x 2	cubic
shale	Falset middle Muschelkalk (Tg22), fresh	0.1 x 0.1 x 0.05	cubic
	Falset middle Muschelkalk (Tg22), highly weathered	0.02 x 0.02 x 0.003	flaky
sandstone	Falset Buntsandstone (Tg1)	5 x 5 x 20	tabular
	Falset Carboniferous	0.2 x 0.2 x 0.15	cubic
granite	Falset, fresh	2 x 2 x 1	cubic
	Falset, highly weathered	0.2 x 0.2 x 0.2	cubic
basalt	Location: Eden bury, UK	0.4 x 0.4 x 5	columnar
tuff	Indonesia		massive
It should be noted that block size and form are highly depending on (sometimes very localized) tectonic and weathering influences.			

Table D-1. Typical block size and form for various rock masses

Experiments (unpublished) to determine orientation and spacing of various discontinuity sets by scanline analyses and by studied assessments done by the author while employed in an underground mine showed that the results of a studied assessment are mostly equal or better than a scanline. The two methodologies resulted in nearly the same values if the discontinuity sets were clearly distinct and if done in small (maximum 2 x 2 m) tunnels with crosscuts allowing for scanlines in all directions (also along the roof). The scanline and statistical analyses often missed discontinuity sets if done in large tunnels or in tunnels without crosscuts (thus not allowing for scanlines in all directions), or if the sets were not clearly distinct or if the sets had a (very) large spacing. Different qualified engineers who also incorporated discontinuity type and properties in the analyses did the studied assessments and statistical analyses. Other observers made similar observations (Gabrielsen, 1990). It may be thought that a studied assessment for the determination of discontinuity properties would not be accurate enough, but it should be kept in mind that the variation of discontinuity properties in one discontinuity set is often so large that a high accuracy is not very important (ISRM, 1978b, 1981a).

D.3.5 Borehole cores

Grouping the discontinuities in sets and determining the mean or characteristic discontinuity properties and parameters of the sets can be done by the methods discussed for exposures. The discontinuity spacing measured in borehole cores may be affected by new discontinuities formed due to the stress relief because of drilling. The measured discontinuity spacing is then lower than in-situ. For borehole cores from a large depth, this effect is more severe than for cores from a relatively shallow depth. In addition, the drilling process itself or vibrations during drilling may cause the cores to be broken giving additional discontinuities (see also next chapter - problems with RQD measurement). It should be noted that borehole cores show only a very small part of a discontinuity surface and that consequently the determination of properties may be less accurate.

[15] If selected discontinuities happen not to be accessible, the orientation can often be measured from a distance by simple means, such as clinometer and compass or photogrammetry. However, properties of a not-accessible discontinuity (set) have to be estimated.

D.4 Block size and block form

Intact rock block size and form are dependent on the number and orientation of discontinuity sets. Block size and form are often of major importance in excavations. Generally is larger better than small and cubic is better than tabular, flaky, or columnar. Some typical values are listed in Table D-1.

D.4.1 Overall spacing of discontinuity sets in a rock mass

Various expressions have been defined to quantify in a single qualitative or quantitative expression the spacings of a number of discontinuity sets in a rock mass. One of the simplest expressions is the RQD (Deere, 1967, 1988, 1989, ch. 0), more detailed expressions, which describe block size and block form in a rock mass, can be found in BS 5930 (1981, ch. N.2), Price (1992, ch. N.2). Taylor (1980) developed eq. [D-1] for the description of the spacing for a maximum of three discontinuity sets:

$$
\begin{array}{l}
\text{For a rock mass with one discontinuity set:} \\
factor_1 = 0.45 + 0.264 * \log_{10} x \quad factor_2 = 1 \quad factor_3 = 1 \\
(x = discontinuity\ spacing) \\
\text{with two discontinuity sets:} \\
factor_1 = 0.38 + 0.259 * \log_{10} x_{minimum} \quad factor_2 = 0.28 + 0.300 * \log_{10} x_{maximum} \\
factor_3 = 1 \\
\text{with three discontinuity sets:} \\
factor_1 = 0.30 + 0.259 * \log_{10} x_{minimum} \quad factor_2 = 0.20 + 0.296 * \log_{10} x_{intermediate} \\
factor_3 = 0.10 + 0.333 * \log_{10} x_{maximum} \\
\text{spacing factor for rock mass} = factor_1 * factor_2 * factor_3 \\
\text{(minimum, intermediate, and maximum refer to the spacing of the discontinuity sets)}
\end{array}
\qquad [D\text{-}1]
$$

The graphical representation is shown in Fig. D-3. The parameter is calculated for a maximum of three discontinuity sets with the lowest spacings. The method according to Taylor is used in Laubscher's classification system for underground excavations (1990, ch. N.4.3). Many engineers, including the author, have extensively used, with success, the Laubscher system for classification of the stability of underground excavations in a mining environment.

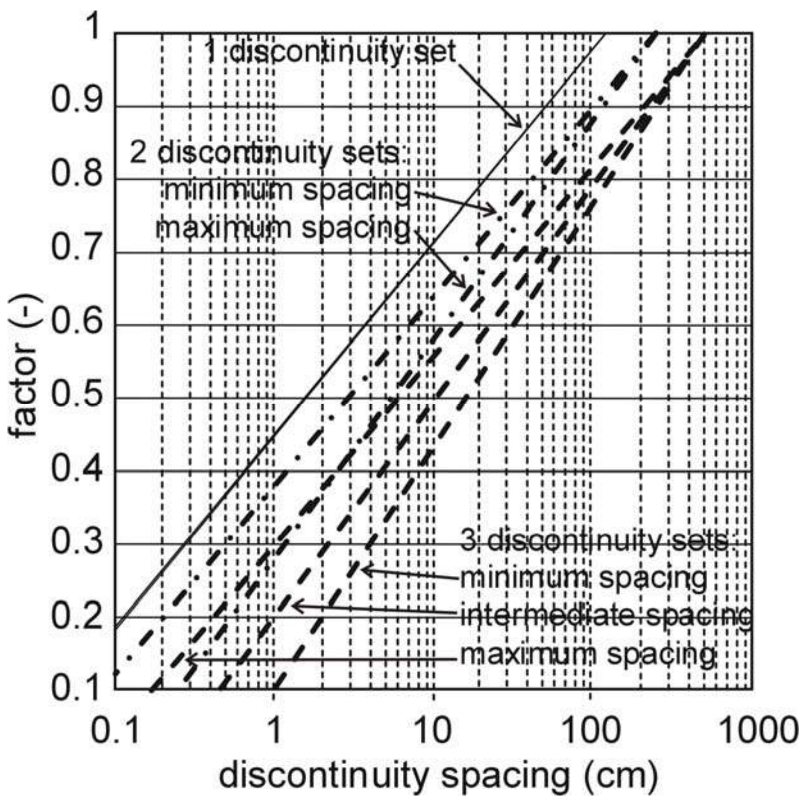


Fig. D-3. Discontinuity spacing factors (after Taylor, 1980)

D.5 Overall condition of discontinuity sets in a rock mass

Several options exist to describe the overall properties describing the shear strength of discontinuity sets in a rock mass. An average or a weighted mean of the condition of the different discontinuity sets can be used. In many classification systems, only that discontinuity set is considered that has the most adverse condition (ch. N.7.)

E EXPOSURE SPECIFIC PARAMETERS[16]

E.1 Rock mass weathering and susceptibility to weathering

Most civil engineering works occur close to the surface and the process of 'weathering' has affected most groundmasses at shallow depth. Because of this weathering of both engineering soils and rocks is one of the moist important problems with which the engineering geologist has to contend. Weathering implies decay and change in state from an original condition to a new condition as a result of external processes. A review of weathering processes has been given by various authors (Anon. 1995a, Price, 1995).

Rocks may be affected by mechanical weathering, mostly caused by expansion and contraction due to thermal changes. They are more commonly affected by chemical weathering, which, as the result mostly of the passage of water through the rocks, changes the minerals in the original rock to some other mineral. This may involve a change in volume, loss of material in solution, perhaps swelling and the development of cracking and a gradual increase in the quantity of clay minerals in the rock. Often the most obvious sign of weathering is brown staining in the rock by the oxidation of iron bearing minerals. While solution is an agency involved in most forms of chemical weathering, it is most prominent in limestones. In clastic limestones, such as chalk and calcarenites, it is displayed by the formation of near vertical solution pipes extending from the limestone/overburden surface to depths of perhaps as much as 40 m. These are usually wholly but sometimes partially infilled by overburden deposits which have flowed or been washed in from above. Crystalline limestones are often strong enough to support the development of underground cavern systems through which rivers may flow. The limestones closely adjacent to such systems may be entirely unaffected by the solution weathering and be as strong as the fresh material.

Weathering of one sort or another takes place in all environments but is most intense in hot, wet climates where, if climatic conditions have remained uniform for long periods, weathering may be expected to extend to great depths. While weathering may reach great depths, it is slow to do so and the style of weathering may change if climatic conditions change. In the northern hemisphere, where large areas have been subjected to phases of glaciation with intervening warmer periods the weathered nature of the groundmass as presently seen may reflect these changes. Thus, in the most recently glaciated areas all weathered materials may have been carved away by ice and almost fresh rock exposed. Beyond the boundaries of glaciation, weathering may reflect periglacial conditions (Higginbottom and Fookes, 1971). It is generally thought that, at the end of the last glaciation, sea level rose by about 100m. This implies that in seas less than 100m deep-sea bed materials have been exposed to sub-aerial weathering for a substantial period of time and their properties may reflect this. Indeed, if at any time in geological history, any material has been exposed above surface, it would have been subject to weathering. Thus in Western Australia, some of the laterites exposed (and perhaps covered by more recent deposits) are thought to be of Tertiary age (Geological Survey of Western Australia, 1974).

[16] Part of the text in this chapter is from chapter 3, "Mass and hydrology" by Price & Hack, from "A primer in Engineering Geology", by Price et al.

Table E-1. Variations in engineering properties of dolerite and granodiorite as a consequence of weathering

GRADE	Density (γ) (kN/m3)	Porosity (n) (%)	Unconfined compressive strength (UCS) (MPa)	Unconfined tensile strength (UTS) (MPa)	Static deformation modulus (Es) (GPa)	Seismic velocity		Schmidt hammer number (H)	Rock mass friction (deg)	Rock mass cohesion (kPa)
						Longitual wave Vp (m/s)	Shear wave Vs (m/s)			
DOLERITE †										
I – II	28.04	0.4	160 -180	42 – 48	16.5	4000 - 5000		64		
III	27.64	0.5	83 - 160	15 – 42	3.3	2500 - 4000		53		
IV	26.96	1	58 - 83	11 – 15		1800 - 2500		45		
V	26.18	3.2	24 - 58	2 – 11		1400 - 1800		25		
GRANODIORITE ‡										
I	25.6 - 26.96	2.6 - 0.4	111 - 165		31 - 34	3749 - 4968	2520 - 2883		47	17
II	25.7 - 26	3.3 - 5.9	60 - 97		14.5 - 15.3	1737 - 2377	1545 - 1840		46	16
III	25.1 - 25.7	1.5 - 2.3	33 - 48		9.4 - 11.5	1545 - 1840	1082 - 1139		38	14
IV	22.9 - 25.1	5.2 - 6.1	8 - 24		3.9 - 5.9	499 - 1447			17	8
V	19.8	24	0.1		0.002 - 0.013				6	3
VI	14.7	44								

†Dolerite data from Price own files; dolerite once exposed at Stirling Castle, Scotland. ‡Granodiorite data from Krank, K.D. and Watters, R.J. (1983), except rock mass friction and cohesion. Granodiorite rock mass friction and cohesion from slope back analysis in Granodiorite in the Falset area, Spain, from Hack (1998). Grade scales follow the classification given in Table E-2.

Fig. E-1. Decrease of bedding spacing ("bankink" spacing) nearer to the original surface due to weathering (photo Verwaal, 2001)

E.1.1 Influence of weathering on rock mass properties

Weathering weakens intact rock, discontinuities, and, hence, the rock mass as well. Table E-1 shows results of tests indicating how rock properties change with increasing weathering. The table in which the weathering is given by grades (I = fresh to VI = residual soil) shows the considerable difference in material properties that are a consequence of weathering. An ordinary geological map showing the granodiorite or the dolerite in the table would not give any indication of the weathered condition of the rock, yet clearly this has great influence on the likely engineering behavior of that rock material and mass (sse also figs ???). Because of this, it is customary to describe the weathered condition of the rock in all engineering geological descriptions of material or mass.

Fig. E-2. Detail of decrease of bedding spacing ("bankink" spacing) nearer to the original surface due to weathering (photo Verwaal, 2001)

E.1.2 Standard weathering description systems

Weathering can be described following a relatively simple scheme, such as in Table E-2, which is compiled from recommendations given in BS5930; 1981 and the report of the Engineering group Working Party on Core Logging (Anon., 1970), or following more elaborate schemes of which BS5930; 1999 (Table E-4) is an example. While the simple scheme is easy to use, it often does not fit a particular state of weathering for a particular material in a particular environment. The more elaborate schemes have far more flexibility to fit all sorts of materials and environments, but have the potential risk that the scheme due to its complexity may not unambiguously be understood.

The factors entering into the description of weathering are the condition of the discontinuities (joints, bedding planes, foliations etc.) and the condition of the material between the discontinuities. Weathering begins on the discontinuities that transmit water. Since the major discontinuities (those of greatest persistence and aperture) transmit the most water, the lowest grade signifying the onset of weathering records weathering on the surface of the major discontinuities. Thence increasing weathering affects more and more discontinuities and eventually starts to affect the rock material. Table E-2 gives grades of weathering for rock cores, rock materials, and rock masses seen in an outcrop. Some conflict appears between the core and mass descriptions in grade III for in this grade cores are 'not friable' while in the mass, for the same grade, less than half the rock is decomposed to soil. 'Friable', in the dictionary, means easily crumbled; grains may be easily de-

Table E-2. Standard terminology for description of weathering of rock cores, outcrops, and material

Weathering description	Grade number	Rock core grades ‡	Rock outcrop grades †	Rock material descriptive terms †
FRESH	I (A)	No visible sign of weathering.	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.	Rock material weathering can be described by using terms such as:
FAINTLY weathered *	1(B)	Weathering limited to the surface of major discontinuities.		
SLIGHTLY weathered	II	Weathering penetrates through most discontinuities, but only slight weathering of rock material.	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering.	Discolored: The color of the original fresh rock material is changed and is evidence of weathering.
MODERATELY weathered	III	Weathering extends throughout the rock mass but the rock material is not friable.	Less than half the rock material decomposed or disintegrated to a soil. Fresh or discolored rock is present as a continuous framework or as corestones.	Disintegrated: The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable but the mineral grains are not decomposed.
HIGHLY weathered	IV	Weathering through discontinuities and material and the rock material is partly friable.	More than half the rock material decomposed or disintegrated to a soil. Fresh or discolored rock is present as a discontinuous framework or as corestones	Decomposed: The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
COMPLETELY weathered	V	Rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.	All rock material is decomposed and/or disintegrated to soil. The original rock mass structure is still largely intact.	The stages above may be qualified by using terms such as 'partially', 'slightly', 'wholly'.
RESIDUAL SOIL	VI	A soil material with the original texture, structure, and mineralogy of the rock completely destroyed.	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume but the soil has not been significantly transported.	

† After BS 5930, 1981. ‡ After Geol. Soc. Engng Group Working Party on 'The logging of cores for engineering purposes' (Anon., 1970)
* Faintly weathered is seldom found in descriptions and may be considered more-or-less obsolete. Grade I only becomes I A if faintly weathered is used.

tached by rubbing with the fingers, which must be the case for many soils. It is, in any case, very difficult to describe weathering of a rock mass from the evidence presented in rock cores because of their limited lateral width. Usually rock mass weathering is assessed from a number of boreholes, and natural or artificial outcrops. Boundaries between weathering grades can never be more than approximate.

Table E-3. Adjustment factors for different geotechnical properties of a rock mass (after Hack et al., 1997, 1998)

WEATHERING FACTORS									
degree of rock mass weathering (BS 5930; 1981)	intact rock strength	overall spacing of discontinuities	condition of a single discontinuity (set)	overall condition of discontinuities (1)	rock mass (1)		number of observations(2)	lithostratigraphic	
					cohesion	φ		sub-units (1)(2)	units (1)
Fresh	1.00	1.00	1.00	1.00	1.00	1.00	12	7	5
Slightly	0.88	0.93	0.99	1.00	0.96	0.95	168	20	6
Moderately	0.70	0.89	0.98	0.99	0.91	0.90	27	12	6
Highly	0.35	0.63	0.89	0.89	0.64	0.59	6	3	3
Completely(3)	0.02	0.55	0.77	0.80	0.38	0.31	2	1	1
						Total:	215	24	7

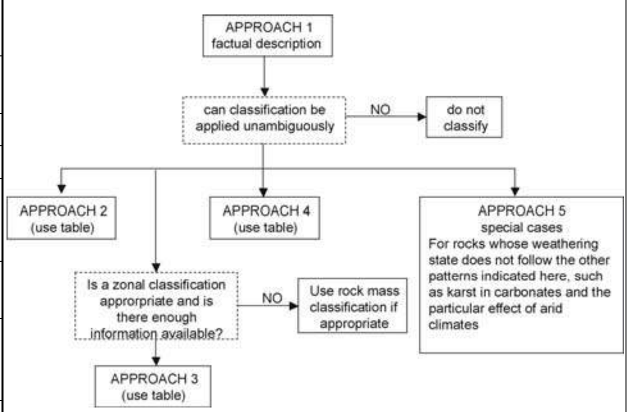
notes:1) Columns 'sub-units' and 'units' are respectively the number of lithostratigraphic sub-units and the number of lithostratigraphic units used.
2) Used for the calculation are only sub-units in which at least two different degrees of weathering have been observed so that weathering effects could be compared in the same 'lithostratigraphic sub-unit'.
3) 'completely weathered' is assessed in granodiorite only.

Table E-4. Description state of weathering (after BS5930; 1999)

APPROACH 2 rock is moderately strong or stronger in fresh state UNIFORM MATERIALS		
grade	classifier	description
I	fresh	Unchanged from original state
II	slightly weathered	Slight discoloration; slight weakening
III	moderately weathered	Considerable weakened, penetrative discoloration; large pieces cannot be broken by hand
IV	highly weathered	Large pieces can be broken by hand; does not readily disintegrate (slake) when dry sample immersed in water
V	completely weathered	Considerably weakened; slakes in water; original texture apparent
VI	residual soil	Soil derived by in-situ weathering but having lost retaining original texture and fabric

APPROACH 3 heterogeneous masses (mixture of relatively strong and weak material) HETEROGENEOUS MASSES		
zone	description(2)	typical characteristics
1	100 % grades I - III	Behaves as rock; apply rock mechanics principles to mass assessment and design
2	> 90 % grades III < 10 % grades IV - VI	Weak materials along discontinuities; shear strength stiffness and permeability affected
3	50 to 90% grades I -III 10 to 50% grades IV - VI	Rock framework still locked and controls strength and stiffness; matrix controls permeability
4	30 to 50% grades I - III 50 to 70% grades IV - VI	Rock framework contributes to strength; matrix or weathering product control stiffness and permeability
5	< 30% grades I - III 70 - 100% grades IV - VI	Weak grades will control behavior. Corestones may be significant for investi-gation and construction
6	100% grades IV - VI	May behave as soil although relict fabric may still be significant

APPROACH 4 (moderately weak or weaker in fresh state) CLASSIFICATION INCORPORATES MATERIAL AND MASS FEATURES		
class	classifier	description
A	unweathered	Original strength, color, fracture spacing
B	partially weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation
C	distinctly weathered	Further weakened, much closer fracture spacing, gray reduction
D	de-structured	Greatly weakened, mottled, lithorelicts in matrix becoming weakened and disordered, bedding disturbed
E	residual or reworked	Matrix with occasional altered random or apparent lithorelicts, bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation



Quantification of degrees of weathering in terms of the reduction of geotechnical properties of a rock mass have been done by Hack et al. (1997, 1998) and are shown in Table E-3. The values should be considered, as a first attempt as for the higher degrees of rock mass weathering the number of observations and different units is small.

E.1.3 Weathering description and zonation

Weathering is, like so many other geological phenomena, a gradational feature and to deal with such features the usual approach is to impose boundary conditions within them so that they are divided into various grades defined by a range of characteristics. In reviewing, the applicability of the grades of weathering proposed in the various systems mentioned above it is well to remember the purpose of describing weathering which, in this case, is to assess its significance with regard to engineering projects to be conducted in or upon the weathered rock mass. This significance depends upon two factors, first the change in engineering properties of the rock material and second, the volumetric regularity or irregularity of this reduction within the rock mass. Regarding the second, the writer distinguishes four basic types of mass weathering. These are:

1. Uniform weathering. A gradual decrease of weathering grade and intensity with depth in thick strata of homogeneous lithology.
2. Complex weathering. An irregular weathering profile in layered lithologies that have different susceptibilities to weathering. It may mean that more weathered strata lie under less weathered strata, particularly if the strata dip from surface outcrop.
3. Corestone weathering. In many, mostly coarser grained, igneous rocks rounded 'corestones' of almost fresh rock may be surrounded by very decomposed highly friable material, similar to a compact sand. The corestones become larger with depth.
4. Solution weathering Carbonate and most salt rocks weather by solution. Joints and bedding planes become open and underground caverns may develop. In strong crystalline limestones, karstic conditions may result. In the weaker calcarenites and calcilutites or calcisiltites (chalk), solution pipes, often infilled with materials from above, may have penetrated deeply into the strata.

Each style of mass weathering is associated with an engineering response. In foundation engineering, for example, uniform weathering gives the least difficulties, requiring, no more than deepening a foundation excavation to reach strata which, although weathered, have adequate bearing capacity. Complex weathering, involving diverse strata with different susceptibilities to weathering, could mean that any foundation excavation could include wide variations in weathered condition within the volume of ground stressed. Corestone weathering indicates extreme variations in foundation quality, with strong nodes set in a much weaker matrix. Solution weathering in crystalline limestones implies the potential presence of cavities under possible foundation elements.

While a complete volume of rock mass might be described using the terms given above within that mass the weathered nature must be described in greater detail in order to more closely target an engineering problem. This is perhaps best done with regard to the engineering significance of the mass weathering observed.

The following terms are proposed:

i) *Effectively unweathered*. The weathering of the rock mass is such that any engineering work may be constructed on or in it without regard to the weathered condition as found.

ii) *Significantly weathered*. The weathering of the rock mass is such that some regard must be taken of the weathered condition of the rock mass in the design and construction of some particular engineering works. For example, weathering of discontinuities could imply an impairment of shear strength if the work involves the construction of rock slopes, but such weathering would have less effect on the design of foundations on the rock mass.

iii) *Severely weathered*. The weathering of the rock mass is such that the weathered condition of the mass dominates the design and construction of any engineering work to be constructed on or in it. This implies both weathering of discontinuities and materials.

iv) *Residual soil*. Sufficient of the rock material is decayed to the geotechnical condition of a soil to make the mass behave as a mass of soil. It may be important, because of their influence on, for example, slope stability, to distinguish between those residual soils that are structure less and those that have relict discontinuities.

The boundary between iii) and iv) is difficult to establish. Engineering problems in i), ii), and iii) are resolved by the application of rock mechanics and in iv) by the application of soil mechanics. A rock mass in condition iv) may contain relict rock blocks which, if in continuous contact, may present the engineering behavior of a rather weak rock mass. If the relict blocks are not in contact, the total mass may behave as an engineering soil. If the volume of soil material exceeds about 50% of the mass then the behavior of the total mass should be that of a soil and be categorized as residual soil iv); if the volume of soil is less than about 30% then behavior should be that of a severely weathered rock mass iii). It will be difficult to estimate such percentages in this shadow zone between soil and rock mechanics and estimates are most likely to be subjective.

The terms given above could be used to describe zones of weathering within a rock mass. Such zones might be observed in outcrop and boundaries evaluated. The author has suggested a ratings system to aid establishing such boundaries (Price, 1993). It also provides a numerical estimate of weathering that may form part of a rock mass classification system and has been so used by Laughton and Nelson (1996) in considering the rock mass parameters that are necessary to predict tunnel boring machine performance.

The data required to zone a rock mass into categories i) to iv) may come from natural or artificial surface outcrops but is derived most commonly from boreholes. Because the categories of weathering given above relate to the mass they cannot be established directly or given in borehole records but the core descriptions in each borehole should contain sufficient information to allow mass weathering zonation to be established from an assemblage of borehole and outcrop data. Thus, in core descriptions note must be made of the weathered condition of discontinuities and the extent of decay of rock materials. The first three grades of the zonation scheme suggested clearly relate to the difficulty of engineering within the zones and would be best assigned with knowledge of the engineering to be undertaken.

In discontinuous rock mechanics the condition of discontinuities is of major importance and it would seem logical to describe discontinuity weathering to provide a link to assessments of discontinuity strength. In both integral and mechanical discontinuities, three main conditions may be considered to exist. These are:

- *fresh*: no sign of weathering
- *surface stained*: no or little penetration of weathering into the wall rock (so that the discontinuity asperities have the strength of fresh rock)
- *surface weathered*: weathering penetrates to a depth greater than the roughness or undulations of the discontinuity surface (so that the discontinuity asperities have a strength less than that of fresh rock)

In the case of rock materials similar simple terms may be applied, namely:

- *fresh*: no sign of weathering
- *stained*: the material is weathered but without obvious loss in strength
- *decayed*: deeply stained and with obvious loss of strength, perhaps friable

In the case of solution weathering an additional term, *absent by solution*, might be applied.

Achieving a weathering zonation of a rock mass investigated largely by boreholes is greatly aided if the style of weathering in outcrops of similar rock can be studied, even if these are not located close to the site. This may help the investigator appreciate the relationship of the linear data given by boreholes to the volumetric reality of the rock mass.

E.1.4 Susceptibility to weathering

The lifetimes of most structures for civil engineering and infrastructure are in the order of 50 to 100 years. To guarantee the safe and sound design for the whole lifetime it is, hence, important to know what the geotechnical properties of the soil or rock mass are going to be at the end of this time span, e.g. what is the susceptibility to weathering of the soil or rock mass? If, for example, a slope is excavated in a sandstone in which the cement between the grains consist of gypsum it can be expected that in a moderate climate the gypsum will dissolve and the sandstone rock mass changes into a soil mass of loose sand grains within a couple of years. Alternatively, a slope made in fresh granite is not expected to undergo any major changes within 100 years in a moderate climate. Quantification of the influence of weathering on soil and rock mass properties and the rate of weathering are only limitedly known. The influence of weathering on intact rock used as building or gravestones has been studied and rates for weathering have been established (Fookes et al., 1988, Selby, 1982). For most soils and rocks, it is also reasonably well known how they deteriorate over long (geological) periods. However, for 50 to 100 year time spans very little is known. For the extremes given in the slope example above, it is generally not difficult to make an estimate of the changes of properties due to weathering. For many other soil or rock masses, for which it is not so clear, the engineer has as only option to estimate the susceptibility to weathering based on other exposures of known excavation date. If these do not exist, an engineering guess has to be made.

There is no readily available test for determining the susceptibility to weathering for rock masses over long (up to 50 years) periods. This is obvious if one considers the amount of different materials, the amount of different processes, the variation in local circumstances, the necessary size of samples, and the time span over which tests have to be executed[17]. Mostly these tests are done on relatively small samples not representative of a rock mass. Tests for establishing susceptibility to weathering of discontinuities in a rock mass are not available for the same reasons.

Laubscher's weathering factors

Research has been done to the weathering rates in underground excavations and its influence on rock mass classification ratings (Laubscher, 1990) (see chapter N.4.3) and shown in Table E-5. The percentages given in Table E-5 are multiplied with the rock mass rating calculated following Laubscher, e.g. the rock mass rating is reduced by about 50 % if a rock mass is expected to weather from fresh to completely weathered within a half year. It should be noted that conditions in underground excavations are considerably different and, in general, with less variation than the conditions which influence weathering at surface.

expected future degree of weathering	percentage adjustment for a rock mass weathered from fresh				
	after ½ year	after 1 year	after 2 years	after 3 years	after ≥ 4 years
fresh	100	100	100	100	100
slightly	88	90	92	94	96
moderately	82	84	86	88	90
highly	70	72	74	76	78
completely	54	56	58	60	62
residual soil	30	32	34	36	38
Note: The adjustment is applied to the rating for the stability of the underground excavation of Laubscher's rock mass classification to predict the future stability. The degrees of rock mass weathering follow BS 5930, 1981.					

Table E-5. Adjustment values for susceptibility to weathering for classification of stability of underground excavations in mining (after Laubscher, 1990)

E.2 Method of excavation

The method of excavation has an influence on the condition of the rock mass and is regarded as an exposure specific property. The rock mass characteristics in the exposure are influenced by the method of excavation. The influence of the method of excavation is discussed in more detail in ch. I.

[17] A possible means to establish susceptibility to weathering is to determine the slake durability (ISRM, 1981a). This test is, however, only a crude simulation of some of the processes involved in weathering.

F EXTERNAL INFLUENCES

F.1 Surface run-off water

Water run-off over a slope and through the near surface of a slope can lead to instability. For example, surface run-off water will have a larger influence on a slope in a rock mass with a small block size than with a larger block size because smaller blocks are more easily flushed away by the water.

F.2 Snow and ice

Snow and ice may block seepage from the discontinuities where discontinuities are outcropping at the slope face or in a tunnel (portal). This may lead to water pressures in the discontinuities. Additionally snow and ice add weight to a slope. In situ frozen water will expand because ice has a larger volume than water and thus widening discontinuities. In underground excavations, snow and ice are fairly seldom, however, near portals ice may occur.

F.3 Rock mass creep and stress relief

Rock mass creep and stress relief can lead to new cracks in intact rock, develop integral discontinuities into mechanical discontinuities, and open existing discontinuities. These effects are normally included in weathering. Creep movements and stress relief can also cause displacements along discontinuities, resulting in non-fitting discontinuity planes. Large movements of the rock mass in a slope may cause an increase in the slope dip angle leading to slope instability.

F.4 Vegetation

The engineering lifetime, for example 50 years, of a slope is more than sufficient to allow some types of trees to develop to full growth. Root wedging will dislodge blocks, allow water infiltration, etc. The prevention of such growth falls within the province of slope maintenance.

In underground excavations, vegetation is normally not present. Actions by other living species such as fungus, bacteria, etc. may have an influence on the lifetime of, for example, timber support and by producing poisonous gasses or by using oxygen creating a shortage of oxygen.

F.5 External stresses

External stresses working on the rock mass in or on which an engineering structure is or will be excavated can have a large influence on the stability. External stresses do not originate in the rock mass directly surrounding the engineering structure, but are, for example, stresses due to a high hill or mountain behind a slope or stresses due to other existing underground excavations. Generally, it is impossible to determine external stresses without stress measurements. A differentiation should be made between the ‘virgin stress field’ and stress fields induced by man-made action.

F.5.1 In-situ stress field

F.5.1.1 Overburden stress

The in-situ or virgin stresses are the stresses in the subsurface before any extraction of material. Stresses in the subsurface are caused by the weight of the overlying material (the overburden). The vertical stress (S_v) at depth H is:

$$S_v(H) = \int_{h=0}^{H} UW(h) \; dh \qquad \text{[F-1]}$$

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

in which *UW(h)* are the unit weights of the groundmasses above depth *H*. Deformation of the mass under the vertical stress causes horizontal stresses. If the material would be water, the vertical stress equals the horizontal stresses (hydraulic stress configuration) because water cannot sustain shear stresses. Other masses do sustain shear stresses and therefore the material will not completely transform the vertical stress into equal horizontal stresses, but only a certain fraction that depends on the material and mass deformation characteristics.

F.5.1.2 K_0 factor

The fraction is depicted by K0 the ratio between horizontal and vertical stresses without any geological, tectonic, or man-made influences for an area with a flat topography. In an isotropic, homogene, linear-elastic material the value depends on the Poisson's ratio (υ) of the mass and:

$$K_0 = \frac{\upsilon}{1-\upsilon} \qquad \text{[F-2]}$$

For many rock masses, υ is about 0.25 and, thus K_0 is about 1/3 and the horizontal stresses are both about equal to 1/3 of the vertical stress. K_0 is higher (normally between 0.4 to 0.5) for loose, non-cemented sand and K_0 becomes 1 on large depth where the masses are under high stresses and high temperatures.

Soil and rock masses deform with time such that the shear stresses in the mass are minimized. The rate of time dependent deformation is governed by factors such as the material, the amount of discontinuities, the temperature, and the magnitude of confining stresses. Some masses may deform in geological time spans in the order of millions of years while other deform within a couple of days or years. This time delay in adjusting to an imposed stress field is the reason for many in-situ stress conditions that are representative for the geological past rather than the present situation. Notably, the overburden stress in the past has been higher than at present because since then the surface has been eroded. The horizontal stresses, however, are still for a part reflecting the high overburden stress belonging to the thicker overburden layer. A similar effect is known from areas that have been glaciated during the ice ages. The weight of the ice is still reflected in horizontal stresses that are higher than what these should be based on the present overburden weight. Note this only applies to the horizontal stresses as the vertical stress reduces immediately after the erosion of surface material or withdrawal of the glaciers. Topography and tectonic stresses are another reason that the stress configuration in orientation and magnitude may be completely different from what could be expected based on overburden thickness. In particular, near to high mountain ranges or near to tectonic active areas, such as continental margins, faults, etc., S_h values have been reported in the literature up to 10 times the vertical stress.

F.5.1.3 *K* factor

The parameter K is used for the ratio of average S_h over S_v, without concern to what it causes:

$$K = \frac{S_h}{S_v} \qquad \text{[F-3]}$$

Average K values as function of depth for various areas in the world are shown in Fig. F-1. In this figure, the average values of Sh are used, but it should be noted that considerable differences might also exist between the two horizontal stresses, particularly, near tectonic active areas. To give a starting point if no other information is available, some rules of the thumb for the in-situ stress configuration are given in Table F-1, but these should be used with extreme care as in many locations the stress field may be completely different as listed in Table F-1. For a new project, it is advisable to investigate whether any other underground workings (mines, tunnels, etc.) have been made nearby. If lucky, the miners, consultants, or contractors may provide measured data on the stress field or may give personal accounts of problems during construction due to the virgin stress field. It may also be worthwhile to go underground in other projects and

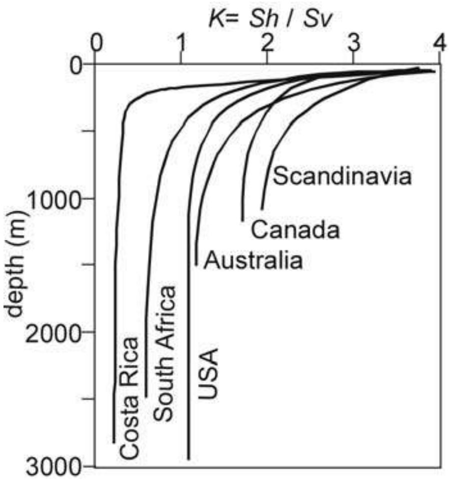


Fig. F-1. Measured virgin stress field (modified after Lopez, 1997; data from Bieniawski, 1984, except Costa Rica, which is from Lopez, 1997) see whether any signs are visible that indicate anisotropic stress fields, such as heavier support, or more fracturing depending on tunnel direction that cannot be explained by differences in the groundmass, etc.

Table F-1. Virgin stress field (unreliable rules of the thumb)

	depth (m)	
vertical stress	< 50	stress due to weight overburden
	50 - 2000	overburden weight, but large scatter has been measured between 0.5 x and 2 x overburden weight
	> 2000	overburden weight
horizontal stress	< 50	K = 0.4 – 0.6 for cohesive material, e.g. clay, 0 - 0.4 for discontinuous rock masses, and 0.25 - 0.45 for loose, non-cemented material
	50 - 500	K = 0.4 – 3.5
	500 - 1500	K = 0.5 - 1.5
	1500 - 2500	K = 0.7 – 1
	> 2500	K = 1
Note: the above are the values as used by the author if no other information is available; the values should be handled with great care (K = *Sh* / *Sv*); *Sh* are the average horizontal stresses.		

F.5.2 Man-made induced stresses

In many situations, the site for a proposed excavation is to be made in a stress field disturbed by other excavations. Consider Fig. F-2, which shows a synclinal ore body that has been mined from surface downwards. The 'crown pillars' made to allow mining of the stopes (mining expression for a 'mining room') without collapse of the hangingwall, were left in place after the stope was mined. These were expected to fall apart or to break under the increased stress concentrations and the hangingwall was supposed to collapse unto the footwall. Then the weight of the central core of the syncline would have been spread over the whole footwall. However, the crown pillars did not collapse, but kept transferring the weight from the central rock mass to the footwall. This resulted in a strongly anisotropic stress field at the locations of the crown pillars and created problems with the stability of the haulages (mining expression for tunnels), and the stresses on the bottom part of the syncline prohibited mining. A solution of the problem was found in blasting the crown pillars causing the collapse of the hangingwall and thus the spreading of the weight of the core over the whole footwall. This example is fairly extreme, but a modified stress field can also be caused by an already existing tunnel or by surface structures in case of a shallow underground excavation. Notably foundations of high-rise buildings may influence the magnitude and orientation of the stress field underground.

F.5.3 Stress measurements

Stress measurements are the only reliable way to investigate a stress field. Stress measurements are, however, very sensitive to local variations in the groundmass, for example, the presence of a discontinuity. This means that many stress measurements at a single location have to be done before the obtained values can be assumed reliable. In-situ stress measurements are expensive. Most standard methods for measuring the stress are based on the same principle: a volume of the rock mass is allowed to expand due to unloading. Reloading until the original volume is obtained gives the original stress condition. Examples of stress meas-

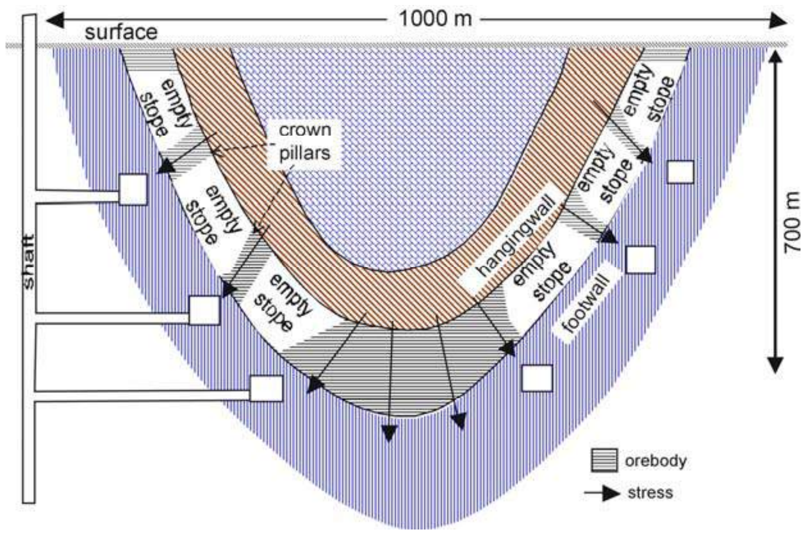


Fig. F-2. Man-made induced stresses due to poor mine planning (a number of stopes has been left out for clarity of the figure) urements are: overcoring, flat jack tests, dilatometer tests, and induced fracturing.

Overcoring is done in a borehole by first drilling a small diameter borehole. In the hole, a device is fixed which will measure strain differences. Then the same hole is drilled with a large diameter bit and the piece of rock where the strain measurement device is mounted is overcored. Due to the overcoring, the stress on the piece of rock with measurement device is released and the piece of rock with strain measurement device will expand. On the same piece of rock or on any other piece of core, stress-strain tests are done to establish the deformation modulus. The modulus divided by the strain measured during expansion give the stress. The strains are normally measured in three directions; hence, the method results in establishing the principal stress field in three directions. Because the test is done in a borehole, it can be done far away from any human influences, such as excavations. The test is only suitable for materials that have a tensile strength, e.g. cemented soils or rock.

The ***flat-jack*** test involves making a half-circular slot in the site of an underground excavation either by drilling boreholes or by a large diameter saw. Because apiece of the rock mass is removed the surrounding rock is allowed to de-stress and expand. In this slot, a flat jack is placed and expanded. The pressure required opening the slot until its original dimensions give the original stress perpendicular to the slot. The test is done from an excavation, hence only the stress influenced by human actions is measured. Similar to overcoring the test can only be done in a material with a tensile strength, e.g. cemented soil or rock.

For the ***dilatometer*** test a borehole is drilled. Due to the removal of material, the borehole will reduce in size. In the borehole a dilatometer test is done. This is in its simplest form the expansion of a balloon. The stress in and the volume of the balloon are measured and hence the deformation modulus of the groundmass can be determined. The stress in the balloon to obtain the original diameter of the borehole is the original stress perpendicular to the borehole. The test is done in a borehole and can be done away from human influences. The test can be done in soil or rock, however, in soil the obviously not-cased borehole should stand open long enough to bring in the test equipment.

Hydraulic fracturing works by inserting a fluid under pressure via a borehole into a rock mass. Due to the high pressure the rock mass material will break and a fracture is created. The stress required to maintain the open fracture without further expansion of the fracture is a measure for the in-situ stress.

G STRESS AROUND EXCAVATIONS

G.1 Introduction

If an underground excavation is made the stress configuration around the opening will change. A re-distribution of stresses will occur because deformation of the rock is possible due to the opening. In general, rock masses around an underground excavation support itself by arching if stresses on all sides of the opening are present. This has implications for the way support in an excavation is installed.

G.2 Stress around an underground excavation

In general, the groundmass around an underground excavation supports itself by arching. Arching effects exist because the groundmass will deform in the direction of the opening. However, at any distance from the excavation the material occupies a larger volume than the volume available when it moves into the direction of the opening. The reduction in volume will cause that the material will deform and that the stress on the material in the direction of the reduction in volume increases.

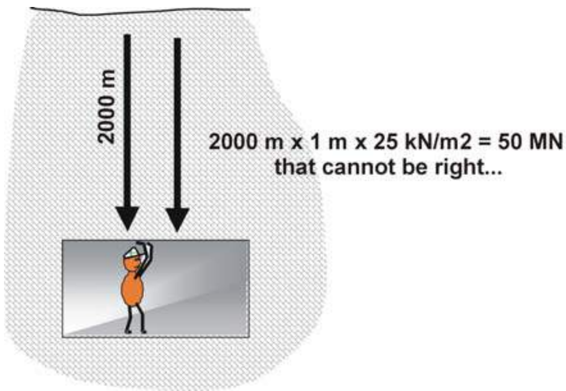


Fig. G-1. The roof does not support the overburden stress

That arching exists can easily be shown by a simple calculation. Assume that a square tunnel of 5 m wide and 3 m high has to be made at a depth of 2000 m and the mass unit weight is 25 kN/m^3 (Fig. G-1). The vertical (overburden stress) is then 2000 m x 25 kN/m^3 = 50 MPa. The force on the roof per meter length of tunnel is: 5 m x 1 m x 50 MPa = 250 MN. Hence, the support per meter length has to take up 250 MN. A concrete beam system is used for support with a compressive strength in the order of 20 MPa. The surface of concrete required is thus 250 MN / 20 MPa = 12.5 m^2. However, per meter length of tunnel there is only 5 m^2 available. Hence, support is just simply impossible if the overburden pressure had to be taken up by the support. Still there are plenty of tunnels in mining and civil engineering at a depth of 2000 m or more, often without any support at all.

G.2.1 Stress around an opening in an ideal elastic, homogene and isotropic medium

The stresses around a circular opening in an ideal elastic, homogene, and isotropic medium will re-distribute following:

$$\sigma_r = \left(\frac{S_h+S_v}{2}\right)*\left(1-\frac{r^2}{d^2}\right)+\left(\frac{S_h-S_v}{2}\right)*\left(1-\frac{4r^2}{d^2}+\frac{3r^4}{d^4}\right)\cos 2\theta$$

$$\sigma_\theta = \left(\frac{S_h+S_v}{2}\right)*\left(1+\frac{r^2}{d^2}\right)-\left(\frac{S_h-S_v}{2}\right)*\left(1+\frac{3r^4}{d^4}\right)\cos 2\theta \quad \tau_\theta = \left(\frac{S_v-S_h}{2}\right)*\left(1+\frac{2r^2}{d^2}-\frac{3r^4}{d^4}\right)\sin 2\theta \qquad \text{[G-1]}$$

S_h = horizontal stress before excavation S_v = vertical stress before excavation

σ_r = radial stress σ_θ = tangential stress τ_θ = shear stress

r = excavation radius d = distance from excavation centre

$K = S_h / S_v$ θ = polar coordinate; horizontal = 0°

Examples of the stresses around a circular excavation as function from the distance perpendicular to the tunnel wall are presented in Fig. G-2. The in-situ stress field is a vertical stress σ_v, and perpendicular to it a horizontal stress σ_h. Along the excavation wall the stress field changes orientation and the major principal stress becomes the tangential stress (σ_θ) parallel to the excavation wall. Perpendicular to this is the minor principal stress, the radial stress (σ_r). If the in-situ vertical stress (σ_v) equals the in-situ horizontal stress (σ_h), the tangential stress (σ_θ) has the value of 2 x σ_v at the tunnel wall (d/r = 1). The radial stress (σ_r) equals 0 at the tunnel wall. The radial stress at the wall of a tunnel that is not supported has always to be 0, as otherwise the tunnel wall would deform. If the in-situ horizontal stress is lower than the in-situ vertical stress, the tangential stress in the roof may become negative and hence tensile (see stresses for θ = 90°).

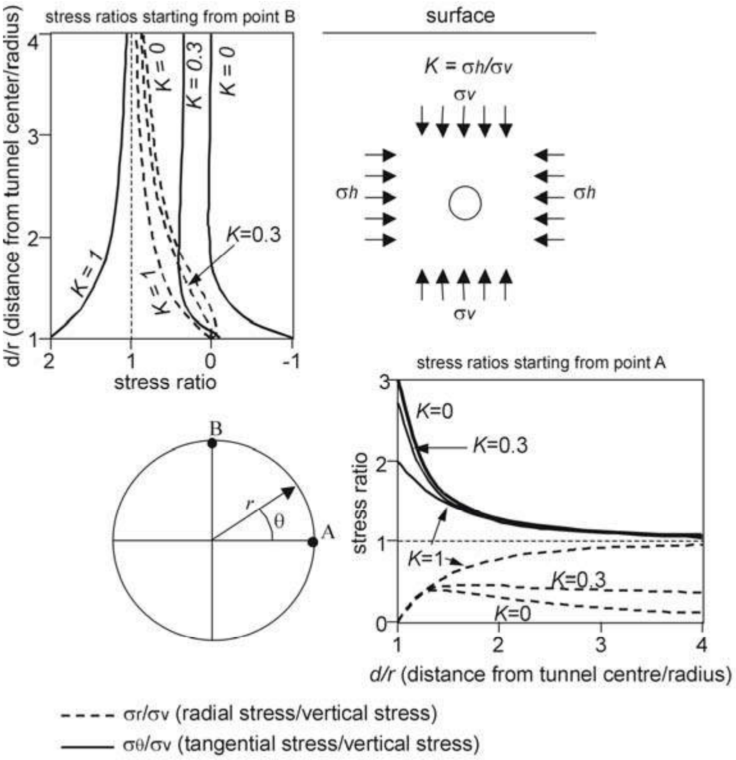


Fig. G-2. Stresses around a circular opening in a homogene, ideal-elastic, and isotropic mass

Fig. G-2 also shows that the magnitude of the stress concentrations is independent from the size of the excavation and that the influence of the presence of an opening is limited to about 4 times the radius. For non-circular openings, the stress concentrations along the wall of an opening increase (Fig. G-3). Generally, the stress concentrations along the wall of an excavation increase rapidly with a more angular excavation.

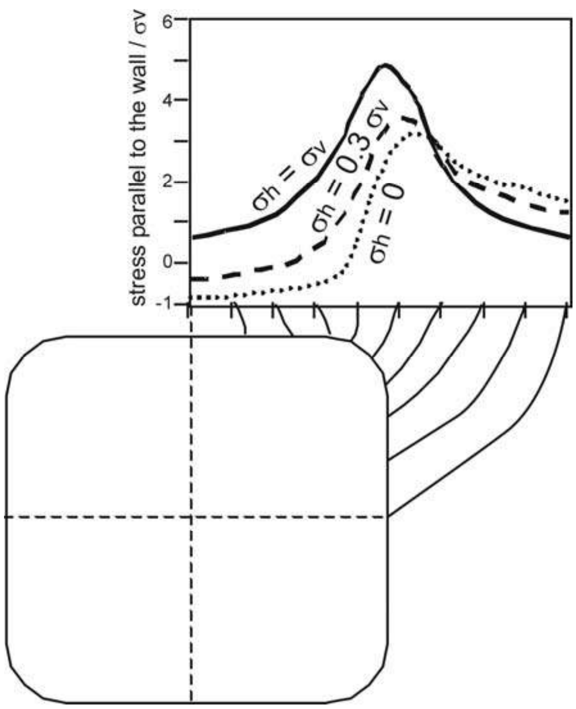


Fig. G-3. Stress concentration along the wall of a rectangular opening

G.2.2 Stress around an opening in an elasto-plastic, homogene and isotropic medium

The chapters above deal with an opening in an ideal-elastic, homogene, and isotropic mass. Real intact rock material behaves only partially as ideal elastic medium whereas soil and rock masses are not ideal elastic at all. Fig. G-4a and c show the stress-strain behavior of groundmasses after reaching the elastic limit in a laboratory test. The same behavior may occur around an underground opening and somewhere at or behind the excavation wall a similar stress-strain condition can exist. Fig. G-4b and d show the corresponding stress concentrations around underground openings. If the material deforms plastically the material will be squeezed in the opening, if the elastic or elasto-plastic limits are reached at the wall of the excavation the material will break (spalling).

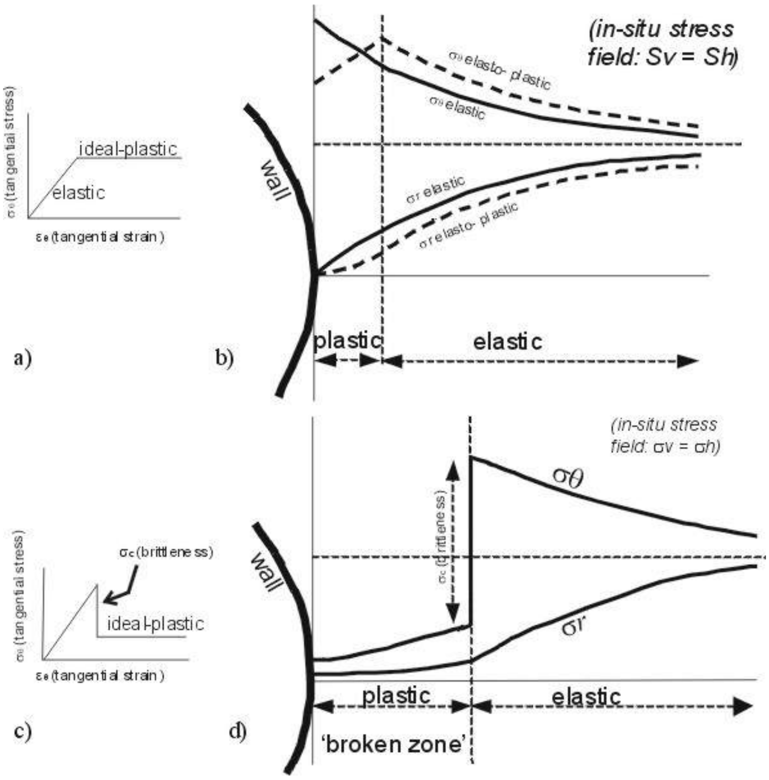
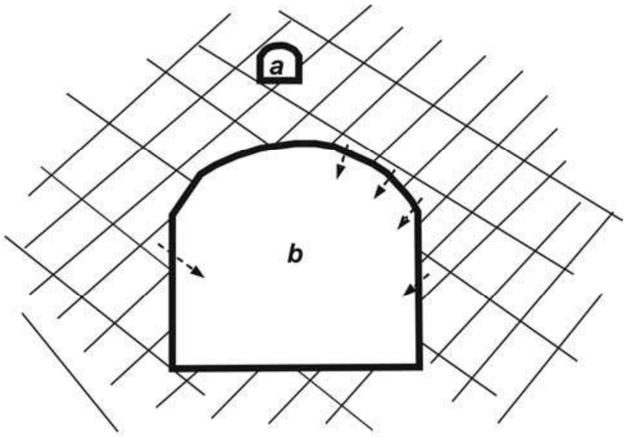


Fig. G-4. Radial and tangential stresses around a circular opening in various media. a) and c) show stress-strain curves for elasto-plastic and elasto-plastic with zero-brittleness. c) and d) show the corresponding stresses around a circular opening (free after Kastner, 1949)

G.2.3 Stress around excavations in discontinuous rock masses

In a discontinuous soil or rock mass the deformation and strength failure behavior become much more complicated, however, the results are fairly similar to those in a continuum mass. In a discontinuous mass the soil or rock mass does not need to fail anymore to create discontinuity planes because these are already present. Movements will take place fully or in part along the discontinuities. The mass adjacent to the excavation can fail because of de-stressing and subsequent gravity failure and/or because the stresses exceed the strength of the intact material (spalling). Squeezing or flowing of the material will occur either if the material is soft and cohesive (clay) or if the block size of the material is small compared to the opening (for example, shale or non-cemented sand). Because a part of the failure modes is related to the existing discontinuities, the stress-strain modes as described above for continuum materials are not applicable. Virtually every mass has discontinuities and the more discontinuities are intersected by the excavation the more chances on movement or falling of blocks is possible. Hence, in a discontinuous mass the size of the excavation becomes important (Fig. G-5).



In ***a*** few discontinuities are intersected and no options for movement along existing discontinuities exist; in ***b*** many discontinuities are intersected and consequently many options for movement, and rock fall or squeezing of blocks.

Fig. G-5. Size matters!

G.3 Dynamic stresses - earthquakes

Dynamic stresses may be superimposed on the virgin stress field by earthquakes. It is generally recognized that underground structures are less sensitive to earthquake motions than surface structures. One of the reasons is that the resonance frequency of most underground structures is well above the range of frequencies common in earthquake waves. During passing of an earthquake wave the underground excavation deforms in a combination of ovalizing (racking), axial compression and extension, and horizontal and vertical curvature. Suggested is that the amplification of the stresses at the opening do not exceed 10 to 15 % of the initial stresses (Anon, 1997). Tension cracks may develop and normal stress on discontinuities may be reduced during tension faces of an earthquake wave, resulting in movements in the surrounding mass, which can lead to loss of integrity of the mass and consequent collapse. Therefore, support systems such as bolts or wire mesh reinforced shotcrete (for support types see below) should give reinforcement to a discontinuous groundmass and support such as concrete segments should be connected together by a bolt and nut system so that the integrity of the support is not lost during an earthquake. The detailed behavior of an underground excavation in a discontinuous, inhomogeneous, and non-elastic mass under the influence of dynamic stresses is very difficult to forecast and can only be approximated by numerical simulations.

G.4 Stresses around portals

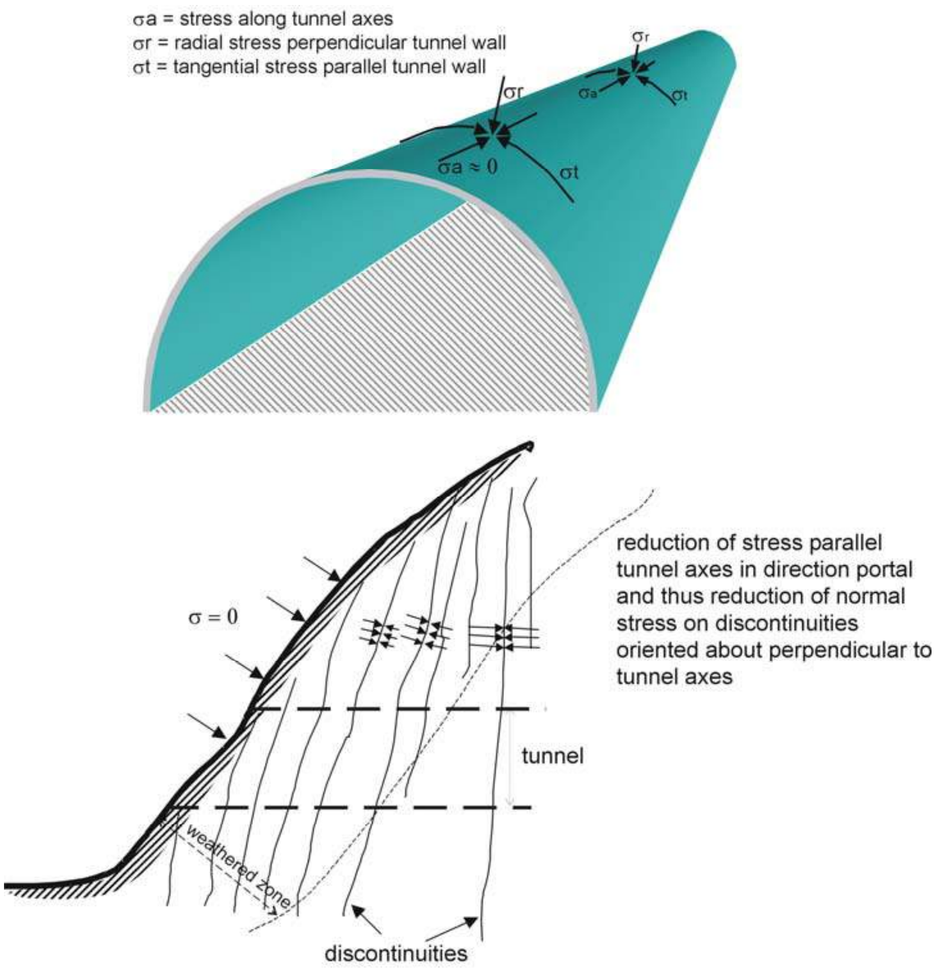


Fig. G-6. Stress parallel to tunnel axes becomes 0 at portal

Along a tunnel, the major principal stress is normally the tangential stress along the wall and the minor principal stress is the radial stress perpendicular to the wall. The intermediate principal stress is normally parallel to the axes of the tunnel. The virgin stress field determines the value of the intermediate principal stress. The value is normally not very interesting for the failure of intact rock because the influence of the intermediate stress on failure of intact rock is mostly absent or minimal. However, the intermediate stress is very important for the normal stress on discontinuities oriented about perpendicular to the tunnel axes (Fig. G-6). Near a portal of a tunnel the stress parallel to the tunnel axes will be nil. Hence, near the portals there is no normal stress on discontinuity planes oriented about perpendicular to the tunnel axes. In addition near portals are the quality of the ground mass and the strength of discontinuities mostly lower due to weathering. The combination of the two effects; less or no normal stress on discontinuity planes and weaker groundmasses often cause major problems with excavating the tunnel portals. Many tunnels in the world have been a financial disaster just due to portal problems.

H FAILURE AND STAND-UP TIME OF MASSES

H.1 Introduction

A rock mass can fail if, for example the stress levels are too high for the intact material or for the mass. Other types of problems that may make a surface or underground excavation unsuitable for its supposed application may be a gradual process such as squeezing or swelling of the material surrounding the excavation.

H.2 Swelling materials

Some materials will start swelling if exposed to the atmosphere and/or water. The swelling material will deform into the excavation and change shape and size of the excavation. Swelling stresses can be extremely high and it is often impossible to counteract the stresses. Therefore, the material should be sealed (by for example gunite or shotcrete, see below) from the atmosphere and/or water immediately after excavation to prevent or reduce the swelling. Notable swelling conditions can be expected in materials containing swelling clay minerals, in particular, montmorillonite, such as shales, claystones, mudstones, altered pyroclastic deposits, and fault gouge. Anhydrite bearing rocks will swell because anhydrite exposed to water is slowly converted to gypsum, which has a larger volume.

H.3 Failure modes of an excavation and why is support needed

As the forgoing chapters show an underground excavation can fail in different ways:

1. Spalling; the stresses at the perimeter of the excavation are such that the intact material breaks (rock or firm cohesive or cemented soil) (spalling of intact material can be expected if $\sigma\theta$ is more than about half of the unconfined compressive strength of the intact material),
2. Gravity failure; material falls into the opening due to gravity, this may be blocks of cohesive material (e.g. clay), rock blocks, or non-cohesive and non-cemented material (e.g. sand),
3. Squeezing failure; the surrounding material deforms so far into the opening that the size or shape of the opening becomes unacceptable (clay, shale, or masses with many discontinuities such as schist),
4. Flowing failure; non-cohesive and non-cemented material may flow into the excavation under the surrounding pressure (sand, residual soil, etc.),
5. Swelling failure; some materials will start swelling if exposed.

Combinations of failure modes are normal, for example, first one unit is squeezed in the opening. This allows de-stressing of other units and discontinuities, which allows gravity failure. A reduction of the strength of the mass due to the method of excavation (backbreak) or weathering may facilitate any of the failure modes 1 through 4.

Support is installed in an underground excavation to keep the groundmass directly adjacent to the excavation walls, roof, and floor in place, or to prevent failing of the groundmass and subsequent falling of material from the excavation walls and roof. Support, except for shallow underground excavations where horizontal stresses are low, does never counteract the stresses that are present in the virgin sub-surface because arching effects will deflect most of the tresses away from the excavation. In a strong rock or cemented soil mass (e.g. strong in relation to the virgin stress field) the radial stresses at the perimeter of the excavation are nil and the tangential stresses are taken up by the material (the failure limit is not reached). In weaker masses, the tangential stresses cause the material in the perimeter to fail and material will fall into the opening causing the excavation to become larger. The material behind the fallen out material forms now the new perimeter of the excavation and will subsequently also fail. This is a progressive failing of the excavation. To stop this failing the material directly adjacent to the perimeter should not be allowed to fall out by support. In shallow underground excavations with no or small horizontal stress, arching effects may not be enough to deflect the virgin stress field from the opening and therefore a shallow underground excavation may need support which actually carries the groundmass above the tunnel.

H.4 Stand-up time and time effects

All groundmasses for which the stress-strain conditions change need time to adjust to the new situation by deformation. The deformation causes the groundmass to loose structure and to become weaker due to forming of new fractures, shearing of asperities on discontinuities, non-fitting of discontinuities, breaking of cement bindings between ground particles, etc. Therefore, the sooner after excavation the support is installed, the stronger the groundmass remains and, hence, the smaller the support required. Time effects also cause that an excavation may fail after a certain time span after excavation. The time between excavation and failure for underground excavations is called the ‘stand-up time’. Stand-up times range from negligible short to 1000’s of years. The stand-up time allows in many groundmasses the excavation to be supported before collapse commences. Most of the mechanisms governing the time-dependent mechanisms are only partially known and available mathematical expressions are not very reliable. The only way to assess stand-up time is by experience, which may be sustained by empirical classification systems (see below).

Time-effects cause many serious accidents in underground workings. Many engineers even those working underground do not realize the existence of stand-up times. Often an excavation is considered save because it has been standing for some time. This can be a fatal error when working underground. It requires experience to recognize a potential imminent collapse, but everybody should be aware that the time it has been standing is of no value to assess the safety. Furthermore, most collapses are preceded by warnings such as small movements in the surrounding groundmass, cracking sounds from the mass, or (small) pieces of material falling from roof or walls. These should be recognized as signals that a potential danger might be at hand.

H.5 Water and underground excavations

Water flowing in the excavation can be a nuisance, but is often also a real hazard. Water pressures reduce the effective stress between particles and between blocks and subsequently the shear strength. Water pressures and flow may also cause particles and blocks from the groundmass to be flushed into the excavation causing overbreak and instability and, possible, hazardous flooding of an excavation (Fig. H-1). If shotcrete (see below) is applied, flowing water will prevent the shotcrete from good contact with the groundmass. Therefore, water flowing into an excavation in large quantities has normally to be prevented. Draining is feasible if permeability of the groundmass is not too high or if sealing (impermeable) layers are present that limit the quantity of water to be drained. If all rock mass is highly permeable drainage will lower the groundwater table above the tunnel. This is normally unacceptable for social, environmental, and geotechnical reasons. Then the tunnel has to be sealed locally with, for example, freezing or grouting. Even if the water inflow is geotechnically no problem, it is not regarded as good engineering if large waterfalls happen to be in the middle of the excavation. Geotextiles or plastic can be used to divert the water to a drainage system or a permanent support has to seal the mass against water inflow.

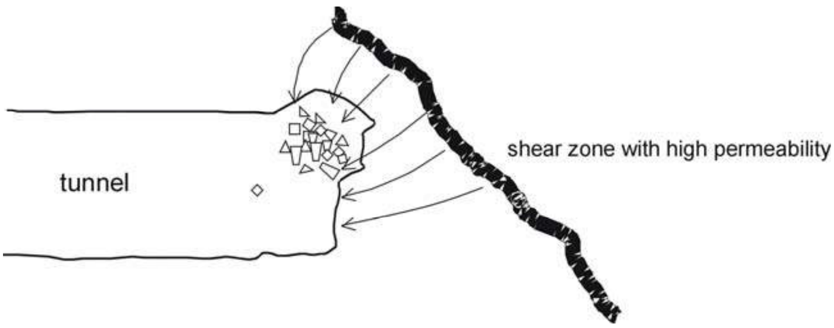


Fig. H-1. Water flow and pressures from localized high permeability zones may break through in tunnel before tunnel reaches the zone

It is not easy to estimate water inflow into an underground excavation before the excavation has been made. The theoretical background can be found in many books, but to determine the nature of the groundwater regime and the permeability of the strata is very difficult. In a homogeneous mass without discontinuities, the water will flow via the pores between the grains. For this, reasonable estimates can be made based on theoretical calculations. Most masses are, however, not homogeneous and not without discontinuities. Reliable estimations of the quantity of water can then only be made from boreholes, pumping and tracer tests and from pilot tunnels. Portals of tunnels may in particular be at risk because these are near to the surface and the mass near portals is often more permeable because of a higher number of discontinuities. Rainfall may then directly flow into the tunnel portal area. Building the portal in a dry season if such season exists can reduce this problem.

I EXCAVATION

I.1 Introduction

Excavations can be made with different means from by hand to highly mechanized tunnel boring machines (TBMs). The used type of excavation has to be suited to the type of soil or rock mass, the time constrains of a project, available skilled labor, and the economics of a project. It is important to realize that an excavation virtually always can be made by any type of method whatever the groundmass. Prisoners have made escape tunnels in rocks by using their nails and spoons. It obviously took ages to make the tunnels, but it was achieved. In normal projects, such an excavation method would not be possible because the labor is likely not to be found and it will take too much time.

Generally, two types of excavation methods can be distinguished: mechanical and blasting which can be further divided (Table I-1). Historically, blasting techniques were more used were nowadays a tendency exists to apply mechanical methods. The later have in many situations various advantages over the blasting methods. Mechanical hammering is often used for smaller works or for reducing the block size after

Table I-1. Various means of excavation

mechanical	digging	man-made/shovel/excavator
	cutting and grinding	borehole
		road header
		trench cutter
		tunnel boring machine (TBM)
		raise borer
	hammering	jack hammer
		hydraulic/pneumatic hammer
blasting		pre-splitting
		smooth wall
		conventional tunnel blasting
		conventional large hole blasting
specials		wood
		chemical expansion
		water (high pressure breaking or jetting)
		sawing (blade or steel cable)

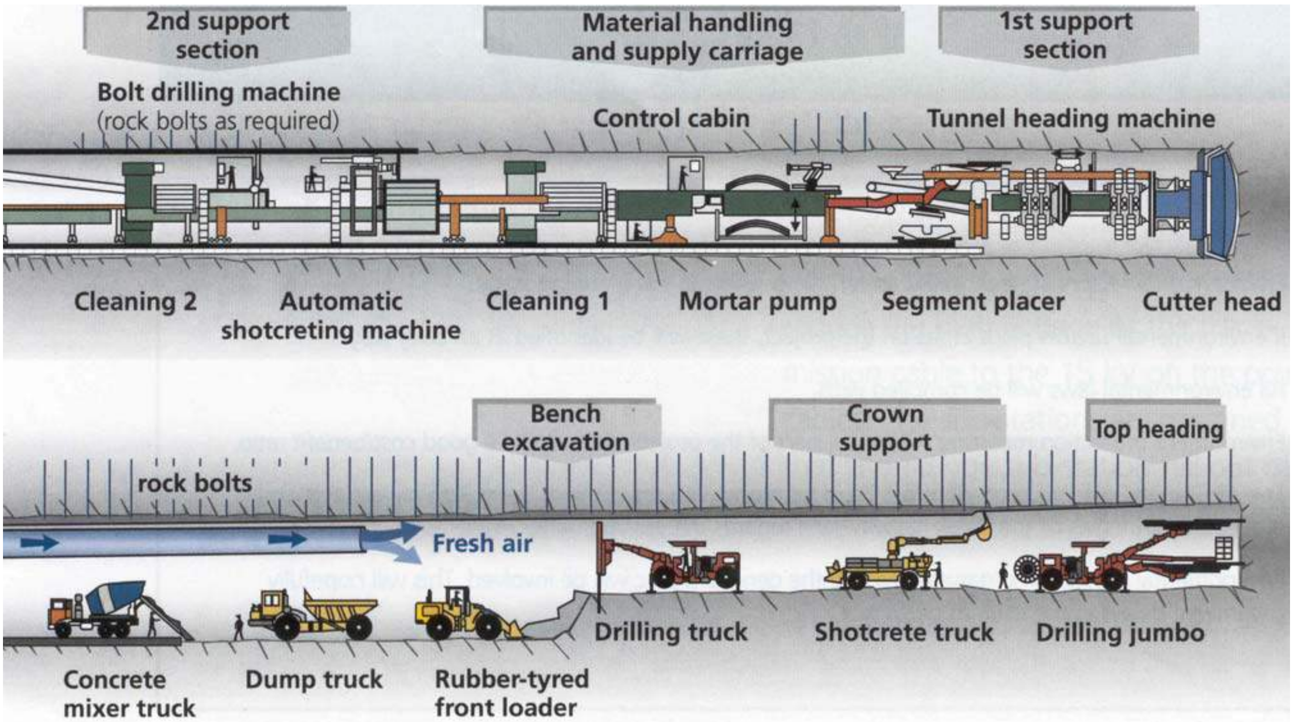


Fig. I-1. Tunnel excavation and support organization for the St. Gotthard tunnel; top mechanical excavation by TBM, bottom: drilling and blasting (© AlpTransit Gotthard AG., after AlpTransit, 2002)

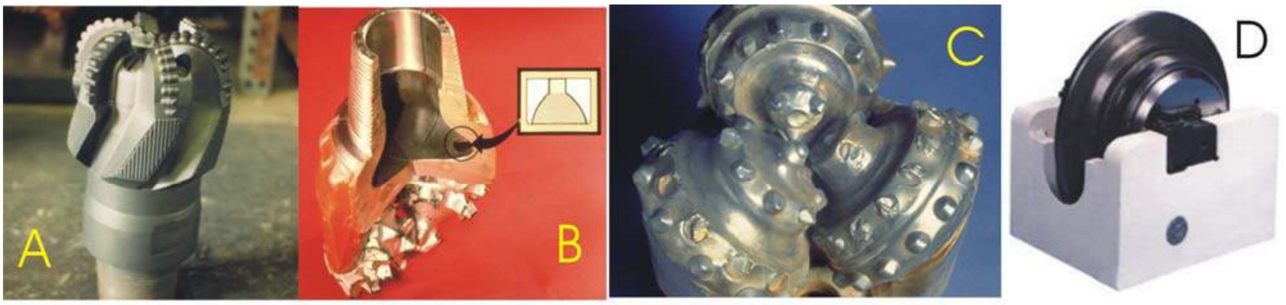
Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

blasting 'Specials' are excavation methods that make use of the expansion characteristics of wood or chemicals, use water under high pressure, use water for jetting to erode the rock mass, or use sawing techniques (for example, the marl mines in South Limburg, Netherlands). The specials are nowadays seldom used in underground excavations and are not further discussed.

The method of excavation has a considerable influence on quality of the groundmass forming the perimeter of an excavation. These will show more discontinuities that are mechanical if high stress levels occurred in the mass during the excavation (so-called 'backbreak'). Natural, hand-made, and bored exposures have in general not been subjected to high stress levels and, hence, show fewer new mechanical discontinuities than excavations made by blasting in the same rock mass.

For large excavations, it is often necessary to do the excavation in sequences starting with a small part of the excavation because the large excavation would not be stable long enough to install support (Fig. I-1 bottom).

I.2 Mechanical cutting and grinding excavation methods



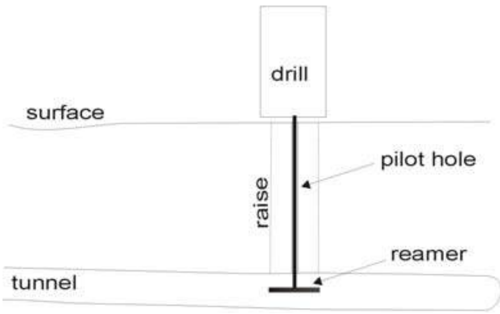
A) PDC (polycrystalline diamond compact), drag or fixed cutter bit. This drill bit uses synthetic diamond disks mounted on the cutting surfaces to continuously shear scrape away rock; B) and C) roller or tri-cone bit. A tool designed to crush rock efficiently while incurring a minimal amount of wear on the cutting surfaces. Invented by Howard Hughes, the roller-cone bit has conical cutters or cones that have spiked teeth around them. As the drillstring is rotated, the bit cones roll along the bottom of the hole in a circle. As they roll, new teeth come in contact with the bottom of the hole, crushing the rock immediately below and around the bit tooth. As the cone rolls, the tooth then lifts off the bottom of the hole and a high-velocity fluid jet strikes the crushed rock chips to remove them from the bottom of the hole and up the annulus. As this occurs, another tooth makes contact with the bottom of the hole and creates new rock chips. Thus, the process of chipping the rock and removing the small rock chips with the fluid jets is continuous. The teeth intermesh on the cones, which helps clean the cones and enables larger teeth to be used. There are two main types of roller-cone bits, steel milled-tooth bits and carbide insert bits. C) tri-cone bit with worn and broken teeth (Schlumberger Oilfield Glossary, photos courtesy of Mark S. Ramsey). D) disk bit (as mounted on a TBM cutterhead, ©Robbins, 2002)

Fig. I-2. Various drilling bits

All mechanical cutting and grinding methods are based on a mechanical device on which drag, roller or disk bits are mounted (Fig. I-2 and Fig. I-3) that cut and grind into the groundmass. The bits may be from steel, hardened steel, tungsten, carbide, or diamond (borehole bits). The machines can be hydraulically, electrically, or diesel engine powered. The forces are normally not very high to prevent breaking of the bits. In particular, impact forces between the bits and pieces of hard material (e.g. rock) should be minimal as these cause extensive wear and breaking of the bits. The excavation is a relatively slow cutting and grinding of the groundmass that breaks small pieces from the mass. The maximum diameters of excavations are typically circular holes up to about 8 m for boreholes and raise borers, and 15 m for tunnel boring machines (TBMs) (Fig. I-1 top and Fig. I-3). These are the present maximum dimensions. TBMs achieve an oval form of excavation by applying two or three cutting wheels (also known as

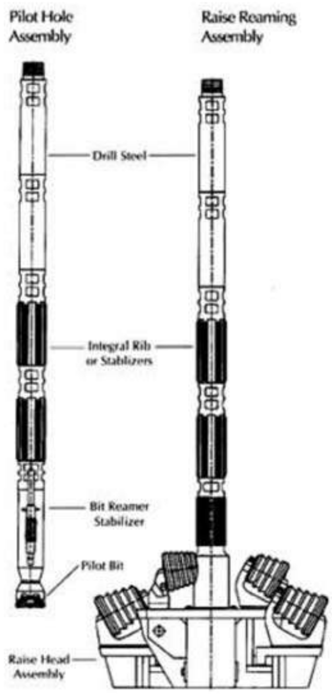
Fig. I-3. TBM cutting wheel with disk cutters. Diameter 8.89 m. St Gotthard tunnel (photo: © AlpTransit, AlpTransit, 2002) cutterheads) mounted in one machine, or by applying two or three TBMs, which bore en echelon overlapping tunnels. A raise-borer gives a circular vertical or sub-vertical shaft (Fig. I-4). A trench cutter is a special design for cutting shallow trenches from surface (fig ??), whereas a road header, which consists of a rotating cutting wheel mounted on, for example, a modified excavator (Fig. I-5) or in a shield (Fig. I-6), is flexible and can excavate any size and form of excavation.

All mechanical cutting and grinding methods are characterized by the following: low energy rate and consequently low stress levels at any time. This prohibits large damage to the rock mass beyond the perimeter of the excavation (e.g. a low backbreak). A very good control over the excavation is possible, resulting in minimum overbreak and a smooth perimeter without inserts. Other more practical advantages are that the process is continuous with the same labor employed continuously and that ventilation of the underground excavation is not as critical as for blasting although exhaust fumes of



RUC's Wirth HG330, South Africa

Top (surface) drill (after TMCC - Thyssen Mining, 2003)



Drill and ream assembly (after Mining Technologies International Inc., 2003)

4.3 m Reamer at Cameco's Eagle Point Mine, Saskatchewan, Canada

Mounting the reamer on the drill string (after TMCC - Thyssen Mining, 2003)

Fig. I-4. Raise borer

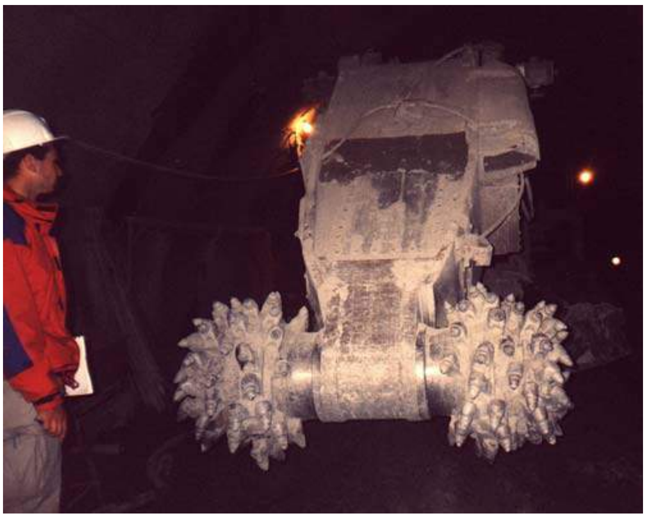


Fig. I-5. Road header

Fig. I-6. In-shield boom cutter (type of road header cutter boom mounted in a shield) (©TBM Exchange Int., 2002) loaders and other machinery should be expelled.

Economic viability

All excavations can be made with mechanical cutting and grinding methods in any groundmass, whether soil or rock. The question is whether the method is economically viable. If a rock mass consists of larger blocks and the intact rock strength is high it will take a long time before a mechanical cutting and grinding device has excavated the space. Secondly, the wear on cutters and machinery may be high and consequently the project costs. Until some years ago, mechanical cutting and grinding excavation techniques (except diamond rotary drilling) were restricted to soil and moderately strong rock masses with intact rock strength not exceeding 50 MPa. The techniques are, however, rapidly advancing and mechanical cutting and grinding methods become economically viable in rock masses with higher intact rock strengths and larger block sizes. In addition, in circumstances that are not economical they are often used for other reasons, for example, if blasting is not allowed for fire risk or vibrations. A disadvantage for some methods, are the relatively (very) high initial costs, in particular, for equipment such as a TBM.

I.3 Mechanical hammering

Jackhammers and hydraulic (Fig. I-7) or pneumatic hammers are examples of excavation by mechanical hammering. The devises are normally mounted on shovels or excavators. The length of the hammers can be up 5 m with a diameter of about 25 cm. The forces on the hammer may be large and rock is broken under the impact of the hammer. The damage to the rock mass depends for a large part on the size of the hammer. Large hammers may cause considerable damage to the rock mass. Note that in the adjustment factor for pneumatic hammering in Table I-2 is in the same order as good conventional blasting.

Fig. I-7. Hydraulic hammer (©Terminator, 2002)

I.4 Blasting

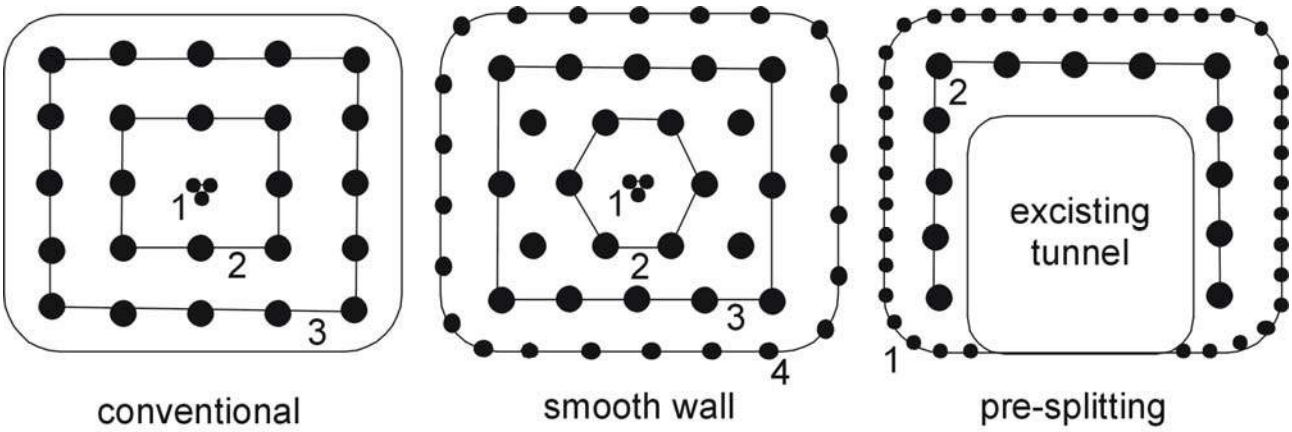


Fig. I-8. Simplified blasting patterns (the numbers indicate the order of the rounds of blasting)

In its most simple form, blasting consists of drilling of a series of holes that are filled with explosives. The holes are drilled and blasted according to a blast pattern that describes the location of the holes, the quantity and type of explosives, and the order and delay times between the holes as they are normally, not all blasted at the same time but with small time intervals (10s to 100s of milliseconds). The first holes to be blasted are normally smaller holes in a triangular or pyramidal pattern (Fig. I-8) in the middle of the excavation (the wedge or burn-cut) to create space for the rock fragments from the following rounds of larger production holes. After the blast the broken pieces of the rock mass are removed ('mucking') (Fig. I-9) and the process can start again. Diameters of holes are in order of 2.5 -10 cm and length in the order of 0.5 - 7 m, depending on the quality of the rock mass and type of explosives. In new to start projects, the patterns should be specially designed for the particular work, rock mass, and type of explosives used.

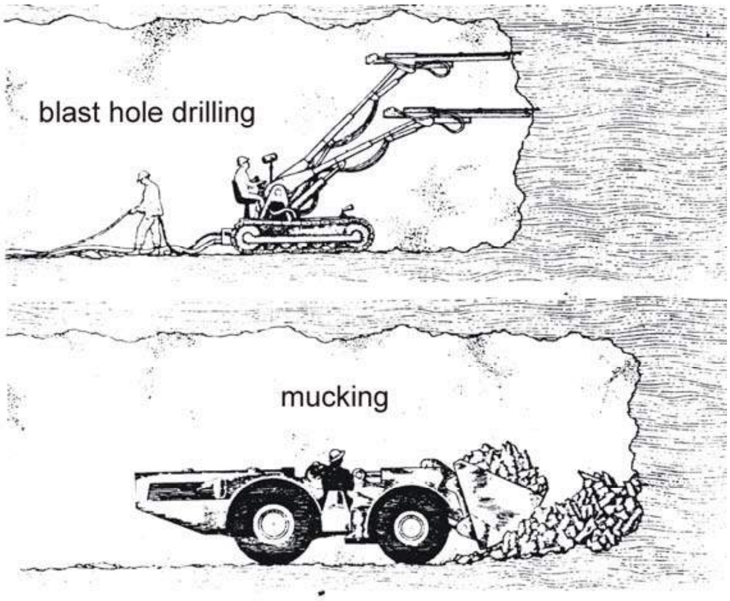


Fig. I-9. Blasting is not a continuous process!. Blast hole drilling (top) and mucking (bottom). Both activities have to be subsequent in time

Smooth wall blasting and pre-splitting

In smooth wall blasting the number of holes along the perimeter of the excavation is increased, and the diameter and quantity of explosives per hole decreased (Fig. I-8). These holes will be blasted simultaneously as last round. If properly executed the excavation will be relatively smooth and half of each hole will remain visible. Back- and overbreak will be low. In pre-splitting the number of holes along the perimeter may be further increased and the diameter decreased (Fig. I-8). The pre-split holes are simultaneously blasted before the main holes and a 'split' along the perimeter is created from one pre-split hole to the next. The split will act as an open discontinuity. Shock waves of the main holes will be reflected for a large part on the split. Hence, the shock waves of the main holes do not enter the mass outside the perimeter and the reflected energy will help in breaking up the middle part of the excavation. If properly done half of each pre-split hole will remain visible and the damage to the mass beyond the perimeter will be minimal. Pre-splitting for full-face tunnel blasting is not very useful as the tight structure and stresses in the face will prevent the development of a split. However, for secondary blasting such as widening an existing tunnel, pre-splitting may give good results.

Conventional large-hole blasting

Conventional large-hole blasting makes use of blasting holes with diameters of about 15 - 25 cm. The quantity of explosives in each hole is large. The rock mass will normally be shattered (Fig. I-10). It is used for underground or surface mining were damage to the surrounding rock mass is of less concern. Mostly it is not used in civil engineering except if large quantities of a rock mass have to be removed.

Fig. I-10. Large hole blasting of a quarry face

Blasting in karstic masses

Blasting in karstic rock masses require special measures. The quantity of explosives should be adjusted to the fact that the rock mass consists partly out of open space or soft karst hole filling material, and the blasting holes should be lined as otherwise the karst holes will be filled with explosives too leading to excessive over blasting.

Special blasting

Blasting is a science in itself. Many different techniques exist to break down rock and rock masses depending on the required size and form of the resulting rock fragments. An example is given in Fig. I-11. The charging of only part of the hole causes that the resulting fragment size in ore and waste (the slate) is different. Digging can then be done on sight between ore and waste.

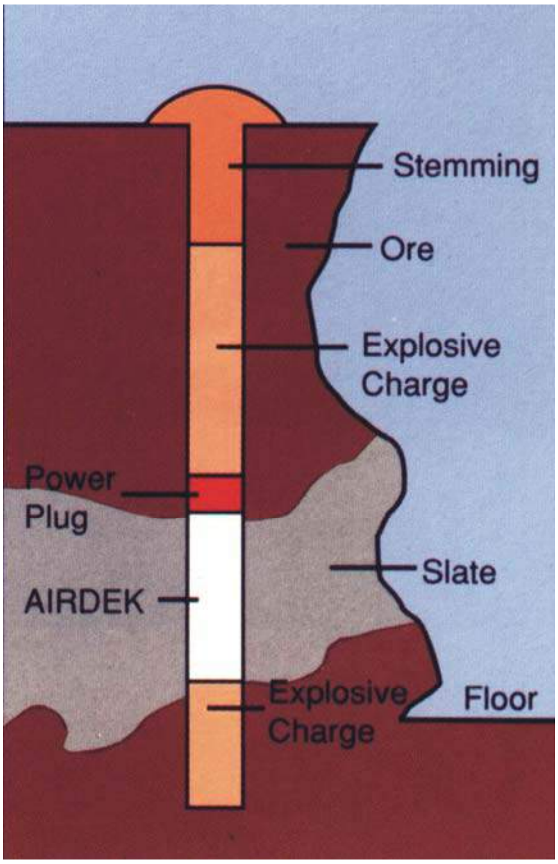


Fig. I-11. Specialty blasting; the charging of the hole controls the fragment size in different formations (after AIRDEK in Downline, Feb, 1991)

I.4.1 Advantages and disadvantages of blasting

Conventional tunnel blasting is characterized by high-energy rates resulting in heavy shock waves and consequently damage (backbreak) to the rock mass outside the designed perimeter of the excavation. Generally, the control on the excavation size is poor and the resulting roof, walls, and floor will not be very smooth and have many inserts. Practical disadvantages are the discontinuity in the process with some of the specialized labor only employed for some of the time, and critical is good ventilation, as the poisonous blasting gasses have to be expelled from the tunnel before the laborers can return to work. Platforms can be used in large diameter tunnels that allow drilling of holes and mucking of blasted rock to happen simultaneously. In many countries, extensive and expensive security measures are required because of the presence of explosives.

Blasting can cause severe damage of the rock mass (Table I-2). The shock wave from the detonation causes high stress levels leading to cracking of intact rock, the opening of integral discontinuities into mechanical discontinuities, and widening of existing mechanical discontinuities. Gasses discharged by the explosion cause similar effects, but giving more opening of existing discontinuities than cracking intact rock or opening of integral discontinuities. Additionally, the roughness of discontinuity planes may be affected by blasting. Asperities on discontinuity planes can be sheared or crushed due to the shock wave, and rock blocks may be displaced, giving less rough and non-fitting discontinuity planes resulting in lower shear strength. The effects from blasting are reduced if special techniques are used like pre-splitting or smooth wall blasting. These methods, however, increase the costs considerably.

Blasting techniques have changed over the years; the numbers of boreholes blasted in one round and the size (diameter and length) of the boreholes have increased. In addition, where in the past blasting was directed towards creating gasses (slow burning or detonating explosives, mostly from black powder), nowadays blasting is more directed towards creating a shock wave (fast burning explosives). A rock mass will have most open discontinuities in the direction of the free face. A blast creating gasses will therefore work more in the direction of the free face than inwards into the rock mass. Shock waves work in all directions, but are reflected by open discontinuities and, hence, the energy is directed more into the rock mass. Therefore in more recent excavations, the rock mass is more damaged in the direction away from the free face than in older excavations.

Often blasting is not done very appropriate. Too large holes with too much or too heavy explosives are used for the type of rock mass. This results in additional damage to the rock mass. It is difficult to assess the quality of blasting in advance. However, a proper site investigation will result in a proper prescription of the blasting patterns, delay times, and quantities and types of explosive. The quality of the blasting engineer is vital. The contracts for the blasting sub-contractor should be carefully drafted as often the mere fact that the contractor is paid for the quantity of explosives or the quantity of blasted rock lures the contractor into using maximums of explosives to do the job as fast as possible with maximum profit, but without much concern for the results.

I.5 Quantification of rock mass damage (backbreak) due to the method of excavation

The origin of an exposure has a considerable influence on quality of the rock mass. The excavation will show discontinuities that are more mechanical if high stress levels occurred in the rock mass during the excavation, e.g. the backbreak will be larger. Table I-2 shows a division of excavation methods in use for underground and surface excavations with quantitative values for the damaging influence of the method of excavation on the rock mass.

Table I-2. Excavation damage factors for a rock mass (the factors are multiplied with the classification results of SSPC or MRMR to account for the method of excavation)

method of excavation		SSPC factor	MRMR factor
natural/hand-made		1.00	
boring			1.00
pneumatic hammer excavation (*1)		0.76	
pre-splitting/smooth wall blasting		0.99	0.97
conventional blasting with result:	good	0.77	0.94
	open discontinuities	0.75	0.80
	dislodged blocks	0.72	
	fractured intact rock	0.67	
	crushed intact rock	0.62	
SSPC factors after Hack (1998), MRMR after Laubscher (1990) note *1) This value is based on hammer sizes up to 5 m with a diameter of 0.2 m			

In general, natural exposures have not been subjected to high stress levels. Therefore, these show fewer mechanical discontinuities than excavated exposures in the same rock mass. The excavation of a rock mass by hand and by mechanical excavators causes mostly also a relatively small amount of damage and thus fewer induced mechanical discontinuities. High stress levels such as may occur in blasting cause more backbreak.

Overbreak may also become larger with larger stress levels during excavation because the number of mechanical discontinuities in (e.g. the backbreak) is increased, giving more opportunities to rock blocks to fall out the excavation walls.

I.6 Excavation in sequences

For large excavations made with a road header or blasting it is often necessary to do the excavation in sequences. This may only be in two stages as shown in Fig. I-1 (bottom) or in more stages (Fig. I-12 and Fig. I-13). The advantages are that the unsupported walls and/or the free unsupported span are smaller. In the time between different excavation phases support can be installed for the already excavated part. A second advance is for blasting excavation that the number of blast holes and hence the quantity of explosives which is detonated in one round will be less. Hence, shock waves will be smaller.

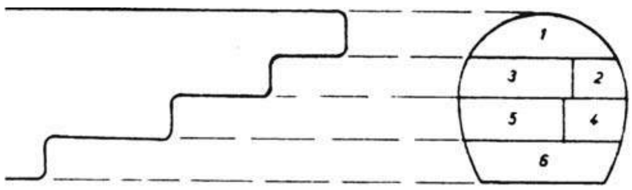


Fig. I-12. New Austrian tunneling technique sequence in weak rock

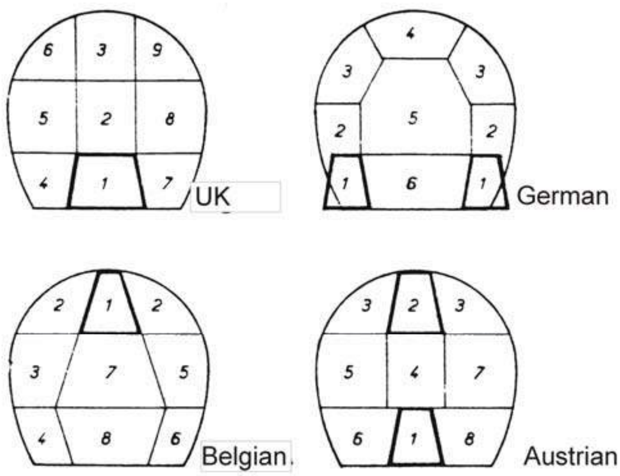


Fig. I-13. Tunneling with partial excavation sequences

J SUPPORT

J.1 Introduction

Underground excavations have to be supported to maintain the opening in many groundmasses. The thinking on support has changed over the years. In the past support was installed with the idea to carry the groundmass. Nowadays it is realized that this is not necessary, but that the mass should carry itself. Support is only needed to help the mass carrying itself. Often a thin layer of shotcrete acting as glue to maintain the integrity of the soil or rock mass in the excavation perimeter is sufficient whereas in the past heavy steel sets with lots of wood timbering would have been installed to obtain the same result.

Any movement allowed in a groundmass will result in a loss of structure of the mass and in a weaker mass. Establishing new equilibriums between stress and strain and the movements required to obtain the equilibrium cost time. Therefore, the faster the support is installed the less strength is lost. It should however be realized that some relaxation of the ground is always necessary to obtain the arching effects with which the ground caries itself. Generally, the time required to install support is enough time to obtain the arching effects. In some situations, it can be necessary to delay support installation or allow additional deformation by other means (as making special de-stressing slots) because it is simply impossible to withstand the stresses if not done. This normally happens if large stresses and deformation occur, which is mainly a situation happening in underground mining. Depending on the time in relation with the excavation process, it is normal to make a differentiation in the types of support: 1) pre-excavation support and drainage, 2) support during excavation, and 3) permanent support (Table J-1).

Table J-1. Support types

<table>
<tr><th colspan="2"></th><th>pre-excava-tion</th><th>during excavation</th><th>permanent</th></tr>
<tr><td colspan="2">freezing</td><td>X</td><td>X</td><td></td></tr>
<tr><td colspan="2">grouting</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td colspan="2">drainage</td><td>X</td><td>X</td><td>X (*1)</td></tr>
<tr><td rowspan="3">forepoling</td><td>jet grouting</td><td rowspan="3">X</td><td rowspan="3">X</td><td rowspan="3">X</td></tr>
<tr><td>spiling</td></tr>
<tr><td>pipe roof</td></tr>
<tr><td colspan="2">shield (with or without face plates, face shield, pressure, hydro, bentonite, or EPB)</td><td></td><td>X(*2)</td><td></td></tr>
<tr><td colspan="2">caisson</td><td></td><td>X</td><td></td></tr>
<tr><td rowspan="2">shotcrete (with or without wire mesh)</td><td>on face</td><td></td><td>X</td><td></td></tr>
<tr><td>on walls, roof and floor</td><td></td><td>X</td><td>X</td></tr>
<tr><td rowspan="2">rock bolt, dowel, anchor</td><td>in face</td><td></td><td>X</td><td></td></tr>
<tr><td>radial in wall, roof, or floor</td><td></td><td>X</td><td>X</td></tr>
<tr><td colspan="2">rock bolts/anchors with straps</td><td></td><td>X(*3)</td><td>X</td></tr>
<tr><td colspan="2">steel</td><td></td><td></td><td>X</td></tr>
<tr><td colspan="2">concrete</td><td></td><td></td><td>X</td></tr>
<tr><td colspan="2">cut-and-cover</td><td colspan="3">not a support type but methodology of working</td></tr>
<tr><td colspan="2">timber</td><td></td><td>X (*4)</td><td>X (*4)</td></tr>
<tr><td colspan="5">notes:
*1) In some situations it is feasible to maintain drainage for the lifetime of the tunnel (e.g. mining)
*2) See TBM, chapter K
*3) Generally very time consuming and, hence, mostly not effective during excavation.
*4) Questionable as support means.</td></tr>
</table>

J.2 Surface excavations

Many support types can be used also for surface applications such as maintaining stability of a road cut or the walls of a quarry. For surface excavations the mechanism of loss of structure is just as important. Also in surface structures any movement of the rock mass should be prohibited to avoid loss of structure and, hence, loss of strength.

J.3 Freezing

The ground is frozen by pumping cooling agent trough a double-tube pipe system in boreholes either from surface, a small diameter service tunnel, or from so-called feeder holes drilled from the tunnel (Fig. J-1). Apart from the scale the process is very comparable to a household refrigerator. The cooling agent is pumped in the borehole through the inner tube and returns through the outer tube. The water in the groundmass is frozen and binds the particles and blocks together, and makes the mass impermeable. It is essential that the ground contain water. Without water, the mass cools, but there is no gluing effect. Even if only some minor layers do not contain water, the method is questionable. The not frozen layers will deform due to the expanding surrounding frozen mass and subsequently de-stress when the surrounding layers are thawed. This may lead to stress relief throughout the whole perimeter of the excavation and subsequent instability. The expansion of the groundmass may displace other nearby structures or foundations of surface structures, and surface heave may be a problem if applied for shallow excavations. The method is expensive.

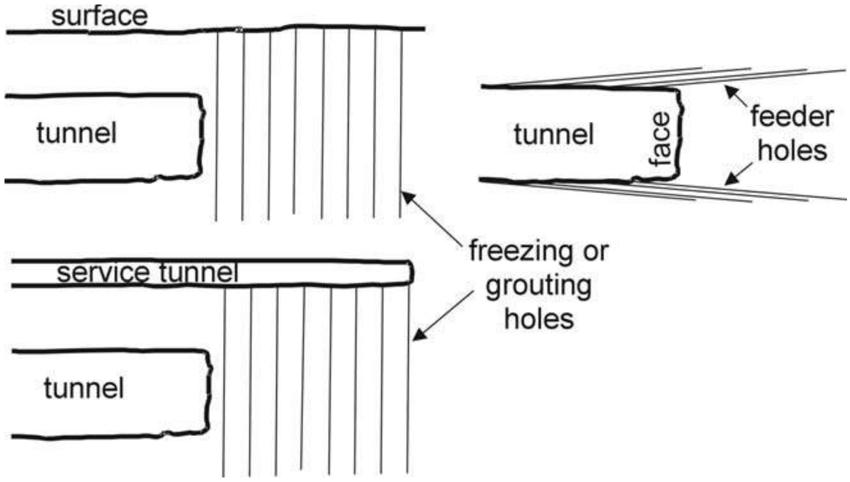


Fig. J-1. Freezing or grouting from surface, service tunnel, or feeder holes (note that the freezing and grouting holes, if cored, will also give information about ground conditions ahead of the tunnel face)

J.4 Grouting

Grouting implies pumping a cement milk or resin into the ground (the ground is *grouted*). This glues the particles and blocks together. The grout is pumped into the groundmass from surface, a small diameter service tunnel, or from feeder holes (Fig. J-1). Grouting from feeder holes is also known under the name forepoling. In soil grouting can be done as jet grouting. A large advantage of the method is that the support is also present as permanent support after excavation. It should be noted that some permeability of the groundmass is essential. If the ground is too impermeable, the grout will not reach all locations and some areas will not be glued together. Generally, the method works very well to improve the strength and deformation characteristics of the groundmass. Whether it will also make the groundmass impermeable is questionable. In many cases the milk or resin will not be homogeneously spread through the groundmass and the ground will not be completely impermeable after one round of grouting. Additional grouting rounds are then necessary to stop leakages. Special care should be taken that the grouting is not done with a too high pressure. A too high pressure will cause the groundmass to break and permeability after the grout process may have increased rather than decreased. In particular, the grouting contractor should be kept under close control, because they are often paid for the quantity of grout, and hence have a tendency to pump as much as possible under high pressure.

J.5 Drainage

Drainage is not really a support measure but often necessary to allow excavation. Water flowing in the tunnel can be a nuisance, but is often a real hazard. Water flow may cause particles and blocks from the groundmass to flush into the excavation causing overbreak and instability (Fig. H-1). Secondly, if shotcrete (see below) is applied as support, flowing water will prevent the shotcrete from good contact with the groundmass. Therefore, water flowing into the excavation in large quantities has normally to be prevented. Draining is feasible if permeability of the ground mass is not too high or if sealing (impermeable) layers are present that limit the quantity of water to be drained. If all rock mass is highly permeable drainage will lower the groundwater table above the tunnel. This is mostly unacceptable for social, environmental, or geotechnical reasons, for example, excessive settlement due to water extraction from shallow clay and peat layers. In such a case, the tunnel has to be sealed locally with, for example, freezing or grouting. Although civil engineering excavations are normally drained forever, it is not regarded as good engineering if large waterfalls happen to be in the excavation.

J.6 Forepoling

Jet grouted beams (see also grouting from feeder holes), grouted bolts, or pipes of steel or concrete can be installed in soil or rock masses as a shield inclined from the tunnel around the face as pre-execration support similar to grouting from feeder holes (Fig. J-1 and Fig. J-2).

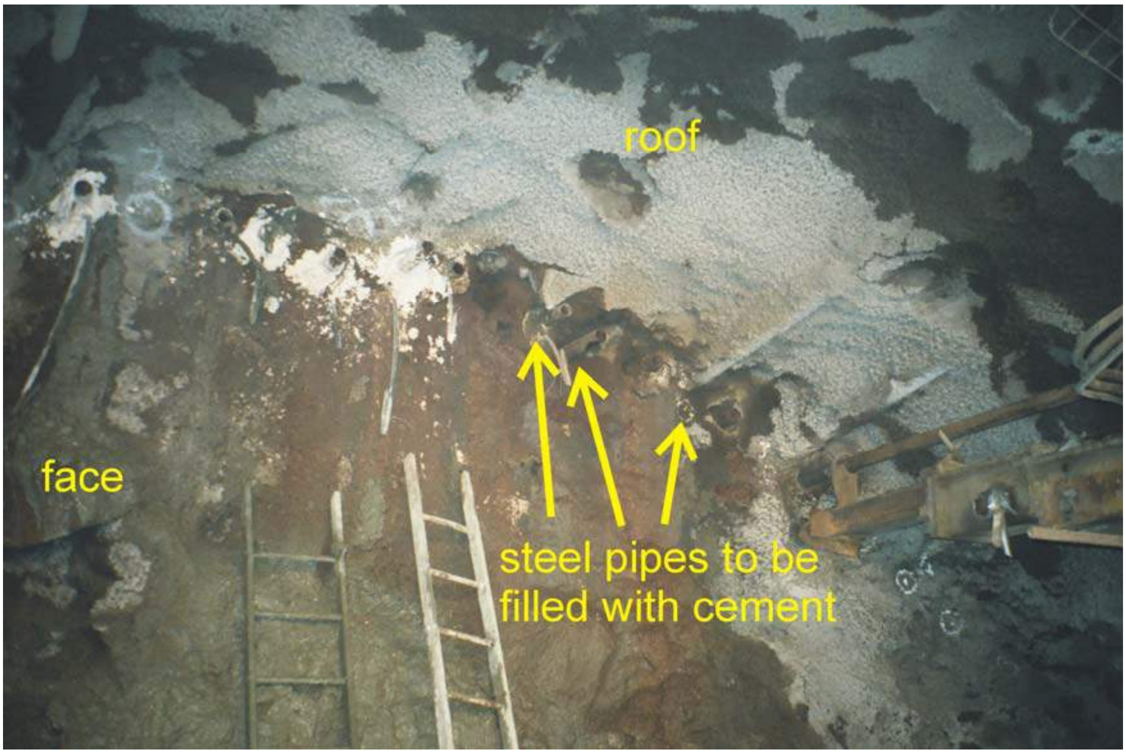


Fig. J-2. Forepoling

J.7 Caisson tunneling

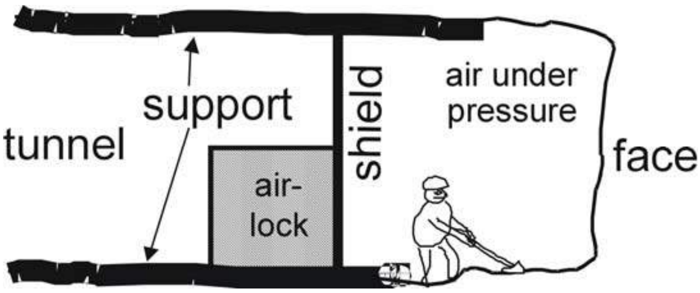


Fig. J-3. Caisson tunneling

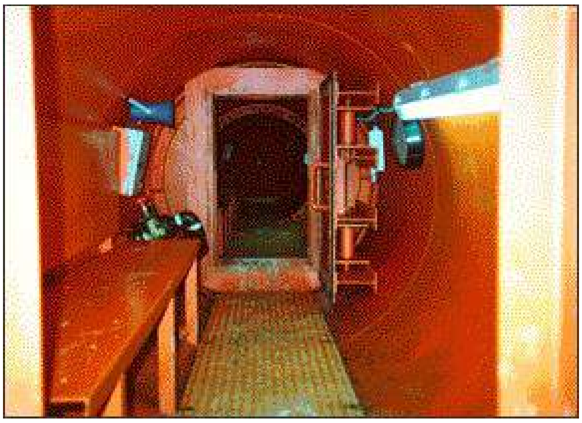


Fig. J-4. Man lock in shield for compressed air tunneling (Singapore Metro: Orchard Road station and tunnel) (photo: Traylor)

In caisson tunneling a shield is installed in the tunnel. Between the shield and the face of the tunnel, the air is pressurized (Fig. J-3and Fig. J-4). Labor and machinery necessary for the excavation pass through an air lock in the shield and work under compressed air. The method only works for cohesive or cemented ground and if water is present the permeability channels in the face should be small and water pressures relatively low. If not, the face will sag or water will flow in the excavation. The labor is expensive, as working under compressed air is considered unpleasant and is not very healthy (risk of caisson decease). In an emergency, the laborers have first to be de-compressed before they can leave the area under compression.

J.8 Shield and bentonite shield tunneling

In soil and weak rock masses, it may be necessary to support the area where excavation takes place by, for example, a shield. The shield is often included in a Tunnel Boring Machine (TBM, ch. K) after the cutting wheel, but this is not necessary (Fig. J-5).

Fig. J-5. Digger shield. The ground moves into the shield under its own pressure. The extraction of the ground from the shield controls the progress. Permanent support is installed behind the shield (photo. Traylor)

In the cylindrical shield a vertical shield can be installed so that the area enclosed by the cylindrical shield, the vertical shield and the face can be under pressure. The space between the face and the shield can be filled with water or air under pressure. The air or water pressure prohibits its flow of water out of the ground mass and the flush of ground mass particles out of the face.

Bentonite may be added to TBM-shield combinations if the ground mass is too loosely packet. In loosely packet ground, the face may sag and the cutting wheel may become clotted with face material. Adding bentonite to the water improves this as it gives a higher counter weight against the soil and rock mass particles in the face, glues the soil and rock particles in the face together, and closes the permeability channels in the ground (forming a so-called 'bentonite cake'). The method is very popular in soil and is also used in (weak) rock masses. The control on the pressures is vital in combination with cutting rates, advance rates, and expulsion of excavated material. This requires highly experienced and skilled labor and operators. Shield tunneling is further discussed in ch. K (Tunnel Boring Machine).

J.9 Gunite and shotcrete

Gunite is a mixture of sand/cement/water sprayed on the ground for waterproofing. It is usually sprayed in thin layers on wire mesh that is held to the rock by bolts and serves to give structural strength to the gunite and prevent cracking. Shotcrete (Fig. J-9d and e, Fig. J-6, Fig. J-7, Fig. J-8, Fig. J-23, and Fig. J-24) is a quick setting mortar sprayed on the groundmass. It is applied in layers about 5 cm thick at a spray velocity of about 150 m/sec and closely follows the excavation contours. Additives can be added that shorten the setting time allowing thicker layers to be sprayed in one layer, also fibers of plastic or steel (up to about 5 cm length) can be added to give tensile strength. Multiple layers can build up to a final layer of (in principle) unlimited thickness; however, mostly the maximum is around 40 to 50 cm. In thicker layers wire mesh, mats of steel, and/or steel beams are installed in the shotcrete as reinforcement to give tensile strength. Bolt support is often combined with shotcrete. Shotcrete supports the mass by keeping all even the smallest, particles of the mass at its location by gluing the mass together. This prevents deformation and movements. The first layer can be sprayed directly after excavation to minimize deformation and to prevent blocks falling out. If necessary also the face can be sprayed. Shotcrete is presently one of the most popular forms of support. Shotcrete can be applied in two ways: dry and wet. In the dry process, the water is mixed in the nozzle with the cement and sand particles. In the wet process, the water is already mixed in the cement - aggregate mixer. Dry shotcrete is easier because the pipes do not need to be cleaned during interruptions, but the mixing with water is not so good and many particles will not stick to the objects being sprayed. This creates dust and forms a health risk for the laborers.

Fig. J-6. Shotcrete applied at a road site (Battle Mountain, Colorado, USA; photo courtesy: Alyssa Kohlman)

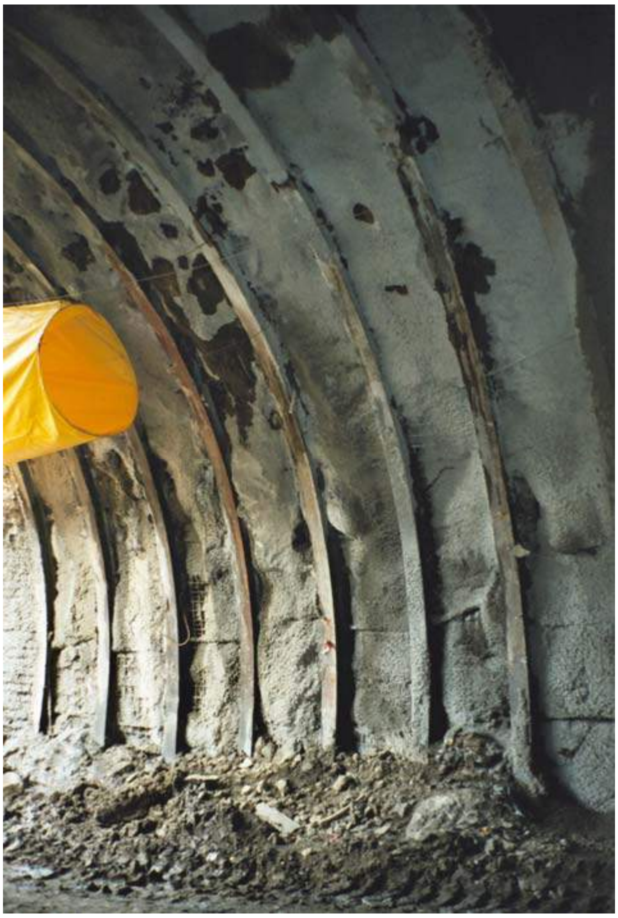


Fig. J-7. Shotcrete with fixed steel beams and mesh (Madeira, Portugal)

Fig. J-8. Shotcrete nozzle mounted on an arm on a bulldozer (Madeira, Portugal)

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

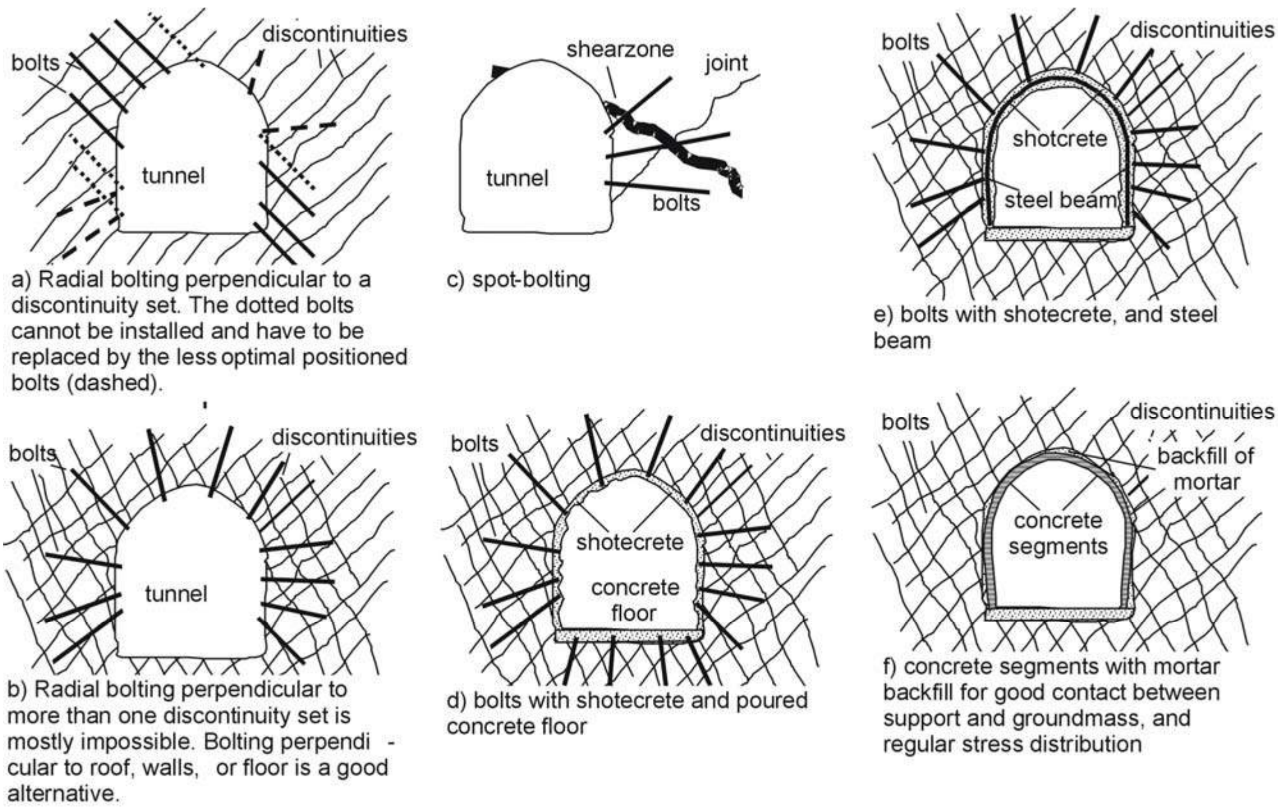
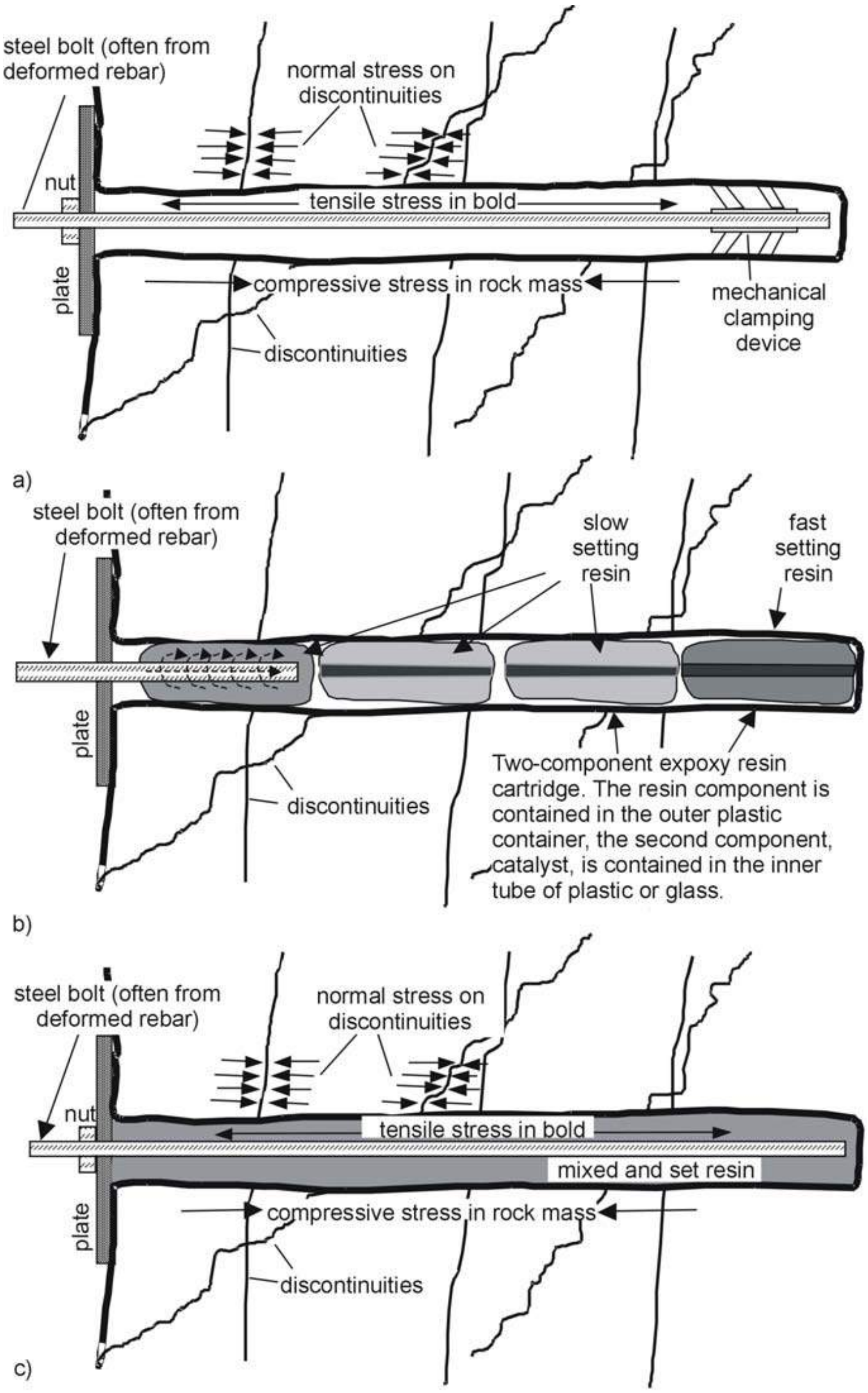


Fig. J-9. Various support types and combinations

J.10 Bolts, dowels, and anchors

Bolts, dowels, and anchors are three similar types of support that work by keeping the groundmass together and providing reinforcement in the mass. The naming is not clearly defined, generally, an anchor has a flexible cable and is long (ranging from meters to many 10s of meters), whereas a bolt is stiff and relatively short (0.5 – 10 m) as the flexible cable can be far larger and still be handled with ease. Both bolts and anchors may be artificially tensioned. This in contrary to dowels which are just a relatively short length (0.5 – 2 m) grouted in a hole, of steel, rebar, cable, wood, bamboo or any other material with tensile strength.

Bolts and anchors (Fig. J-9 and Fig. J-10) can best be compared to screws as used in household appliances. They keep the mass together similar to screws that keep panels of for example wood together. The bolt or anchor is tensioned and this creates a compressive stress between the particles and pieces of the mass. The compressive stress also causes the shear strength between particles and along discontinuities to increase (Fig. J-10). Bolts and anchors can be used in soil or rock masses, but are most effective in rock masses with reasonable intact rock strength and a not too small block size. If the intact rock strength is too low, the rock mass will squeeze or shear along the bolt or anchor, and if the block size is too small the small blocks will be squeezed out in-between the bolts or anchors. In these types of ground bolts and anchors are normally combined with grouting (to be done before the bolts or anchors are installed) and serve then as reinforcement in the grouted soil or rock mass.



A borehole is filled with cartridges containing two -component resin (b). The rebar is forced in the hole while rotating and brea ks the cartridge and mixes the resin and catalyst components. When the deepest installed fast -setting (hardening), cartridge has set, the rebar is tensioned. The resin contained in the other cartridges sets later, gives additional strength, and protects the rebar from corrosion (c).

Fig. J-10. Mechanical bolt (a); resin bolt (b and c)

J.10.1 Tensioning

In support for underground and surface excavations, various types are in use. Important is whether tensioning is artificial or through movements in the mass. Artificially tensioned bolts have a thread and nut system to tension the bolt and anchors are normally tensioned with jacks and a retaining clamp system to keep the tensile stress on the cables. Bolts are fastened in the mass by cement or epoxy resin, by clamp systems (Fig. J-10) or by the elastic expansion of steel (Swellex© and Split-set© types). Anchors are normally fastened with cement or resin.

Fig. J-11. Swellex© rock bolt. Top installation before expanding, below: after expanding due to water pressure (after Atlas Copco, 2002)

J.10.2 'Swellex©'

'Swellex©' works by expanding a steel tube in a hole (Fig. J-11). A hole is drilled with a diameter slightly larger than the tube. The tube is inserted and a pump with very high compression is connected to the tube via a one-way valve. Water is pumped in the tube under very high pressure. This causes the steel of the tube to expand and fit exactly to the irregularities of the rock. The interconnection between the steel and the rock provides then the friction between the tube and the rock. If the rock mass deforms the tube will be tensioned.

J.10.3 'Split-set©'

A 'Split-set©' consists of a steel tube slit open along the length the tube (the 'split') (Fig. J-12). A hole is drilled with a slightly smaller diameter than the 'split-set'. The 'Split-set' is inserted in the hole with force. Because the 'split-set' has to fit in the slightly smaller diameter hole, the split is closed. The steel of the split-set will be under bending stresses and the elastic response of the split-set will cause the steel to be pressed against the rock. The friction between the steel and the rock provides then the shear strength along the split-set. If movements in the rock mass occur the split set will be tensioned. The advantages of the 'split-set' are that installation is fast and easy, and that the 'split-set' will maintain working even if very large deformations and movements occur in the rock mass. The disadvantage is that 'split-sets' are relatively expensive.

Fig. J-12. 'Split-set' (photo: Crowder Supply Co Inc., 2003)

J.10.4 Self Drilled Rock bolt" (SDR©)

A so-called "Self Drilled Rock bolt" (SDR©) or hollow stem auger type of bolt can be used in poor ground in which a drill hole does not stand open (Fig. J-13. These are drilled into the mass, and then grout is injected through the hollow bolt or stem and into the surrounding mass.

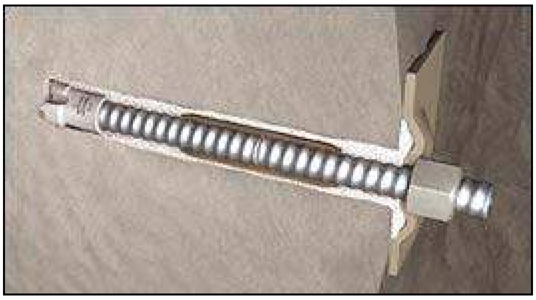


Fig. J-13. Self Drilled Rock bolt (photo: Mining & Construction; Atlas Copco, 2003)

J.10.5 Dowels

The cheapest forms of rock bolting are dowels. A hole is drilled and apiece of steel, rebar, cable, wood, or even bamboo is inserted. The hole is further filled with cement or (more expensive) resin. Deformation and movements in the rock mass will give the tension stress in the dowel.

J.10.6 Anchors

Anchors (Fig. J-15) work similar to the mechanical or grouted rock bolts with artificial tensioning (mostly) a flexible cable (or cables) is used instead of a rebar. The advantage is that the flexible cable can be far larger than a rebar, and still can be handled with ease. Fig. J-14 shows large scale anchoring of the foundations for the abutments of a bridge.

J.10.7 Residual strength after failing

The advantage of bolts based on a clamping system or on the elastic expansion of steel is the maintaining of residual strength even after exceeding the maximum shear strength of the bolt. This in contrary to cement or resin types which if fail loose all strength.

J.10.8 Bolting patterns

Bolts can be installed in soil or rock masses as an umbrella inclined from the tunnel over the face as forepoling. These serve as a pre-excavation support. Bolts can also be applied to stabilize the face of an excavation if required. For radial bolting different opinions exist on how bolts should be installed: as much as possible perpendicular to as many as possible discontinuities (Fig. J-9a), or just perpendicular to the wall, roof, or floor of the excavation (Fig. J-9b). The first is often difficult to execute and requires skilled labor. The later is easy to do, also by unskilled labor. It is clear that bolts should be installed perpendicular to the discontinuities in (academic) situations with one clear discontinuity set. Most masses contain more than one set and it will mostly be impossible to install the bolts perpendicular to all discontinuities. The easier installation of bolts perpendicular to the walls and roof is cheaper so that more bolts can be installed and the final support is normally better even if theoretically less effective. Bolts should be installed perpendicular to the discontinuities for individual situations where a few bolts are to be installed because a limited amount of mass is instable (Fig. J-9c). This normally is called 'spot bolting'.

Fig. J-14. Large scale anchoring of the foundations for the abutment of a bridge. Only the anchors plates (small black squares) can be seen on top of reinforced concrete beams (after Geoconsult, Salzburg, 1978)

J.10.9 Corrosion

Bolts, dowels, and anchors used in civil engineering need to be protected against corrosion. This can be achieved by filling the hole with grout or resin, coating of the bolts and cables, or using an impermeable sheathing along the bolts and cables (Fig. J-15). During installation, special care should be taken that the coating or sheathing is not damaged.

J.11 Bolts with straps or cables

Metal straps or tensioned cables can be placed between bolts. The straps will keep blocks between the bolts on their location and cables will force the blocks in the mass together and, hence, create a tighter structure.

Generally, only used in hard rock where large stresses are expected. The straps and cables have to follow the excavation perimeter as good as possible. The installation is labor intensive and time consuming.

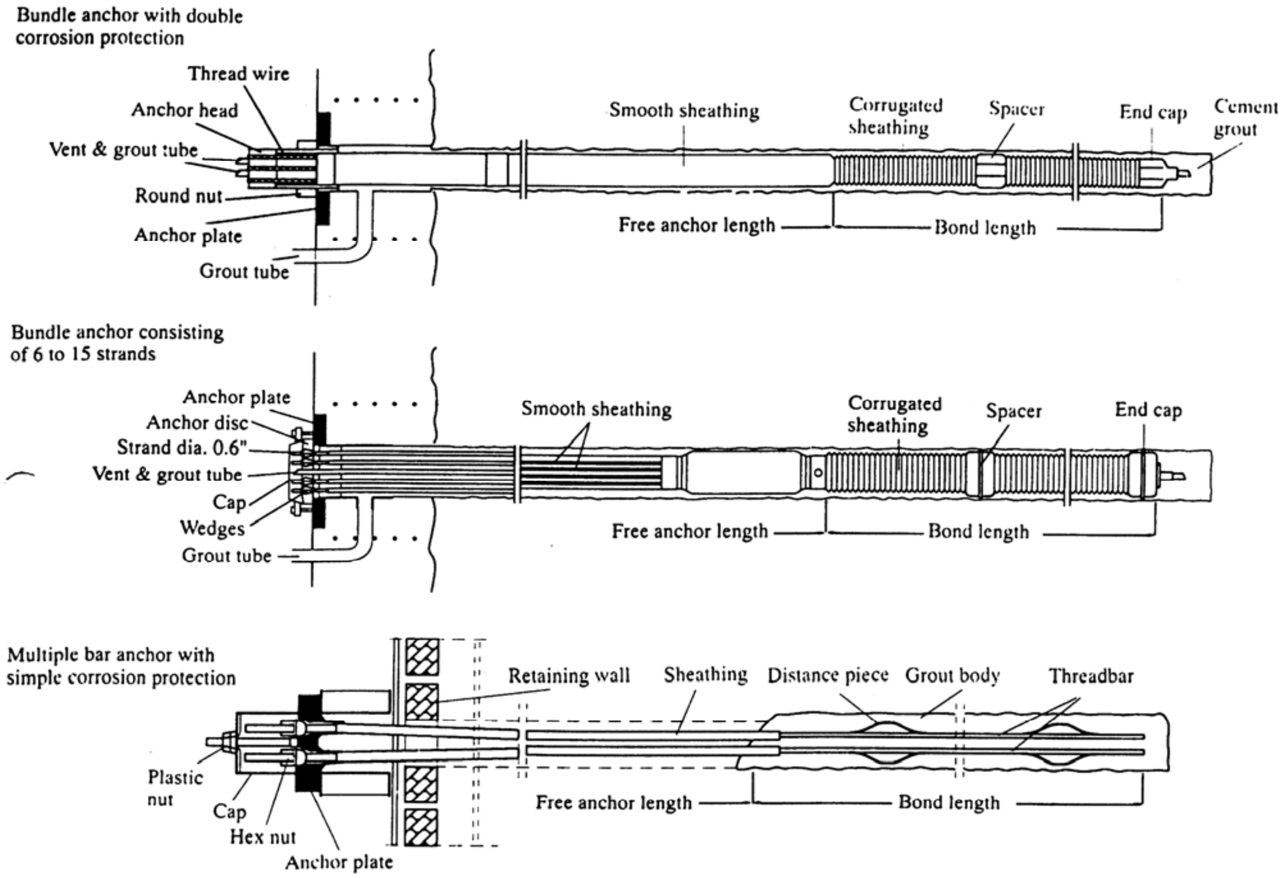


Fig. J-15. Different anchors and corrosion measures (after Hanna, 19??)

J.12 Soil Nailing

Soil nailing is the expression used for stabilizing soil or weak rock masses with a combination of bolts or anchors and wire mesh or steel mats, textile or any other material that keeps the soft material together (Fig. J-16).

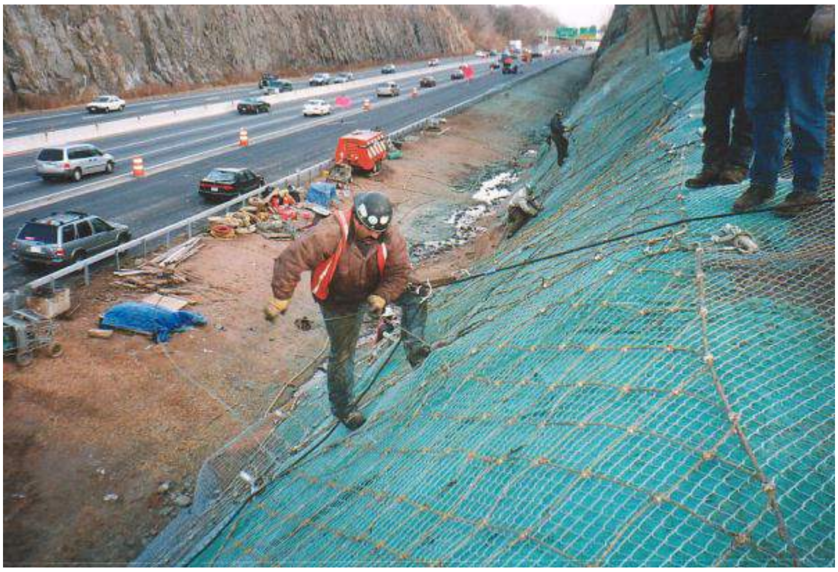


Fig. J-16. Soil nailing (photo: Vertec Contractors Inc., 2003)

J.13 Steel support

Steel support exists in two forms: flexible and fixed.

J.13.1 Flexible steel support

The flexible steel support, normally in the form of so-called ‘yielding steel arch sets’, are able to deform and therefore release the stresses in the rock mass. Bolts keeping the steel parts together can regulate the maximum (yielding-) strength of the arch set. The support is in particular suitable for environments were large movements and stresses are expected that cannot be stopped, mostly in mining. The stress on the steel arch should everywhere be about the same. If not bending and buckling of the arch occurs and the support fails (Fig. J-18). Small pieces of timber or another flexible material between the mass and the steel are used to equally load the steel. When the major part of the stress relief and deformations has occurred the arch sets can be covered with shotcrete or concrete. Obviously, the concrete or shotcrete covering should not be applied before because then the concrete or shotcrete will crack and fail.

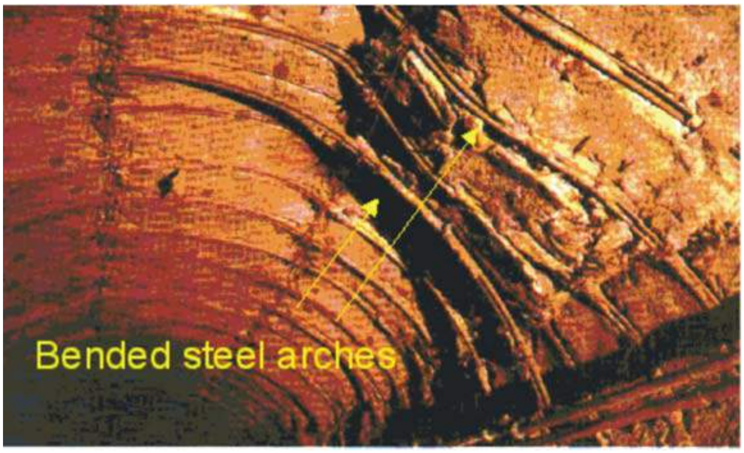


Fig. J-18. Bended steel arch sets. Arch sets are set in concrete. (after Geologija in geotehnika, Predor Karavanke Tunnel, Ljubljana, 1991)

J.13.2 Fixed steel

Fixed steel sets consist of H-beams. Fixed steel is seldom used alone (Fig. J-7, Fig. J-17and Fig. J-9e), but incorporated in shotcrete or concrete to give structural (tensile) strength. Fixed steel support is used where heavy loads on the support are expected and movements are not allowed. This would for example, be the case in shallow underground excavations in an urban environment.

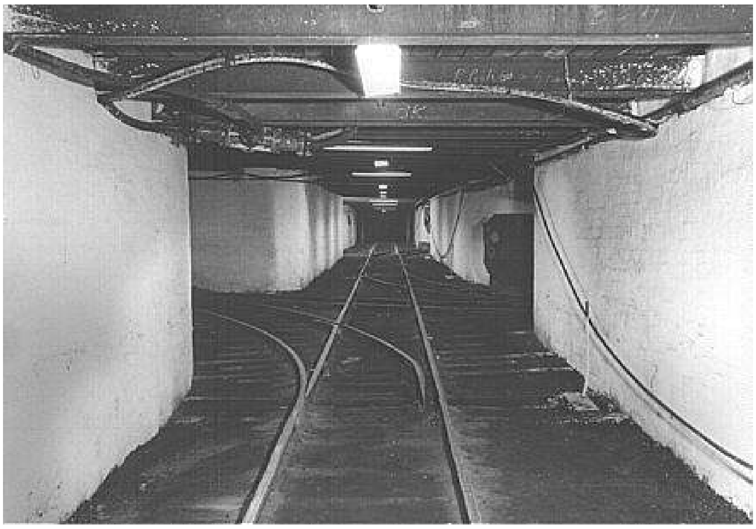


Fig. J-17. Fixed steel beam roof and concrete formed wall support (photo: Underground, Burwood Colliery; Lake Macquarie City Library, Australia)

J.14 Concrete support

Reinforced concrete support is used if large stresses are expected on the support and movements are not allowed. The concrete can be in the form of pre-fabricated plates and segments (Fig. J-9f and Fig. J-19) or formed concrete (by pouring behind a shuttering) (Fig. J-20, Fig. J-21, Fig. J-23 and Fig. J-24). The first will act as support immediately after installation whereas the second will only start working if the concrete starts to get its strength with a maximum of 28 days after pouring. It should be noted that in an irregular form of an excavation the space between the concrete segments and the groundmass have to be filled. Pumping grout or concrete behind the segments fills up the space, however, this grout or cement will need some time to obtain strength. Tunnel boring machines are often combined with an automatic installer for concrete segments and backfill, if such support is required.

J.15 Cut-and-cover

Cut-and-cover implies making a trench, the (concrete) tunnel segments are laid in the trench or built on location in the trench, and the trench is covered again with soil or rock. A variation is the making of deeps walls from surface or installing large size overlapping concrete piles as walls, excavating the ground between the deep walls or piles, installing the floor and roof of the tunnel, and backfill with soil or rock.

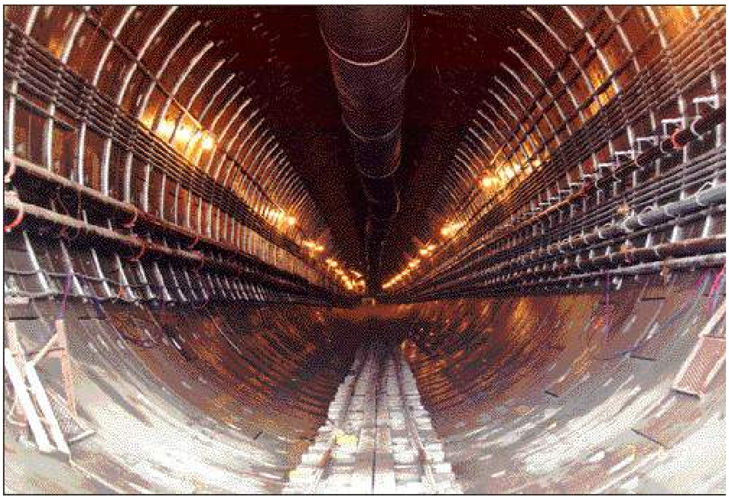


Fig. J-19. Pre-fabricated concrete support. The 10 m (32 ft) diameter EPB tunnel drilled by TBM was excavated with as little as 3.3 m of soft clay between the tunnel and the 21 m deep river bottom. The tunnel approached the river beneath a refinery site, with sensitive laboratory buildings and storage tanks directly over the tunnel. Forces and soil pressures were controlled with computer-monitored sensors. Expected settlements did not occur. (CN Rails's U.S-Canada Tunnel beneath the St. Clair River at Port Huron, MI, USA) (photo: Traylor)

Fig. J-20. Installing reinforcement for poured concrete support. Note the plastic sheeting for diverting water to the sites of the tunnel (Madeira, Portugal)

Fig. J-21. Tunnel portal. Note the heavy concrete reinforced and anchored retaining support above the tunnel (Madeira, Portugal)

J.16 Timber

Timber has been used extensively as support in the past; nowadays timber support in civil engineering is limited. Timber is used in the form of timber beams, or as blocks of wood in combination with steel or other support. Timber is very compressible and has a high bending possibility before it will break. This makes timber very suitable in locations were large displacements (and forces) are expected that cannot be stopped, for example, in mining. The compressibility makes that timber from a geotechnical point is a very poor form of support. The compressibility makes that the surrounding groundmass can move this causing loss of structure and reduction of shear strength along discontinuities. Another disadvantage is that timber rots in time and has to be replaced regularly. This allows relaxation of the groundmass and further movement and loss of strength. Preferably, timber should hence not be used in civil engineering. Measures exist to reduce the negative effects of compressibility, for example, with prestressing (Fig. J-22). A hazard of extensive timber support is that when rotting, it uses the oxygen in the excavation, which may cause expiration and can be very hazardous in poorly ventilated underground spaces.

Fig. J-22 Timber pack (An APOLLO modular pack installed with PACKSETTER Non-Weeping prestressing) (photo: MONDI TIMBER)

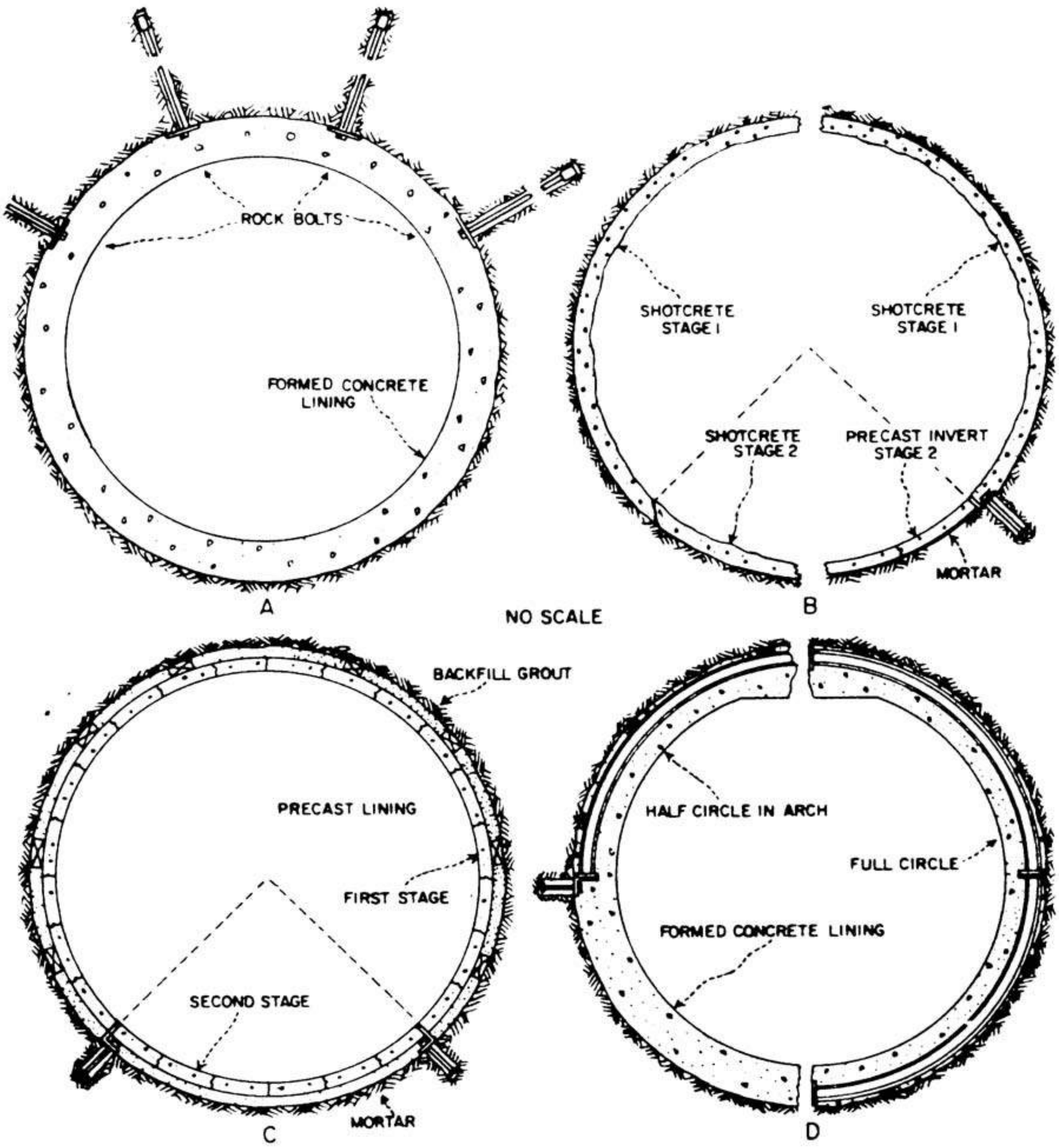


Fig. J-23. Various types of support for a circular tunnel

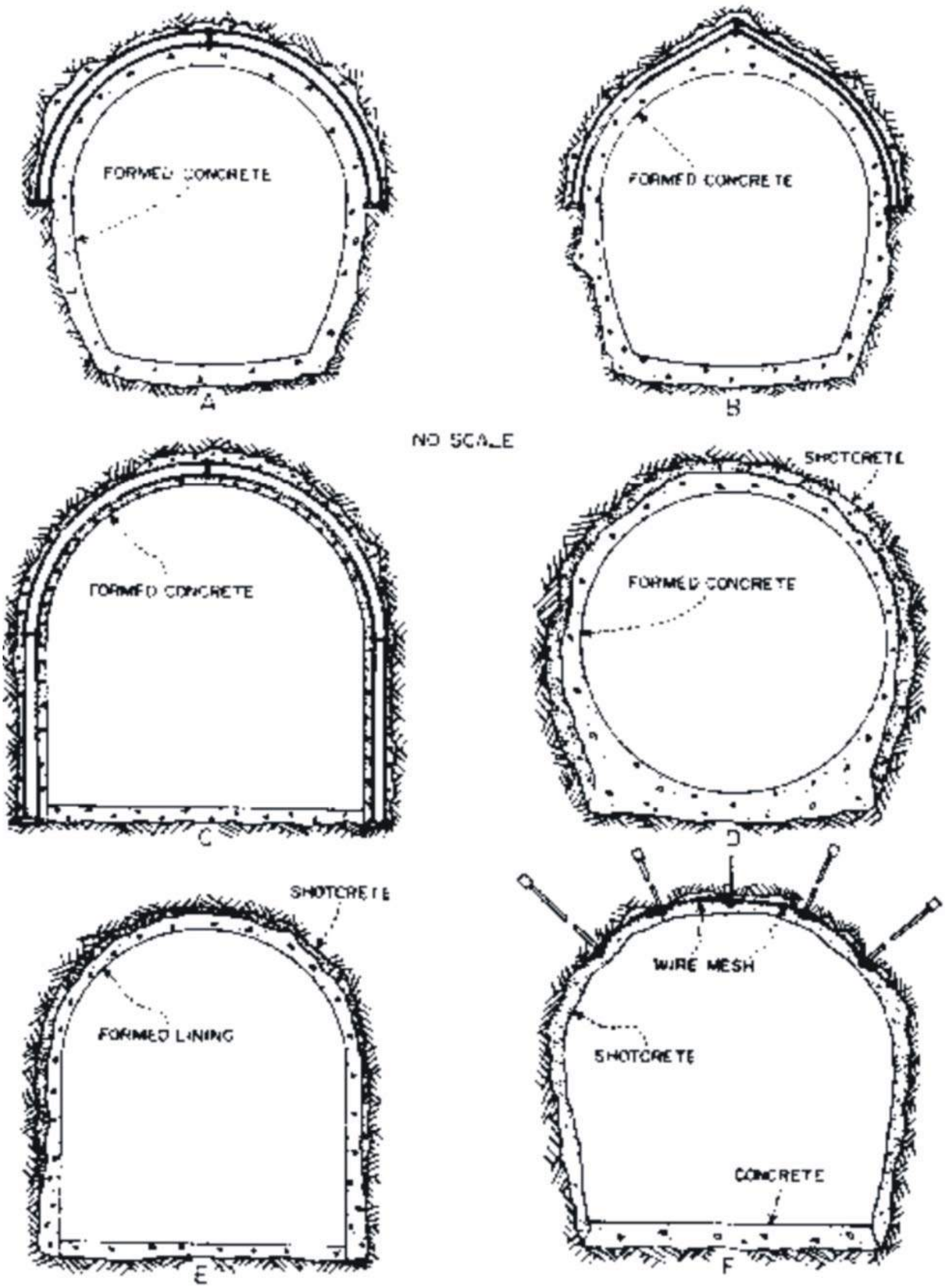


Fig. J-24. Formed concrete and shotcrete lining

K TUNNEL BORING MACHINE (TBM)

K.1 Introduction

Tunnel Boring Machines (TBMs)[18] are mechanized, more or less automatic machines that excavate the tunnel, if necessary provide support to the groundmass temporarily during excavation, and install permanent support (Fig. I-1top and Fig. K-1). The excavated material is transported out of the tunnel by pipes if watery slurry, conveyor belt, or trains. Support materials are brought into the tunnel by trains or by a rail system mounted along the roof of the tunnel. Gradients, horizontal and vertical, with which the tunnel can curve, are limited. TBMs excavate the ground by cutting and grinding, but excavator or ripper loading may also be used (the later are more commonly named a 'digger shield'). TBMs are often specially designed for one project in a particular type of ground. The various parts that may be included in a TBM, are:

1. ripper or excavator,
2. cutting wheel,
3. face plates, shield support, or hydro, pressure, or earth pressure balance shield,
4. grouting facilities,
5. steering assembly between cutting wheel and shield,
6. permanent support installation facilities, and
7. foam and grease injection installations.

In a good organized project, just one person may be able to handle and oversee the full operation. The operator can be located in the TBM, however this is not required. The author has once visited a site in Japan where the entire TBM operation for an 8 m diameter concrete segment supported tunnel was run by remote control from a nice sunny surface office by a tinny Japanese lady. Obviously, a better working environment makes it easier to find skilled employees and keeps the salaries low, hence, improving the economic viability. Disadvantages of TBMs are the relatively (very) high initial costs and the very costly consequences if the wrong type of TBM has been chosen. Many examples exist of TBM projects heavily delayed and going over budget because the subsurface conditions worked out to be different from those initially expected, even such that TBMs have been left in the ground and the already built tunnel abandoned. The choice of TBM becomes less and less critical because TBMs become increasingly versatile so that one TBM can coop with different subsurface conditions.

[18] Nuclear powered TBM

A very special form of a TBM may be the nuclear powered TBM, if it exists. Already for years, reasonably sustained rumors are around that the American forces developed a highly secret TBM (also known as Nuclear Subterrenes) that melts away the soil or rock mass by nuclear power. The machine is rumored to work with a high intensity nuclear source that is cooled by lithium. The lithium is pumped to the face, and melts and evaporates the soil or rock mass. After cooling, the walls of the tunnel consist of glass and do not need any further support. Another big advantage is supposed to be that no or little removal of material is required as the material surrounding the tunnel is denser compared to the original volume of soil or rock and part of the material evaporates. Similar rumors exist on a laser powered TBM. Obviously, if these types of TBMs exist it would be a major advance in tunneling as it solves two main topics in tunneling: support and removal of excavated material. However, it may also be a hoax, or it may just be ideas that never materialized or that did not survive a (small-scale) testing phase.

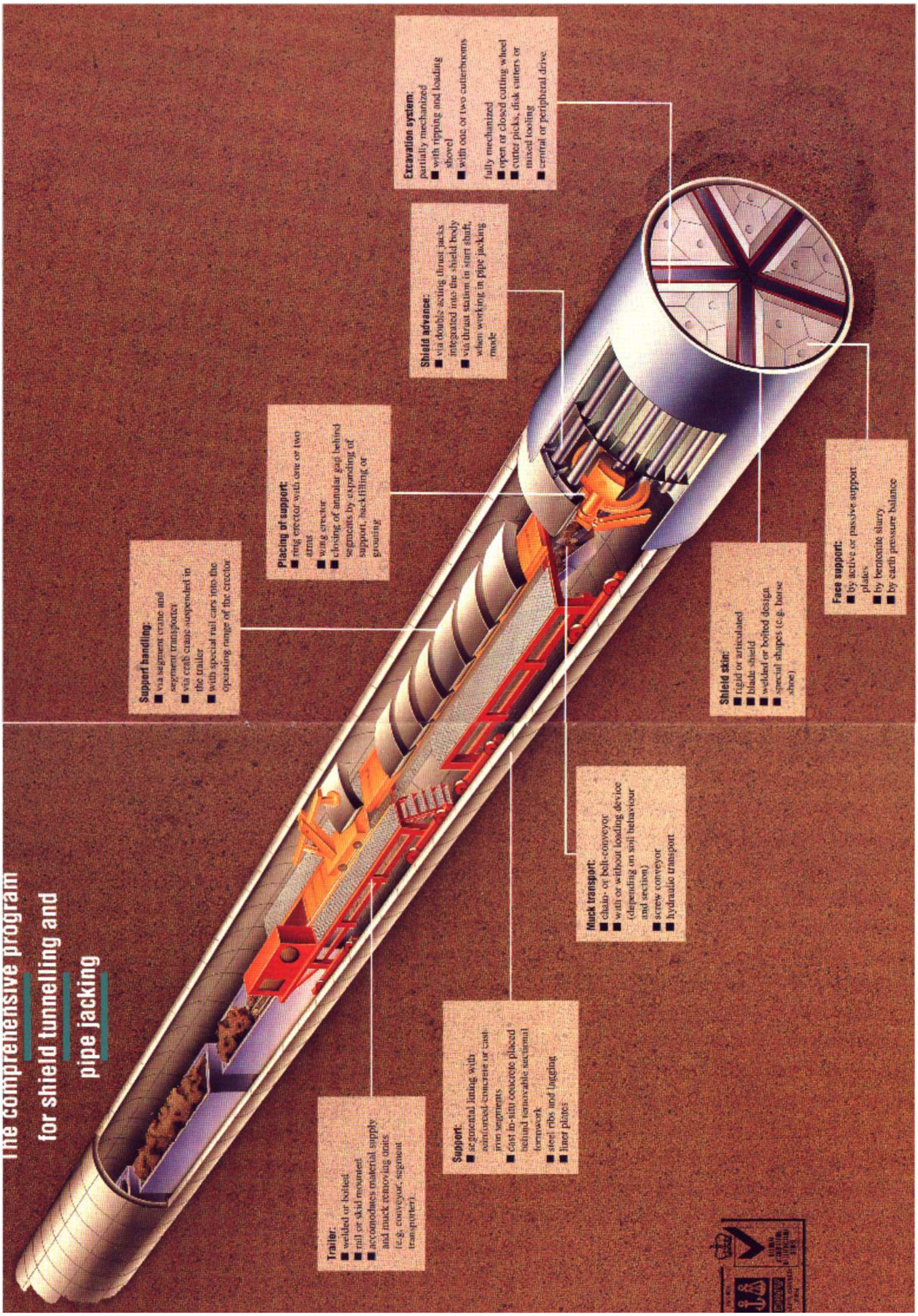


Fig. K-1. Tunnel Boring Machine (TBM) layout (drawing: Voest-Alpine Bergtechnik)

K.2 Dimensions

A TBM including support assembly installation may have a length up to 20 m or more. The start of a tunnel to be made with a TBM requires a considerable working space. Not only space needs to be provided for assembling the TBM but also an installation has to be made that gives a counter weight for the jacks to propel the TBM into the ground. Generally, a space is required double the length, width, and height of the TBM itself. TBMs can be dismantled into fairly small parts for transport, except for the main bearing of the cutting wheel which has a maximum dimension of about 2/3 of the diameter of the TBM. Transport roads or shafts should thus allow this size to pass.

Fig. K-2. Shield support with face plates (visible middle left turned into the machine) and ripper-loading device

K.3 Ripper-loading shovel

If the ground has no or little water influx a face support may not be necessary and excavation may take place by a ripper-loading shovel device (Fig. K-2). The shield gives the support direct after excavation and further in the TBM a facility for permanent support installation may be included.

K.4 Face plates, pressure, hydro, bentonite, or earth pressure balance shield (EPB)

If the groundmass in the face is not stable the face may sag and too much material may enter the TBM. Face plates (Fig. J-5and Fig. K-2) or a nearly closed cutting wheel may prevent this. If this is not enough, for example, if the mass easily squeezes or if water pressures are high, it is necessary to use a closed shield (installed vertically behind the cutting wheel, if present), in which only a relatively small opening allows the ground to pass into the TBM. The space between the shield and the face will then be under pressure, supports the face, and counteracts water pressures. The shield may be a pressure shield, bentonite shield, hydro shield (Fig. K-3), or earth pressure balance shield (EPB) depending on how the pressure is maintained. Buffer zones containing compressed air may be installed to improve regulation of the pressure on the face. The later is, in particular, important in soft soil masses in which a risk exists of blowouts, e.g. if the pressure becomes higher than the surrounding ground and water pressures.

K.5 Cutting wheel (also cutterhead)

A cutting wheel is required to excavate the ground in cohesive ground (cohesive soil or rock). The cutting wheel is adjusted to the type of ground to be excavated. In soft cohesive materials, such as clay, the cutting wheel is more a type of knife cutting slices of the ground. In harder ground such as rock, the cutting wheel has drag, disk, roller, or tri-cone bits (Fig. I-2,). The cutting wheel itself may act as face support if for a large part closed (Fig. K-4). Water jets may be mounted to ease excavation or to ease transport of the ground material. Various cutting wheels are presented in Fig. K-5, Fig. K-7, Fig. K-8, and Fig. K-6.

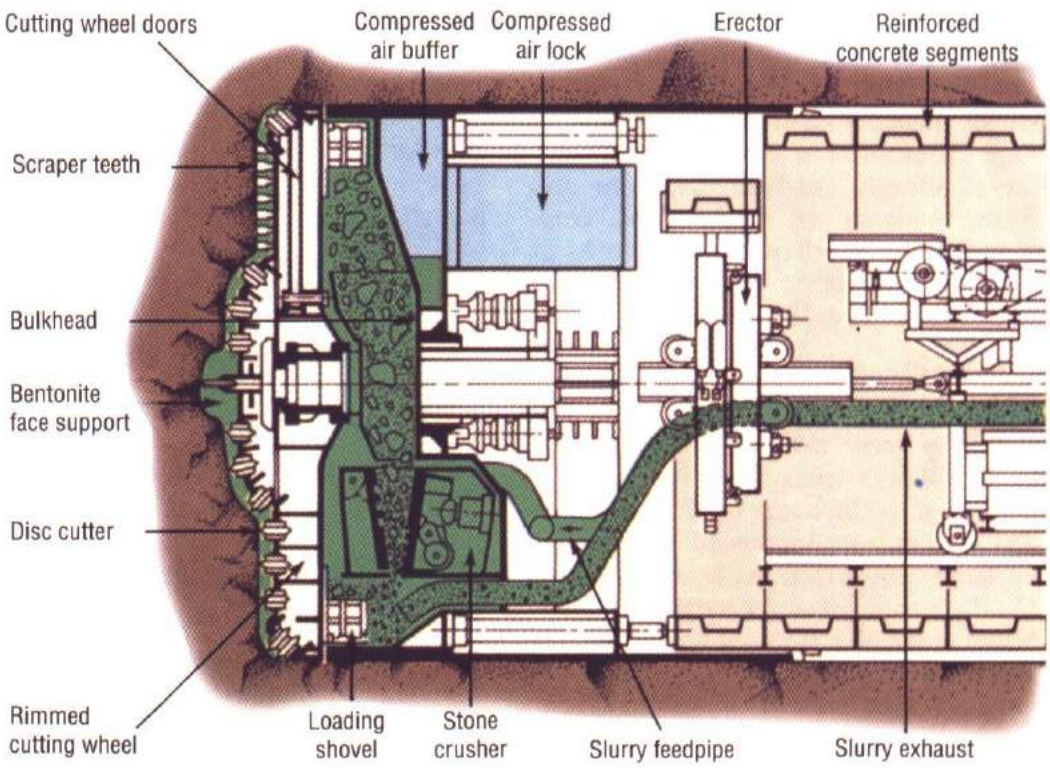


Fig. K-3. TBM with hydro shield (©Voest-Alpine Bergtechnik, Austria, 2000)

Fig. K-4. Cutting wheel for non-cohesive or low cohesive soft ground (soil) (after Voest-Alpine Bergtechnik, Austria)

Fig. K-5. TBM with shield used to drill through 413 MPa !! (60,000 psi) granite and a fractured and weathered surface formation, which required immediate support (Cowles Mountain Water Tunnel, San Diego, USA (photo: Traylor)

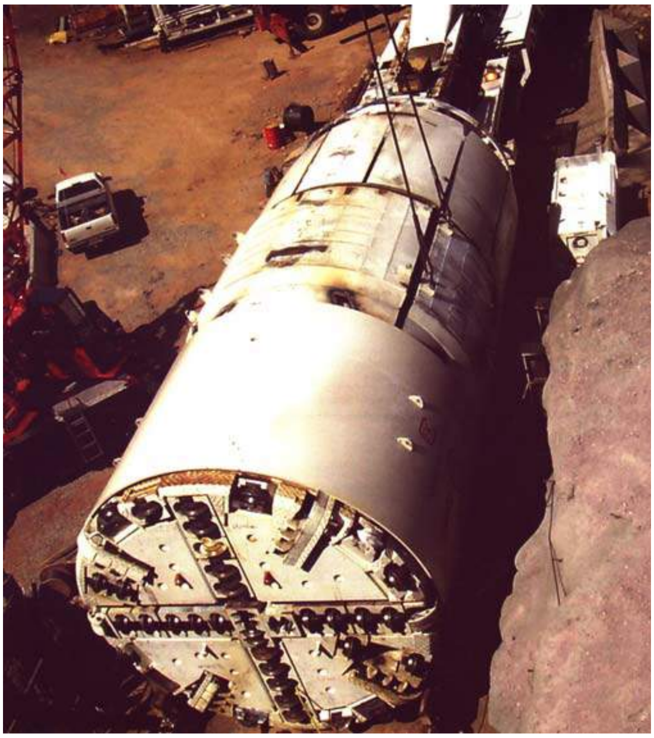


Fig. K-7. Rock tunnel TBM (2 m diameter) (Laboratory TBM at Excavation Engineering and Earth Mechanics Institute, EMI, Colorado School of Mines; photo International Mining & Minerals, UK, Oct. 2001)

Fig. K-8. Cutting wheel for relatively soft sediments. Diameter 6.6 m. Used for the underground railway in Rome. (after Voest-Alpine Bergtechnik, Austria)

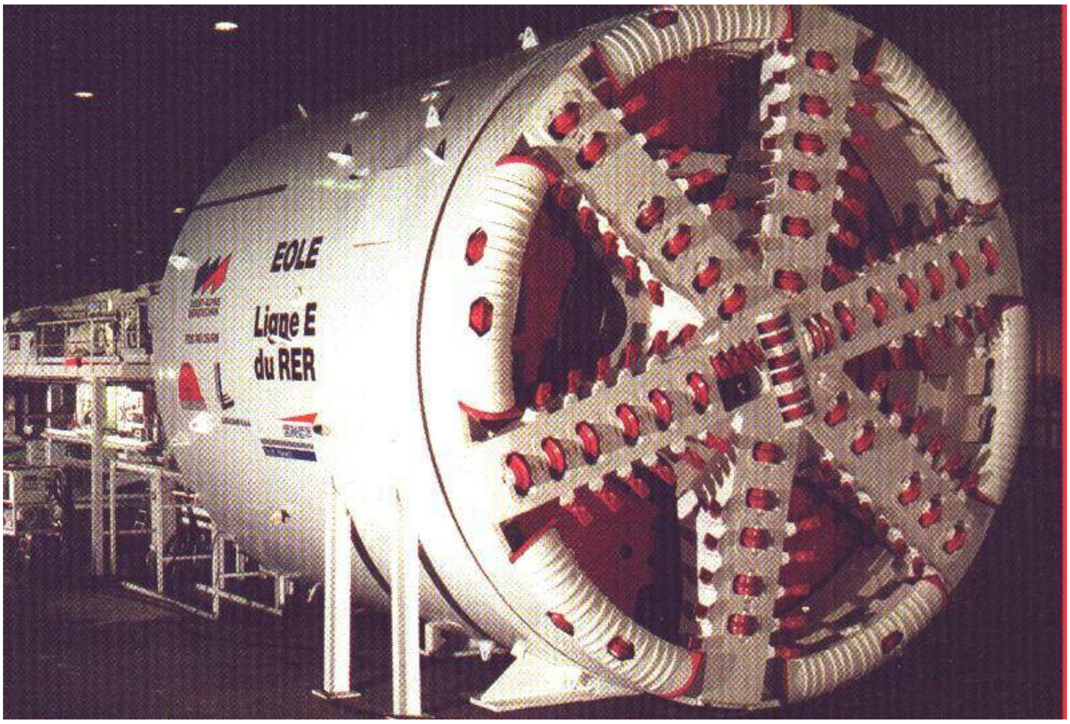


Fig. K-6. Rim-type cutting wheel with mixed disk cutters and drag bits. Diameter 7.4 m. Used for the connecting tunnel between two railway stations in Paris. (After Voest-Alpine Bergtechnik, Austria)

K.6 Jacking system

‘Jacked tunnel’

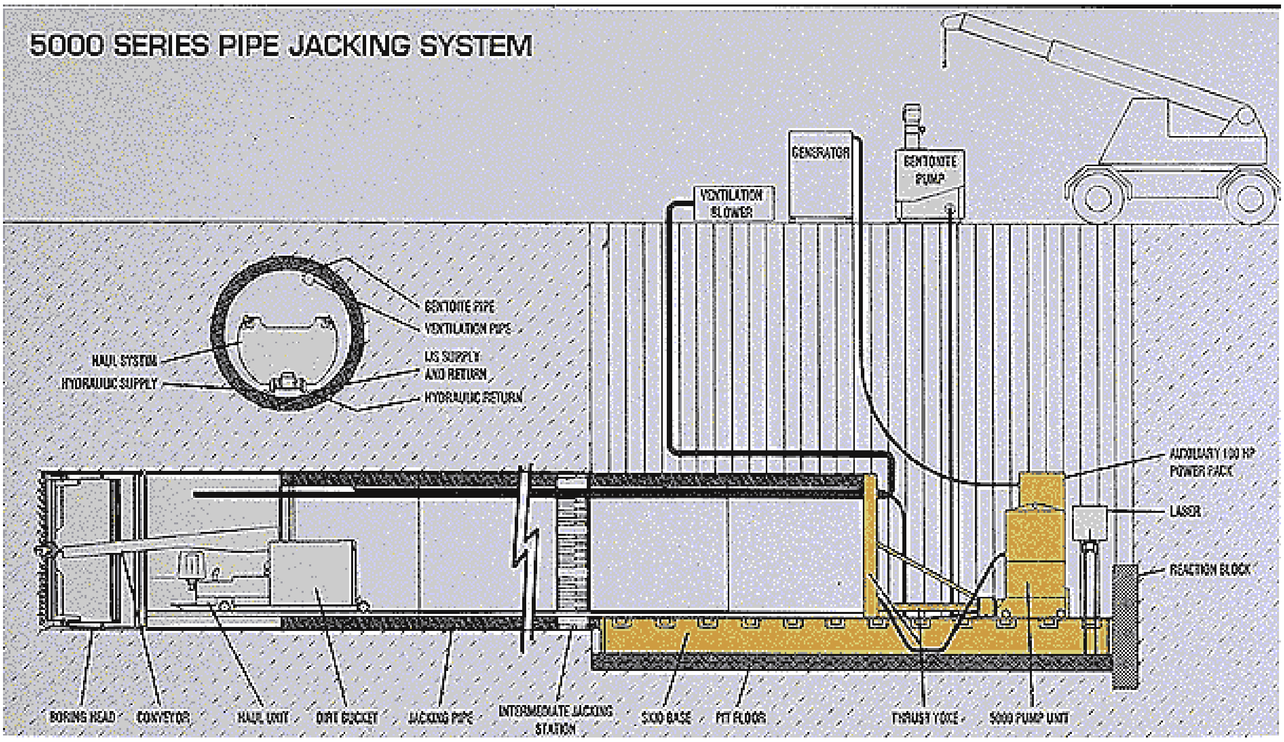


Fig. K-9. Pipe jacking set-up (©Akkerman Inc. 2002)

The propelling of the TBM through the ground is done by the trust on the TBM from a jacking system. The jacks can be positioned outside the tunnel between the (concrete) support rings and a heavy metal and concrete installation outside the tunnel that acts as counter weight (Fig. K-9). The jacks transfer the trust on the support and push the support lining with the TBM into the ground (‘jacked tunnel’). This system is only suitable for smaller diameter tunnels of relative small length as otherwise the friction between the ground and support lining becomes too large.

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

Jacks between cutterhead or shield and ground or support

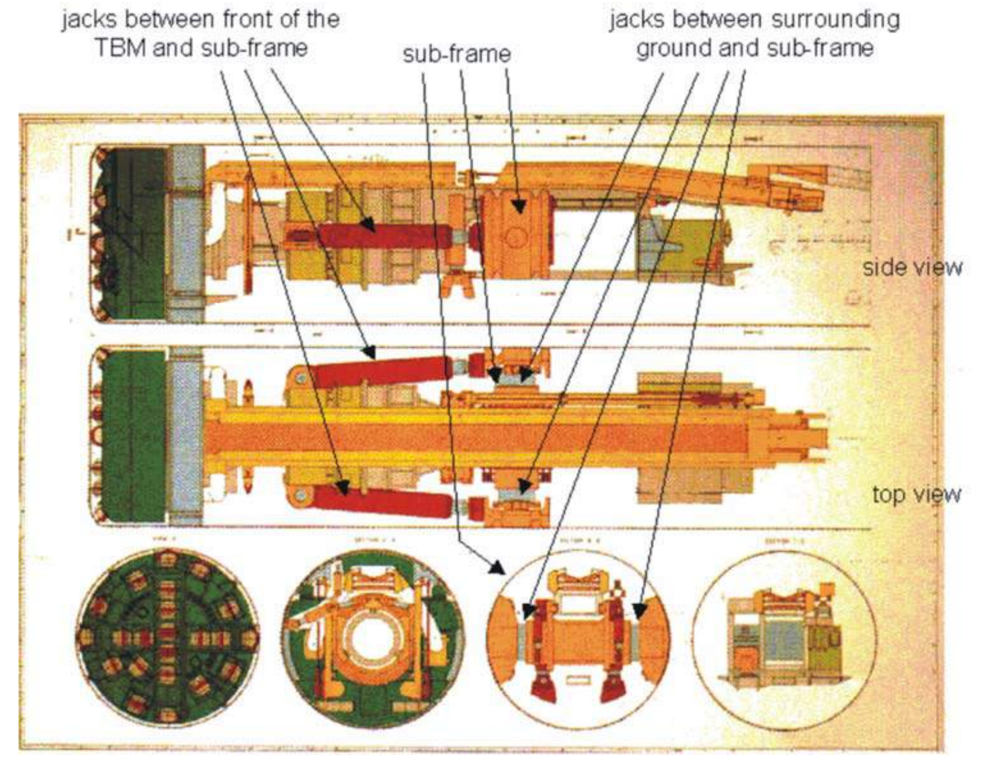


Fig. K-10. Jacking system for a TBM that moves itself by the trust against a sub-frame that is jacked against the surrounding ground (after Voest-Alpine Bergtechnik, Austria)

Alternatively, the jacks are mounted between the TBM cutterhead or shield and the support or if the groundmass is strong (e.g. rock) between the shield and the surrounding ground (Fig. K-10).

K.7 Grouting, crease, and foam installations

A grouting installation may be present to grout the backfill of a concrete segment lining (see chapter on support). Crease or foam installations may be used to reduce friction between shield and groundmass. Different types of foam can also be used to make the excavated ground easier to handle, or to increase the stability of the face.

K.8 Start installation at tunnel entrance

The start of a tunnel to be made with a tunnel-boring machine requires a considerable working space. Not only space needs to be provided for assembling the TBM but also an installation has to be made that gives a counter weight for the jacks. Generally, a space is required double the length, width and height of the TBM itself.

K.9 Stress around Tunnel Boring Machine

An opening excavated in the ground will deform. The deformation will generally be the largest in the direction of the maximum virgin stress field. For most excavation methods, this is no problem as long as the deformation does not inflict on the minimum dimensions required for the excavation. However, even relatively small deformations (in the order of cm's) may cause that a TBM get stuck, or that parts of the TBM will deform. In particular, deformation of the pressure shield may be a major problem as the sealing of the shield against the rock is reduced. If the stress field is strongly anisotropic, deformation may lead to deformation of the TBM, e.g. from circular to elliptical.

K.10 Production performance

Production of tunnel boring machines can be expressed in so-called ‘production monograms’ (example Fig. K-11). The figure shows a relative simple monogram in which rock masses are only classified in three classes soft, medium and very hard rock. More elaborate monograms may include discontinuity spacing or block size, orientation of discontinuities, etc.

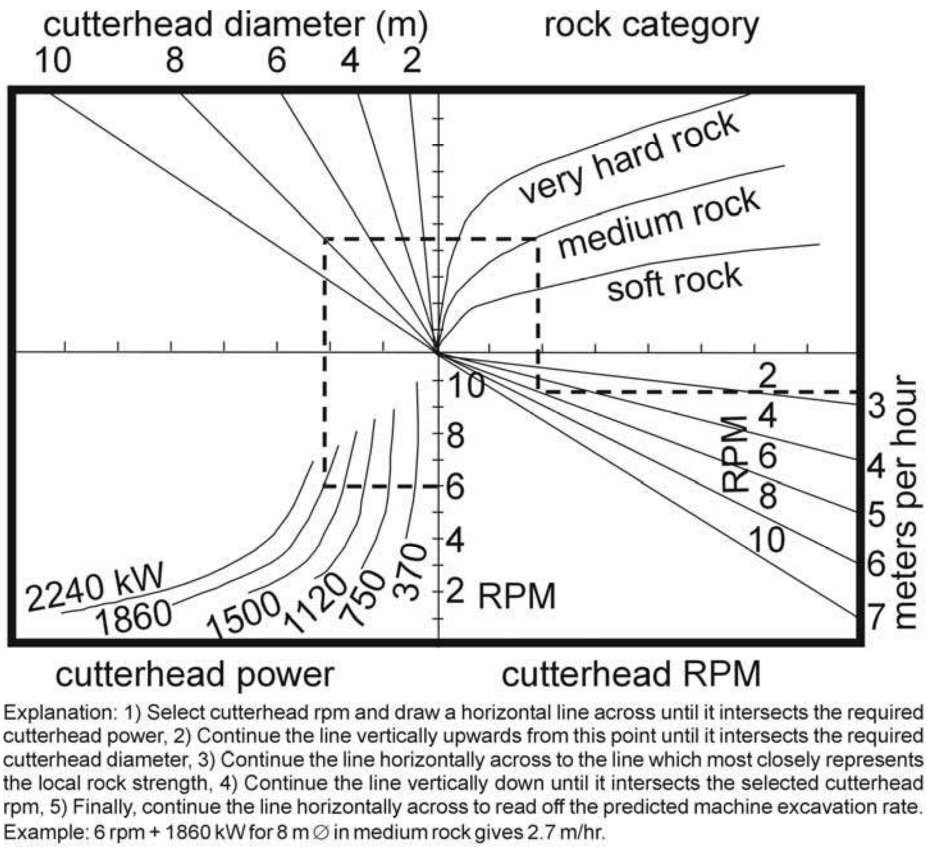


Fig. K-11. Production monogram (after Terratec) A graphical representation of the Rock Cutting Nomograph.

L MONITORING

L.1 Introduction

Monitoring is one of the most important aspects in surface and underground excavations whether in soil or rock. Monitoring gives an **early warning** for possible hazards as slope (Fig. L-1), dam or tunnel collapse or foundation failure, and it gives the option to **adjust the design** based on the data from the monitoring. Further, it is often requested by law, insurance companies, or by the client.

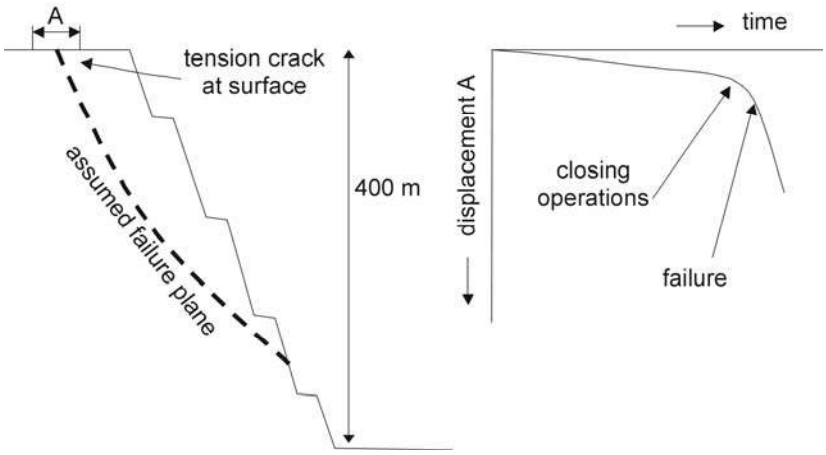


Fig. L-1. Monitoring for early warning in an open-pit mine. The movement in the slope was noted and monitored. When the displacement rate started to increase operations in the pit could be stopped in time to avoid loss of material or dangerous situations for the labor.

Virtually all monitoring in soil and rock mechanics is about measuring four features: the distance between two points, the angle between two points, the pressure, or the water level. Sometimes supplemented by temperature measurements to be able to compensate for temperature dependent extension and shrinking.

In the past (say for some 10's of years) only mechanical and fairly simple analogue electrical measuring devices where available. The digital revolution, reduction in size of measuring devices, for example, the nano-technology, and the enormous reduction in price of digital processing equipment has opened completely new fields of monitoring possibilities that are also economical to apply. Secondly, the digitalization has caused that where in the past most measurements had to be done manually at the location of the device; nowadays devices mostly measure automatically at a prefixed time interval and send data by cable or wireless to a remote processing station. This obviously increased the number of data enormously and the accuracy in time.

In this chapter, an overview is given of some of the techniques and methods (stress measurements in soil or rock masses have been discussed in chapter F.5.3). However, the detail is limited as full descriptions, in particular of the digital equipment, would be beyond the scope of these notes. For descriptions that are more detailed is referred to the literature.

L.2 Installation and protection

In principal, it is fairly simple to set out a monitoring program and install the devices. However, in practice many things tend to go wrong. A major problem is often due to the installation site. By nature, this will often be a site where heavy equipment and unskilled labor is around. Monitoring devices that are made to be accurate to micrometers or less, do not like it when a 10-ton truck runs over it. Similar a measuring peg to be assumed accurately fixed to 0.1 mm may not be as accurate if a laborer uses it to hang his coat. In public accessible places equipment tends to get stolen. Also in public non-accessible areas, for example, in a mine, it is not seldom found that the cable of a monitoring device has gone missing because a laborer needed a cable for his coffee kettle. Other problems are caused by vegetation and animals. In long time span measurement programs, growing trees and other vegetation may cause measuring devices to get displaced. Animals can even form a larger nuisance as many like to eat cables or excavate the measuring device for whatever reason. Obviously, an elephant stamping on an extenso meter does not help the accuracy either. Small animals and insects can also be a nuisance or even a hazard as bees and other stinging insects seem to think extensometer boreholes a perfect place for a nest if they can get in. In short measuring devices have to be installed idiot prove, out of way for heavy equipment and laborers, and well protected from any harm-doers. This makes setting up a decent monitoring program sometimes quit a hassle. Advisable is always to use a considerable surplus of devices to allow some to be lost or damaged without jeopardizing the monitoring program.

L.3 Installation and soil and rock characteristics

Apart from the problems detailed above, soil and rock masses may also inherently prove to be problematic for installation of monitoring devices. Any form of a fixed location on a soft soil or weak rock mass may prove very difficult to install because there is no firm base. Chemicals in groundwater or soil or rock may attack devices and cables (salts, gypsum, etc.). Measuring devices may be very sensitive to the location in a soil or rock mass with respect to discontinuities. If near to a discontinuity, a local displacement may be measured not representative for the whole mass. In particular, stress measurements are very sensitive for this (as already explained in chapter F.5.3). Hence, also with respect to the soil and rock mass characteristics, it is advisable and often just simply necessary to install a considerable surplus of devices to get an accurate and reliable monitoring result.

L.4 Mechanical and simple analogue electrical devices

- *Theodolites*
- *Pendulum and inverse pendulum (*)*
- *Inclinometers and tilt meters (*)*
- *Extensometer (*)*
- *Dilatometer*
- *Stress measurements (see chapter F.5.3)*
- *Load cell*
- *Tremor sensors (geophones)*
- *Piezo meters*
- *Time domain reflectometry (TDR) (*)*

Those with an * are more specified in the following chapters.

Other tools that may be used for monitoring are: positioning systems (satellite position systems, GPS), laser distance and angle measurement (including Lidar), satellite imagery, radar images, photo and pattern deformation imagery, electrical/digital miniature deformation measurement cells, and all sorts of geophysical methods. These are not discussed as these are beyond the scope of these notes.

L.4.1 Pendulum

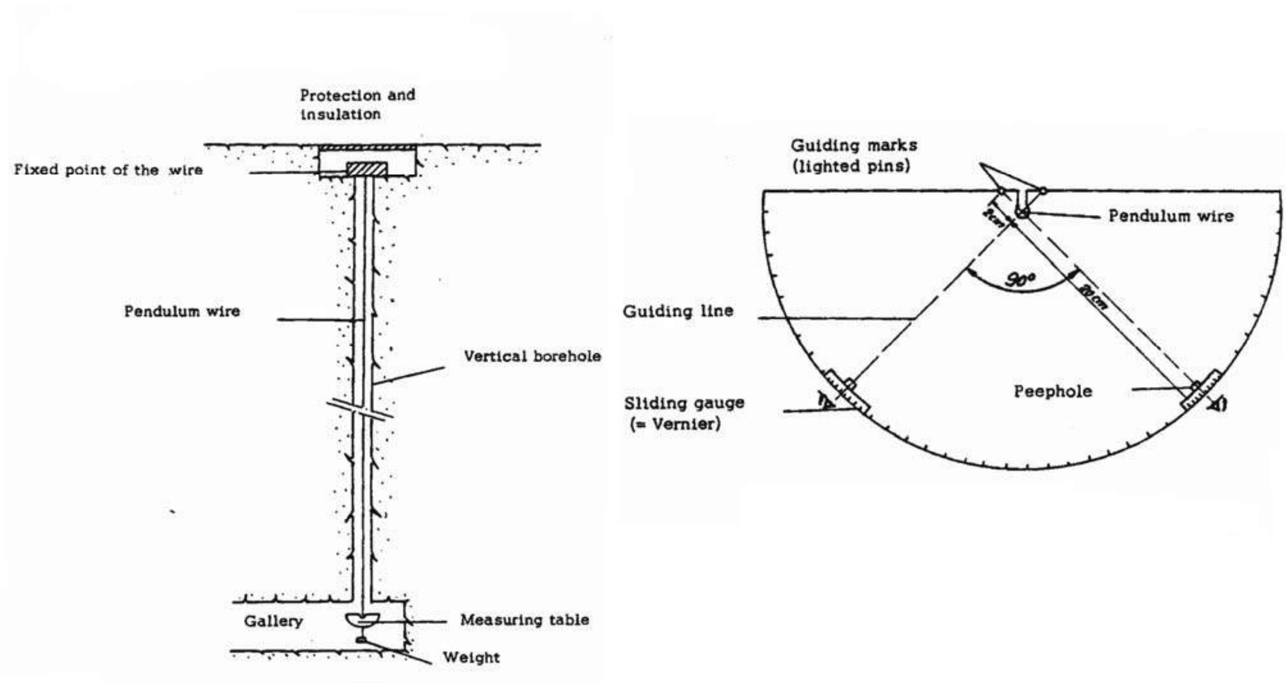


Fig. L-2. (Normal) Pendulum (after ??)

A **(normal) pendulum** is a free-hanging wire fixed at the top and with a weight at the end, e.g. a plumb line. Tilt or displacements of a structure or soil or rock mass will show as displacement of the end of the wire against a fixed measuring table (Fig. L-2). It should be noted that a (normal) pendulum requires access to the top and bottom of the wire. Mostly pendulums are mounted in a vertical borehole in a soil or rock mass, or in a vertical shaft in a structure and end up in a tunnel. In the past, the measuring of the displacement of the wire was done manually at the measuring table. Temporarily an optical device was mounted on the measuring table that measured the position of the wire with micrometers of accuracy. Nowadays this is replaced by laser or magnetic measurement devices that measure with high accuracy and send the data by cable or wireless to a remote station.

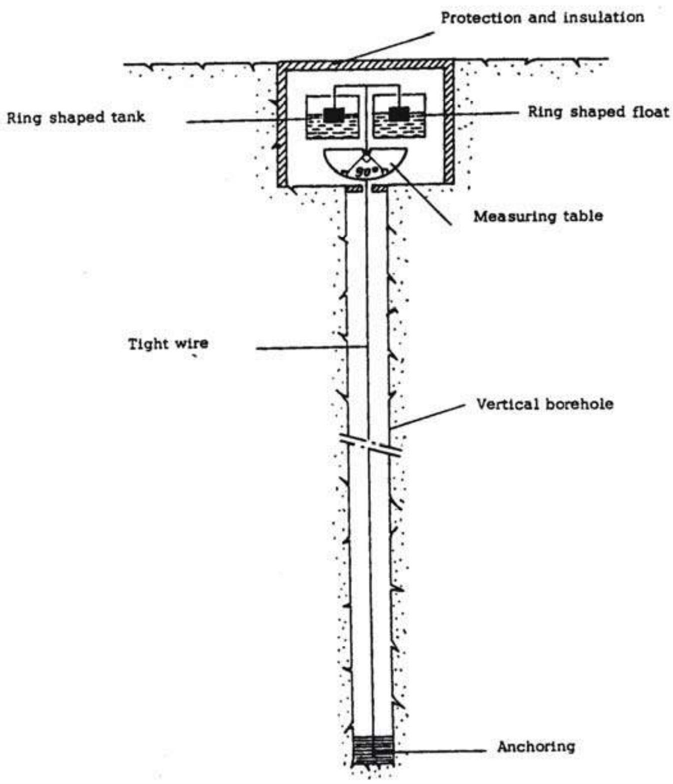


Fig. L-3. Inverse pendulum (after ??)

An **inverse pendulum** consists of a wire that is fixed at the bottom and a floating device at the top (Fig. L-3). The fluid, mostly oil or water, on which the floating device floats creates a stress in the wire, and, hence, keeps the wire straight (if the fluid level is high enough). An inverse pendulum does not need access to the bottom of the wire. Measuring table and measuring procedure are similar to the normal pendulum.

L.4.2 Inclinometer and tilt meters

Fig. L-4. Inclinometer (photo: Soil Instruments Ltd., 2003)

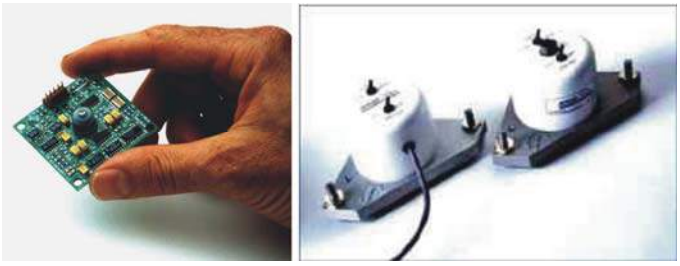


Fig. L-5. Tilt meter (photos: Applied Geomechanics Inc., 2003)

Inclinometers (and tilt meters) measure the angle against the vertical. In modern versions, the direction of tilt is also measured. The equipment is normally very small and can be mounted in a borehole probe (Fig. L-4) to measure the orientation of a borehole, or can be mounted on a rock or soil mass or structure (Fig. L-5). The accuracy can be up to 0.01°. Newer versions digitize the signal and send the data by cable or wireless to a remote processing station.

L.4.3 Distance meters

Distance meters measure the length between two points on surface or in an underground excavation (Fig. L-6). Pegs are fixed in the soil or rock mass, or structure and special tape is used to measure the distance between the pegs at regular time-intervals. The tape is expensive because the length has to be temperature independent. The special device will control the stress on the tape so that with a prefixed stress is measured to keep the elastic extension of the tape the same for each measurement. More and more the tape is replaced by laser equipment.

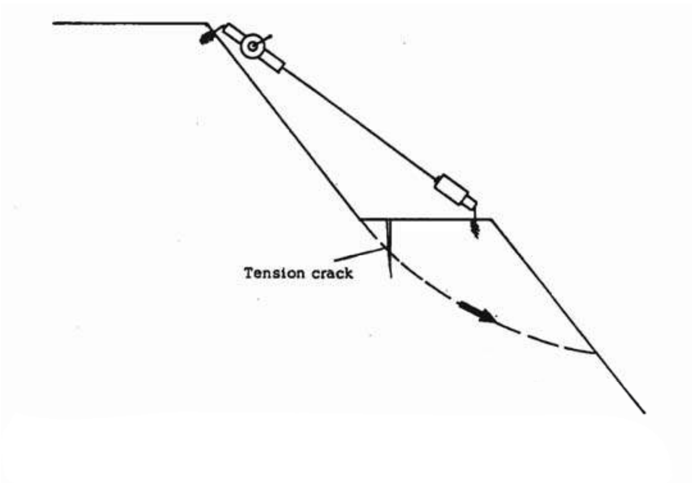


Fig. L-6. Distance meter along slope with tension crack

L.4.4 Extenso meters

Extenso meters measure the length between two points. The extensometer consists of a wire or steel rod fixed on one site and free to move at the other site (Fig. L-7 and Fig. L-8). These are mostly mounted in boreholes.

L.4.5 Piezo meter

Piezo meters measure the pore water pressure (Fig. L-9 and Fig. L-10). The device consists of a porous pot that contains water. Via the porous pot the water pressure inside the pot is equal to the pore water pressure surrounding the pot. In the pot is a pressure measuring device that (nowadays) electronically measured the water pressure and sends the data by cable to a remote station. It should be noted that in a low permeability soil or rock mass the time after installation before the water pressures outside and inside the pot are equalized may take a long time. The smaller the size of the pot the shorter this time span. Special devices as additional water pipes to add water to the pot may be installed to reduce the time span before equalizing, but this makes the piezo meter more expensive.

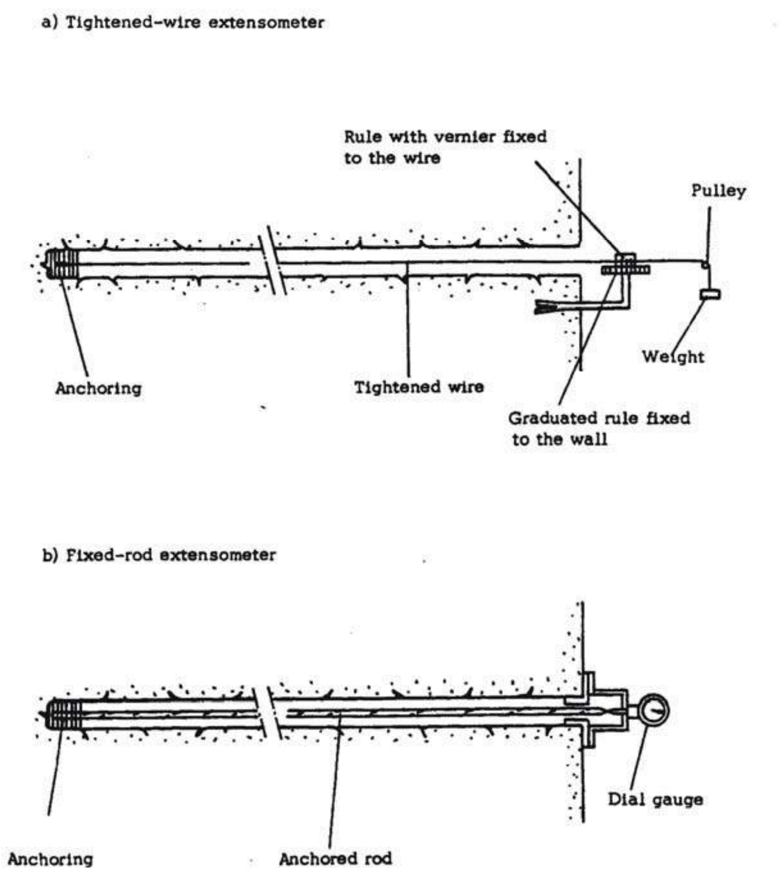
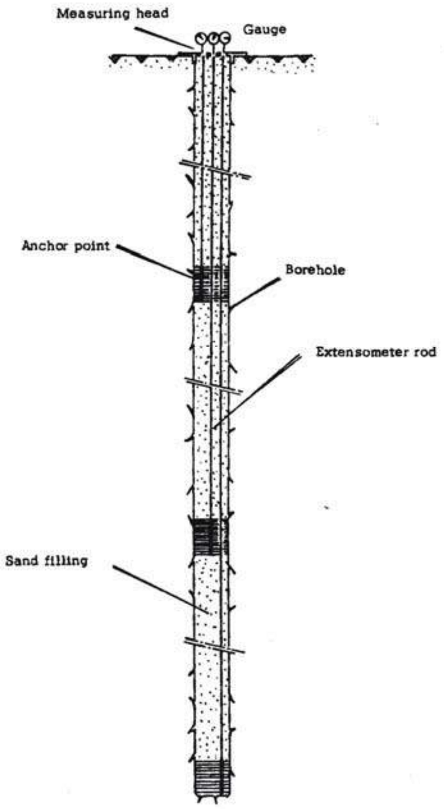
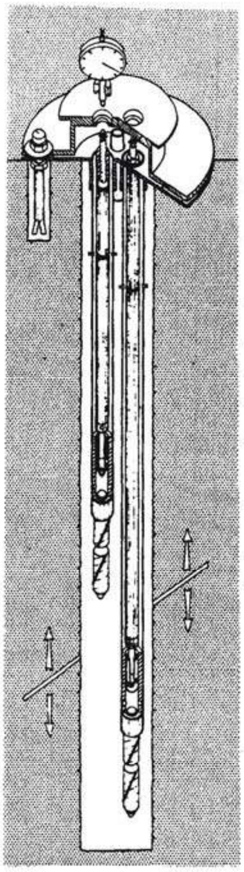


Fig. L-7. Extenso meter (after ??)





- 8 -

APPLICATION:

The multiple-point rod extensometer is the most important instrument for the performance of precision measurements of length changes (movement in the direction of the borehole axis). They can be installed in rock, in concrete, and in brickwork. A special field of their application is the supervision of faults or thrusts in earthquake areas.

The extensometers are conceived in a meccano principle. Their maximum length is 200 m. They may be installed in boreholes of any direction between horizontal and vertical. A built-in calibration device allows to control their functioning even after years. When the measurements have been finished, most parts of the instrument can be retrieved and re-used. Sensitivity is 0.01 to 0.1 mm, depending on extensometer length.

TECHNICAL DATA:

Length: up to 200 m, or more under special conditions

Sensitivity: 0.01 to 0.1 mm, depending on length

Range: for mechanical and electrical reading ± 10 mm, resettable

Types: Single extensometer (one fix point) for minimum borehole diameter 25 mm; multiple extensometer (up to eight fix points) for minimum borehole 60 mm

Direction of installation: any direction between horizontal and vertical, upward and downward inclined

Special purpose design: with movable temperature sensor

Reading:

1 - manual reading device
2 - registration with mechanical recorder
3 - electrical remote reading and recording
4 - limit switch for alarm device

DESCRIPTION:

1 - ribbed steel anchor, fixed by mortar injection
2 - anchor cone with measuring and calibrating notch for functional control
3 - extension rods in plastic protective tube, with lock mandril at anchor end, and with sensor bar at head end
4 - extensometer head with rod guides and rock bolts for fixation at borehole mouth
5 - sensor bar and dial gauge bar; recess for placing dial gauge or electrical sensor
6 - dial gauge with dial gauge bar
7 - electrical sensor, screwed in

Fig. L-8. Multiple extenso meter (after Interfels)

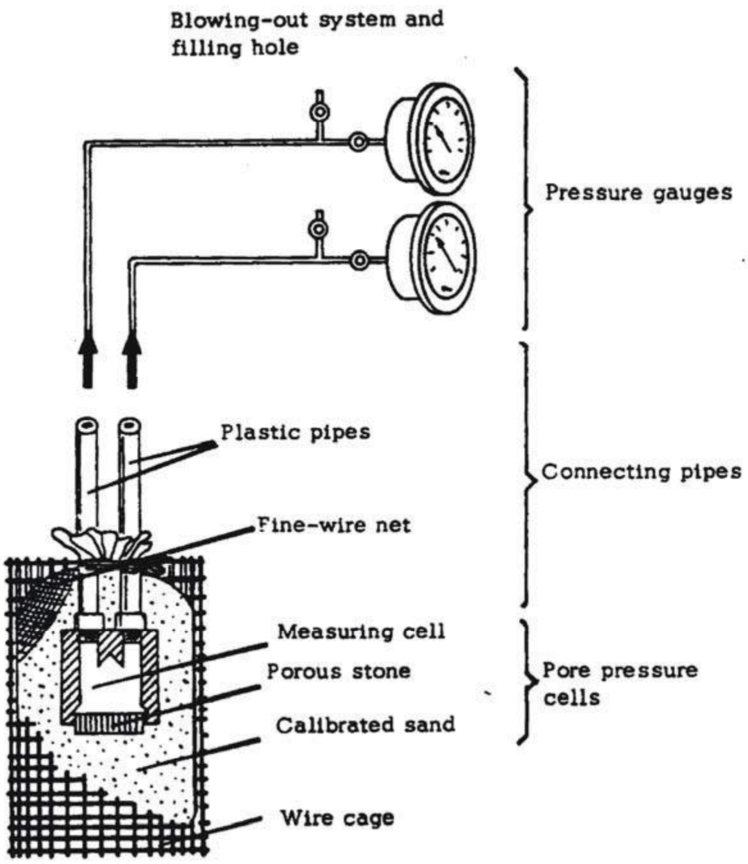


Fig. L-9. Piezo meter

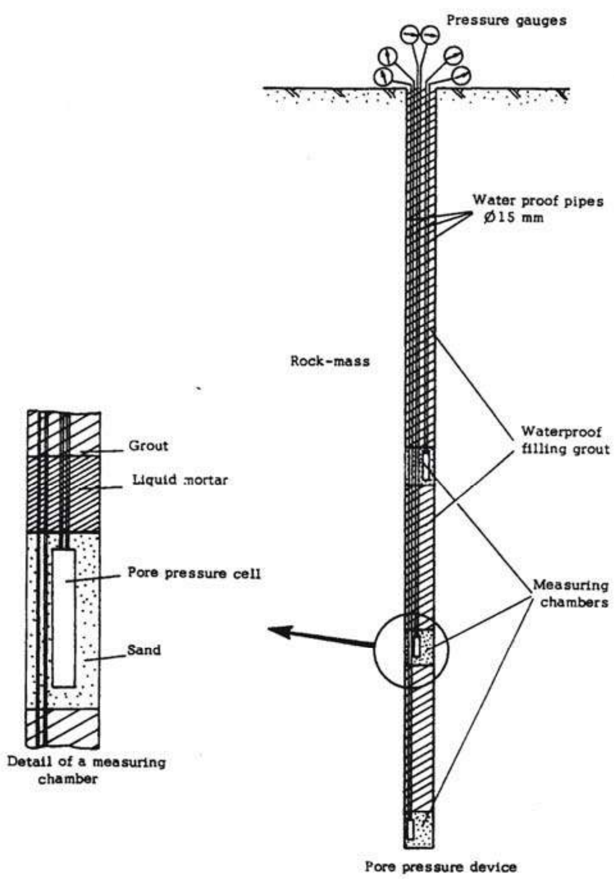


Fig. L-10. Multiple piezo meters in a borehole

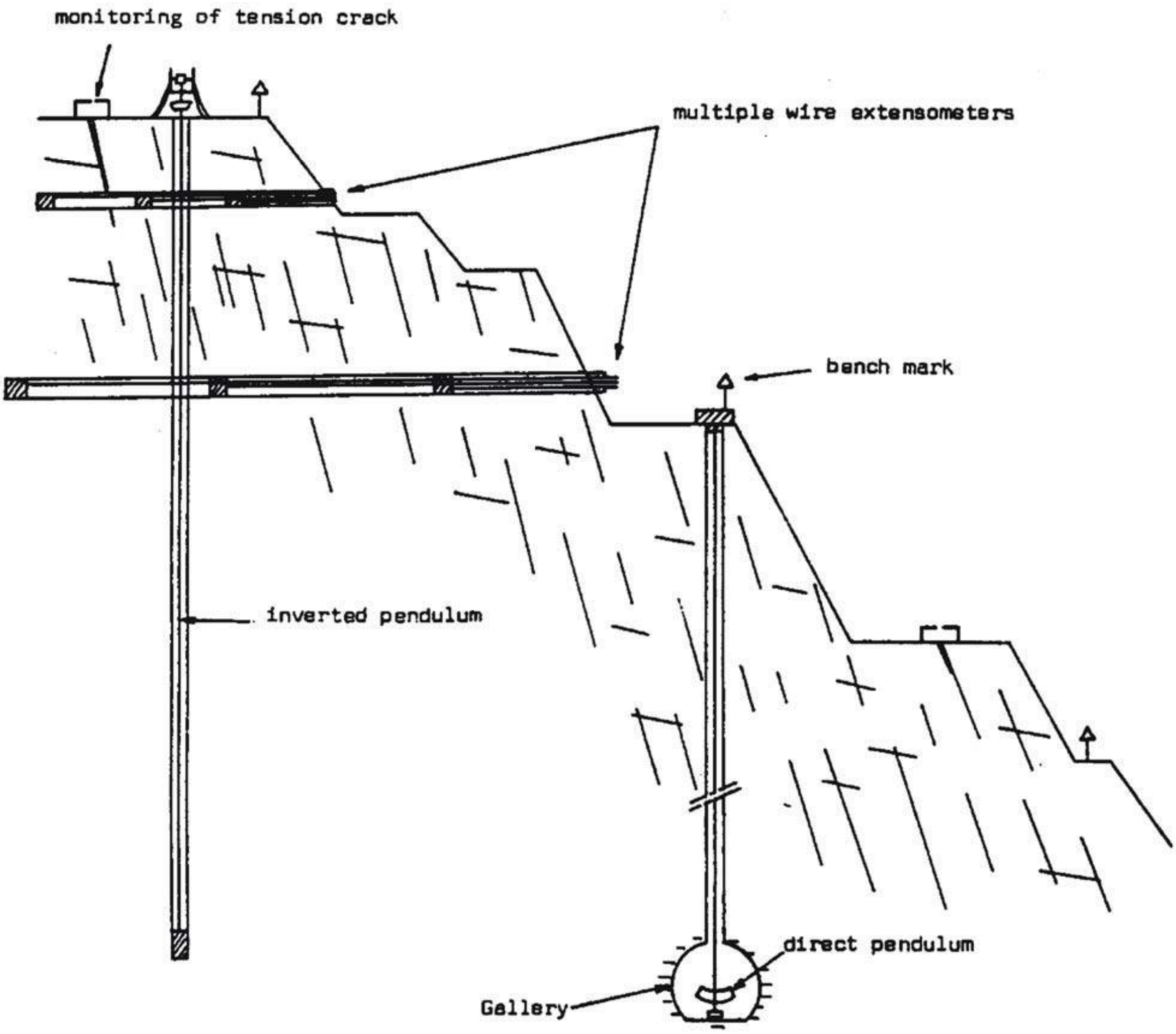


Fig. L-11. Various monitoring tools on a slope

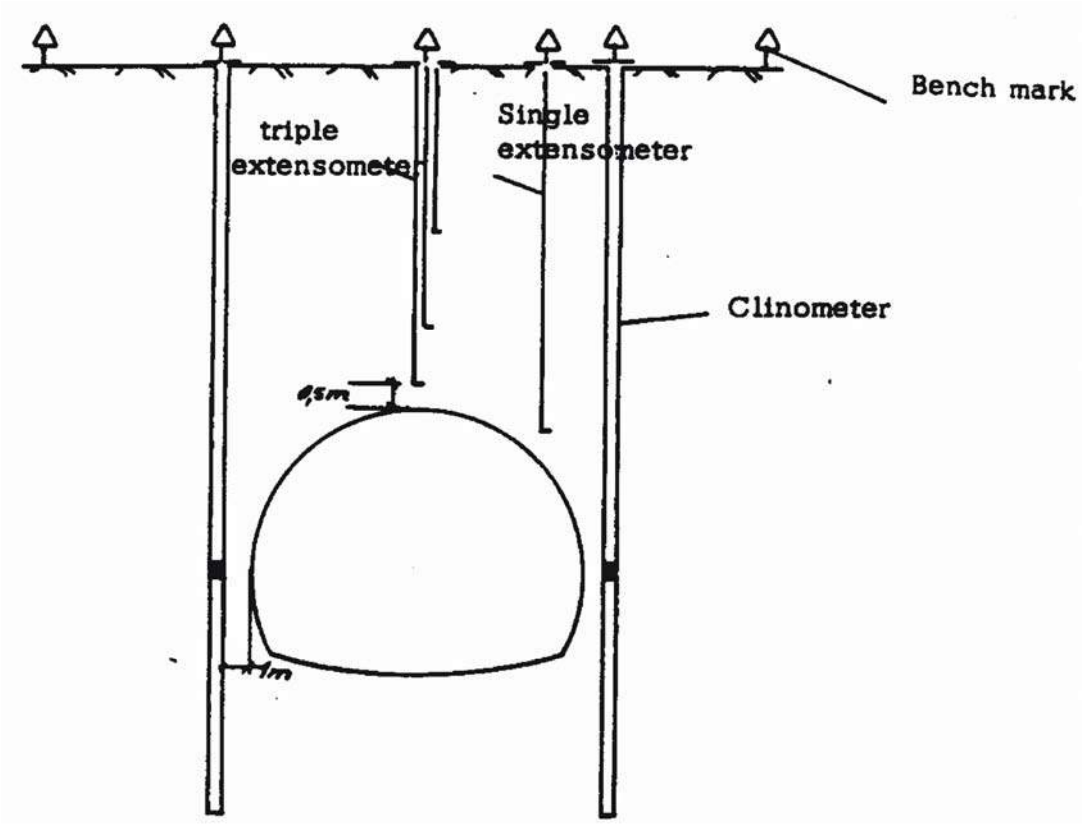


Fig. L-12. Various monitoring tools for a tunnel

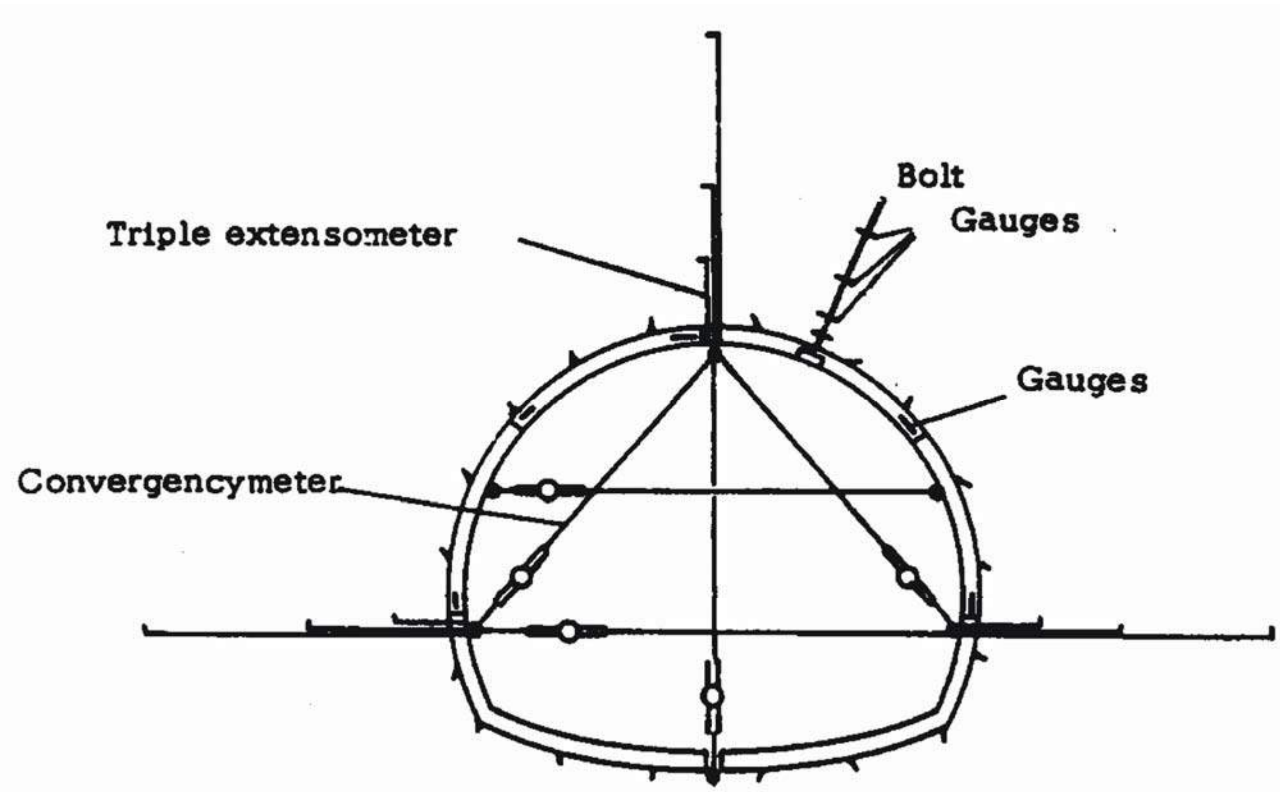


Fig. L-13. Various monitoring tools in a tunnel

L.4.6 Time domain reflectometry (TDR)

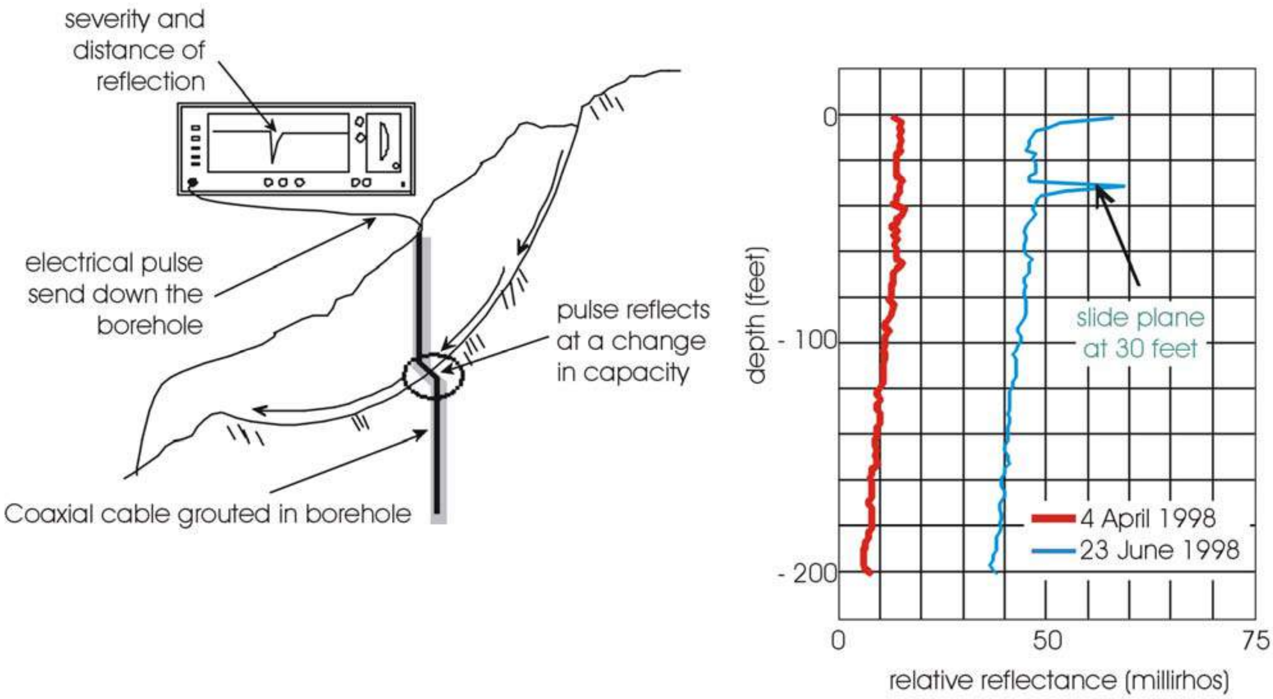


Fig. L-14. Time domain reflection (TDR) (modified after Kane, 2000)

Time domain reflectometry (TDR) is a method of locating the depth to a shear plane or zone. TDR uses an electronic voltage pulse that is reflected like radar from a damaged location in a coaxial cable[19]. To monitor movement, coaxial cables are grouted in boreholes and interrogated using a cable tester. Characteristic cable signatures can be stored and compared over time for changes, indicating movement. Advantages of using TDR over conventional servo-accelerometer probes include relatively low cost, time savings, immediate determination of movement, and the capability to monitor cables remotely using data loggers and telemetry. (Dowding, 2002, Kane, 2000). Fig. L-14 shows TDR applied to a slope. Between 4 April and 23 June the slope has moved and the peak in the reflection signal shows the bending of the borehole at the depth of the sliding plane. It is important to note that the method only works if a considerable bending of the cable occurs or if the cable breaks or shears. A gradual change due to tilting will not be measured. For this inclinometers are better options.

[19] Normal is to record the reflection in units of 'millirhos' which equals 1/1000 of the original voltage pulse.

M EXPLORATION SEISMICS IN DISCONTINUOUS ROCK MASSES

(In part overprint from Geophysics for slope stability, 2002, Robert Hack)

M.1 Introduction

Many geophysical tools exist to investigate discontinuous rock masses and to establish material inhomogeneities, boundaries, and properties of materials. Most methods exist for many years, but certainly, in the last decades most methods have undergone a large change due to the availability of cheap computer power. As a result, geophysical surveys are easier to do by non-specialists, interpretations are more reliable and more accurate, and finally yet importantly, are considerably cheaper. Still regrettably, geophysical methods are relatively seldom used and often only boreholes or soundings are made to investigate the sub-surface.

Boundaries between different materials can be obtained by all geophysical methods. It is, however, to be noted that the boundary measured is often not based on a difference in properties (a ‘contrast’) that is mechanically interesting. In, for example, electromagnetic methods the properties measured are the dielectric constant and/or conductivity of the materials. If a change in either of these coincides with a boundary in mechanical properties, the found boundary is of interest for rock mechanics. However, if the boundary measured is only a local enrichment of the slope material with, for example, manganese or iron, the boundary is of none or of little interest.

M.2 Seismic methods

Seismic methods are based on the measuring of an elastic wave (also: seismic, shockwave, or acoustic wave) traveling through the sub-surface. The wave is reflected or refracted on boundaries characterized by different densities and/or deformation properties. Seismic methods can nearly always be used to determine the internal structure of materials in a rock mass. Sometimes logistics and practical problems as how and where geophones and sources can be placed, may make the method impractical. Refraction seismic studies have been the standard tool for geotechnical work for years. However, state-of-the-art computerized seismographs for use in geotechnical work handle 24 or more channels each connected to one geophone and, hence, measure the signal of many geophones in one round. This reduces the quantity of sources necessary, but more important, has opened the option to do seismic reflection surveys in geotechnical work with relatively low costs. A problem often encountered is the frequency content of the source signal. For many rock mechanics related investigations, it is important to investigate the structure with a high resolution. This means that high frequencies should be available in the seismic signal.

M.3 Type of waves

Seismic waves may be compression (P-waves), shear (S-waves), or surface waves, such as Raleigh and ground waves. Different properties of the soil or rock mass material have different influences on the behavior of the wave depending on the type of wave. Traditionally compression (P-) waves were used, as these are easy to generate. Compression waves are most sensitive to changes in the normal (compression) stiffness of the materials while shear and surface waves are more influenced by the shear stiffness of the materials through which the wave passes. Shear stiffness is, however, often of more interest than normal stiffness because shear stiffness can often be related to the shear strength of sub-surface materials (Helbig and Mesdag, 1982). Sources for shear waves have been cumbersome, but new devices have been developed recently (Ghose et al., 1996, Peeters et al. 1998). It should be noted that shear waves only exist in a medium that contains a shear stiffness, e.g. these cannot pass through open or water-filled discontinuities. Velocities measured in a rock mass with open or water fluid discontinuities will be of a ray path around the discontinuity (see also anisotropy).

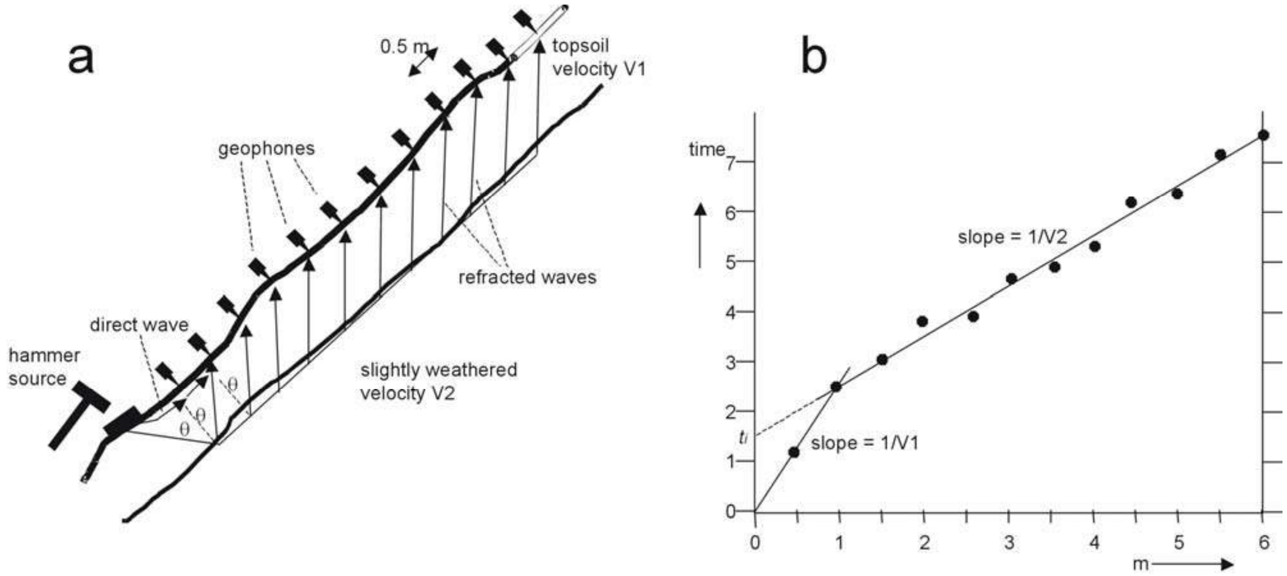


Fig. M-1. Seismic refraction survey for a boundary parallel to surface. a) ray paths, b) travel time versus distance for the first arrived signal

M.4 High-resolution seismic sources

Traditionally, sources in seismic surveys for geotechnical work consist of a hammer blow on a metal plate. The contact between plate and soil or rock, often via a weathered topsoil layer, does not allow the introduction of a seismic signal into the ground with a high-energy content in high frequencies. Explosives, and in particular, fast burning explosives, such as, caps or fuses, give a far better ('spiked') signal with more energy in the high frequency components of the signal. Obviously, the use of explosives is often forbidden or otherwise problematic. Alternatively, sources that emit a controlled signal ('vibro-seis') into the ground can be applied (Ghose et al., 1996, 1998). The energy level per unit of time of these sources is considerably less than the energy per unit of time released by explosive sources, but the controlled signal can be correlated far better with the received signals and allows for an increase in the noise/signal ratios of the received signals. In addition, all energy is concentrated in the required high frequencies and no energy is lost in low frequencies that are not of interest.

Apart from applying a source with high frequencies in the signal, also the geophone and line spacing influences the resolution. It makes not much sense to obtain a resolution in very high detail in depth and have a very low resolution in the directions over the plane of the measurements. It should be noted that time and costs increase dramatically if the resolutions have to be high in all directions.

M.5 Seismic refraction

Seismic refraction is based on the first arrival of a signal that travels through a layer with a higher velocity. Table M-1 gives some characteristic seismic P-wave velocities for sub-surface materials. The method has been standard used for years (Stötzner, 1974, Telford, 1990, Williams & Pratt, 1996). Fig. M-1 shows a simple situation of a more-or-less regular topsoil layer on top of a slightly weathered rock mass on a slope. The first arriving signal at the geophones is for the first two geophones the direct signal traveling through the topsoil layer and for geophones 3 to 12 the refracted wave traveling through the slightly weathered rock mass. For the refracted waves applies that the angle of incidence and angle of refraction equal θ. The topsoil has velocity $V1$ and the slightly weathered rock mass velocity $V2$. The angle θ is given by:

$$\sin(\theta) = \frac{V1}{V2} \qquad \text{[M-1]}$$

The thickness of the residual soil layer can be calculated easily from figure 1b and equation [M-1].

Table M-1. Characteristic P and S wave velocity ranges (P-wave velocities modified after Anon, 1995b, S-wave velocities after Hack))

material	P-wave velocity	S-wave velocity	material	P-wave velocity	S-wave velocity
	m/s			m/s	
air	360	0	weathered igneous and metamorphic rock	450 - 3700	10 - 60% of P-wave velocity provided that discontinuities are filled with material that contains a shear stiffness
water	1500	0			
dry sand	400 - 1000	10 – 60% of P-wave velocity	weathered sedimentary rock	300 – 3000	
			metamorphic rock	1000 – 6000	
clay	300 – 1800		unweathered basalt	1000 – 4300	
			limestone	500 – 6700	
Note that the material descriptions are crude and do not account for variations in, for example, water content, number of discontinuities, or whether discontinuities are open, filled, or closed, etc. These factors influence the velocity values far more than most of the material constituents. In particular, S-waves are very sensitive because these do not pass through open or water filled discontinuities.					

$$\text{depth} = \frac{1}{2} * \frac{V1 * t_i}{\cos(\arcsin(V1/V2))} \qquad \text{[M-2]}$$

$$t_i = \text{intercept time (see graph figure 1b)}$$

For an inclined plane boundary, the survey should be repeated with the source position at the other end of the geophone spread (Fig. M-2). If the inclination between boundary and surface is relatively small, the velocities and depth become:

$$\frac{1}{V2} \approx \frac{1}{2}\left(\frac{1}{Vdown} + \frac{1}{Vup}\right)$$

$$ti_{down} = \frac{2z_{down}}{V1}\cos(\theta) \quad ti_{up} = \frac{2z_{up}}{V1}\cos(\theta) \qquad \text{[M-3]}$$

$$z_{down}, z_{up} \text{ are the depths below down-dip respectivily up-dip source point}$$

Non-computerized interpretation of refraction seismic studies is easy for simple 2 and 3-layer situations with plane boundaries, but becomes very difficult and inaccurate if the boundaries are irregular or if the number of layers increases. State-of-the-art computerized interpretation techniques include programs based on wave front methods (Sandmeier, 2000, Telford, 1990, Tomo, 2000). In these programs for each arrived signal, a hypothetical path through the sub-surface is constructed. The travel times through each layer and the veloci-

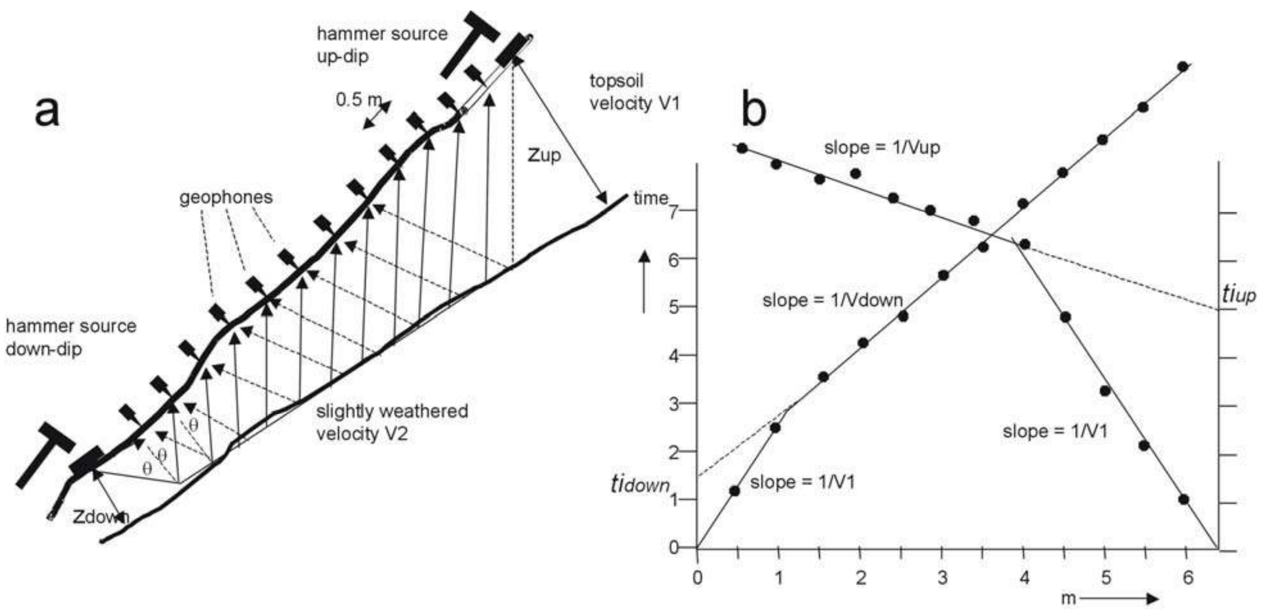


Fig. M-2. Seismic refraction survey for interface inclined to surface. a) ray paths, b) travel time versus distance for the first arrived signal

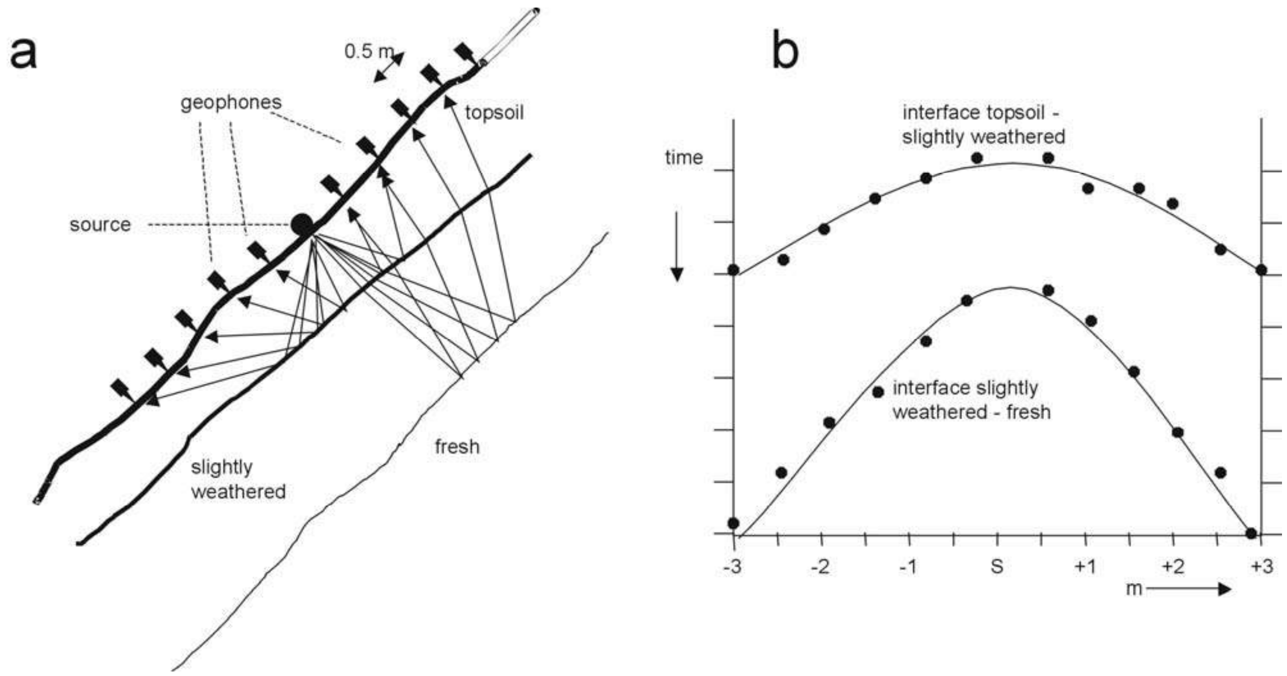


Fig. M-3. Seismic reflection survey. a) ray paths, b) travel time versus distance for the arrival of the reflected signals from the two reflectors

ties of the layers are optimized to the arrived signal by an algorithm in the computer program, and the hypothetical travel paths are adjusted. This is repeated until a best fit of the data on the travel times is obtained. The best travel-times/velocity section is then converted to a depth section. Interpretation of irregular boundaries and multi-layer situations are usually no problem with these programs. The resolution that can be obtained is dependent on the frequency content of the source signal and the spacing of the geophones.

M.6 Seismic reflection

Traditionally the equipment and the computers to analyze the measured signals in reflection seismic surveys were too expensive to be used in geotechnical work. In recent years, the availability of cheap and powerful computers allows the introduction of reflection seismic in geotechnical work (Bruno et al., 1998, Kurahashi et al., 1998). Reflection seismic has a similar set up of source and geophones as for refraction seismic surveys, however, not only the first-arrivals are considered, but also the complete received signals are incorporated in the interpretation (Fig. M-3).

Reflection occurs on an interface where the deformation characteristics of soil or rock mass are different on both sides of the interface. This is governed by the so-called 'acoustic impedance' (Z), which equals the product of density and seismic velocity. The 'impedance contrast' (δ) is the ratio of the acoustic impedances on both sides of the interface:

$$\text{acoustic contrast} = \delta = \frac{Z_1}{Z_2} = \frac{density_1 * velocity_1}{density_2 * velocity_2} \quad \text{[M-4]}$$

The acoustic impedance and the angle of incidence of the incidence seismic wave govern the energy in the transmitted and reflected waves. The relations between reflected and transmitted energy contents are complicated, but the reflection coefficient (R) is simple for normal incidence (90°) of the incidence seismic wave:

$$\text{reflection coefficient} = R = \left(\frac{\delta - 1}{\delta + 1}\right)^2 \quad \text{[M-5]}$$

Table M-2 gives values for velocities, densities, acoustic contrast, and reflection coefficients for some material boundaries. The reflection coefficient is independent whether the seismic wave passes from medium one to medium two or vice versa. Note the very high reflection at weathering horizons, interfaces with water, and the virtual 100 % reflection on interfaces with air. The higher the reflection coefficient the more energy is reflected and the more easily the reflector boundary is detected with reflection seismic surveys.

Table M-2. Characteristic P-wave velocities, densities, acoustic contrasts, and reflection coefficients for some soil and rock mass materials)

material	first medium		second medium		acoustic contrast	reflection coefficient
	P-wave velocity	density	P-wave velocity	density	δ = Z1/ Z2	R
	m/s	kg/m3	m/s	kg/m3		
fresh sandstone to fresh limestone	2000	2400	3000	2400	0.67	0.040
fresh limestone to fresh sandstone	3000	2400	2000	2400	1.50	0.040
highly weathered sandstone to slightly weathered sandstone	500	2200	1200	2400	0.38	0.20
fresh sandstone to open discontinuity (air)	2000	2400	360	1.2	11111	0.999
fresh sandstone to water	2000	2400	1500	1000	3.2	0.27
clay to slightly weathered sandstone	400	1500	1200	2400	0.21	0.43
granitic residual soil to fresh granite	600	2000	3500	2500	0.14	0.57
dense sand to slightly weathered limestone	1000	1800	2500	2400	0.3	0.29

Processing the received signals is extensive. Commonly used processing techniques in geotechnical work are 'Common Depth Point' gathering (CDP), and digital filtering. Computerized processing programs are available for use in the seismograph or can be run in laptop computers.

M.7 Seismic tomography

Seismic tomography works on the bases of a series of geophones and source positions on the surface or in boreholes. Fig. M-4a gives an example of a seismic tomography survey between two boreholes. Geophone and source are lowered in two different boreholes. At regular intervals, a measurement of the travel time between the source and geophone is made. Thereafter the source and geophone are swapped and the same procedure is repeated. The result is a set of travel times from all source positions to all geophone positions from borehole 1 to borehole 2 and vice versa. A computer program optimizes the velocities of the materials in-between the two boreholes on the measured travel times. The results are normally presented as velocity contours (Fig. M-4b), which give an idea about the boundaries between different areas in the soil or rock mass between the boreholes. The velocities can be correlated with actual material boundaries or with the

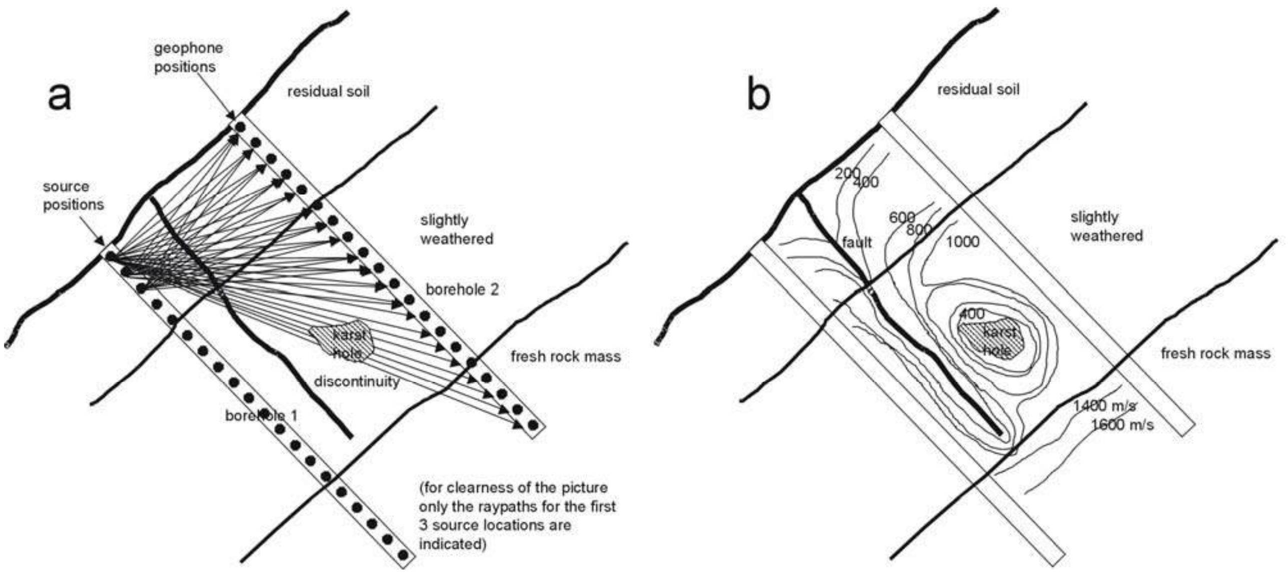


Fig. M-4. Tomography to determine rock mass quality. a) source and geophone positions, b) velocity contours in m/s

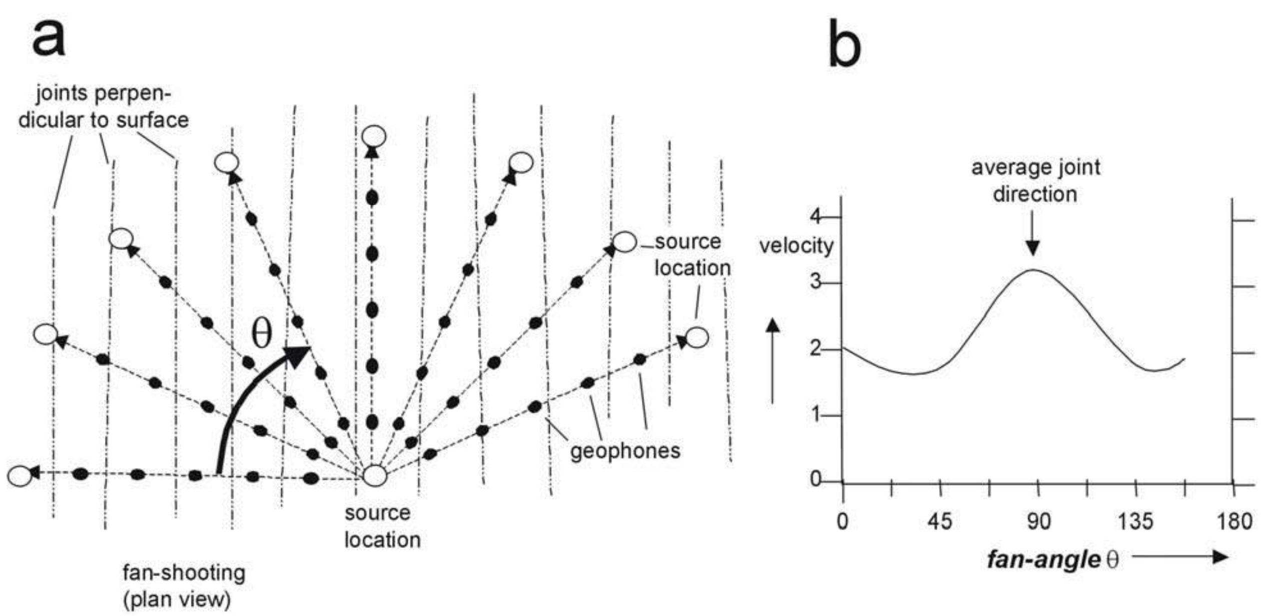


Fig. M-5. Fan-shooting. a) surface layout of sources and geophones, b) seismic velocity versus 'fan-angle θ' (after Hack & Price, 1990)

quality of the soil and rock mass in-between the boreholes, for example, with the number of discontinuities in a rock mass, degree of weathering, presence of karst holes, etc.

M.8 Anisotropy

Soil and rock masses are often anisotropic. Reasons for anisotropy are orientated minerals or sets of orientated discontinuities, such as bedding planes, joints, fractures, etc. The anisotropy causes that deformation properties of soil and rock mass are not the same in every direction. This also influences the behavior of seismic waves in the mass. For example, a rock mass with one set of discontinuities will have higher seismic velocities parallel to these discontinuities and lower velocities perpendicular to the discontinuities. Fig. M-5 shows a so-called seismic refraction fan shooting and the resulting seismic velocities measured in different directions. Velocity differences can be 50 % or more in different directions.

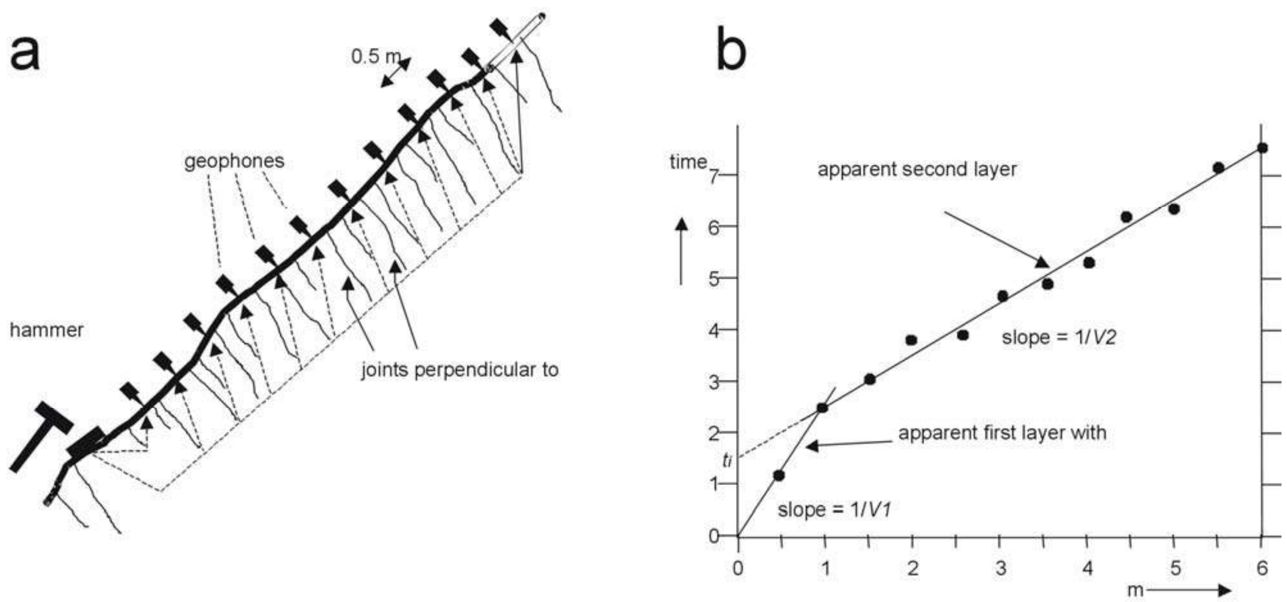


Fig. M-6. Anisotropy in vertical direction. a) ray paths, b) travel time versus distance for the first arrived signal

Anisotropy in the depth direction may cause erroneous interpretations. Fig. M-6 shows a slope with a series of discontinuities that are open at surface but become closed at a certain depth. The intact rock material in-between the discontinuities is the same near the surface as deeper in the rock mass and has seismic velocity *V2*. The difference between the surface rock mass and the deeper rock mass is, hence, only the presence of the discontinuities. A seismic refraction survey is done on the slope perpendicular to the direction of the discontinuities. The seismic waves have to cross or go around the discontinuities. However, the energy of a seismic wave crossing the open discontinuities will be reduced strongly because most energy is reflected on the open discontinuity surfaces (Table M-2). The energy is often so far reduced that the arrival of this signal is not noticed and the signal measured is from the wave that travels around the open discontinuities. This causes the time-distance graph to show an apparent two-layer case of seismic refraction: an apparent first layer with a low velocity (*V1*) being the layer with open discontinuities, and a second layer with the velocity of the rock mass without discontinuities which is the velocity of intact rock *V2*. Note that parallel to the discontinuities the waves do not need to cross the discontinuities and only direct waves are measured with velocity *V2*.

M.9 Attenuation and absorption

The energy contained in a seismic wave spreads spherically with distance from the source point. The energy flowing through an area of 1 m^2 is related to the distance from the source point by:

$$\frac{E_2}{E_1} = \left(\frac{r_1}{r_2}\right)^2 \qquad \text{[M-6]}$$

$$E = \text{energy flowing through 1 m}^2;\ r = \text{distance from source point}$$

The 2nd power relation in equation [M-6] causes a rapid drop of the energy with distance. Another reason for reduction of energy with distance is absorption. Absorption of energy takes place by loss of energy due to non-ideal elastic behavior of the soil and rock masses. The absorption is related to the frequency of the wave. Higher frequencies are more absorbed than lower frequencies. This causes a change in the overall model of the wave with distance. Further energy losses occur due to reflection, refraction and diffraction, and change of wave type at interfaces, for example, from P-wave to S-wave and vice versa.

The rapid loss of energy flow per m^2 and in particular, the rapid loss of energy of higher frequency components in a seismic signal cause that high-resolution seismic studies for geotechnical work are difficult to do. The energy emitted in high frequencies by a seismic source is normally relatively low and the rapid loss of energy causes that with larger distances the high frequency components are lost in noise.

M.10 Measurement of soil and rock properties

Measurement of mechanical properties of soil and rock masses is to a certain extent possible with seismic methods. If P and S waves are measured, a so-called ‘seismic E modulus’ and ‘seismic Poisson’s ratio’ can be determined:

$$\text{P}-\text{wave velocity} = V_p = \sqrt{\frac{\lambda + 2\mu}{\rho}} \quad \text{S}-\text{wave velocity} = V_s = \sqrt{\frac{\mu}{\rho}}$$

$$\text{Seismic Young's modulus} = E = \frac{\mu(3\lambda + 2\mu)}{(\lambda + \mu)} \quad \text{Seismic Poisson's ratio} = \sigma = \frac{\lambda}{2(\lambda + \mu)} \qquad \text{[M-7]}$$

$$\lambda, \mu = \text{Lamé's constants} \quad \rho = \text{density of material}$$

It should be realized that these properties are not the same as the Young’s modulus and Poisson’s ratio obtained from static or dynamic laboratory or field tests. This is because the deformation behavior of most soil and rock masses depends on frequency and is non-linear. The frequencies in seismic waves are normally not the same as frequencies used in dynamic testing and, more important, the stress and strain in seismic waves are very small compared to stress and strain in laboratory or field tests.

The relation between the frequency and the wavelength is:

$$V = f * \lambda \qquad \text{[M-8]}$$

$$V = \text{velocity of seismic wave};\ f = \text{frequency};\ \lambda = \text{wavelength}$$

Inhomogeneities in a soil or rock mass, such as boulders in a soil, joints in a rock mass, will not individually be detected if the wavelength of a seismic wave is low compared to the dimensions of the inhomogeneities. A rule-of-the-thumb is: if the dimension of the object is more than half the wavelength, it can be recognized as individual object (whereas the object will not be determined individually if the dimension is less than half the wavelength).

The influence of discontinuities on seismic P-wave velocity has been established empirically by many authors (among others Deere et al., 1967, Merkler et al., 1970). The relation by Deere et al. relates Rock Quality Designation to seismic velocities measured in the field and in the laboratory:

$$\left(\frac{V_{field}}{V_{laboratory}}\right)^2 * 100\% \approx RQD$$

V_{field} = seismic velocity measured in the field [M-9]

$V_{laboratory}$ = seismic velocity measured in the laboratory

RQD = Rock Quality Designation

The seismic velocity measured in the laboratory is done on intact rock. The velocity measured in the field is the velocity of a signal passing through intact rock but also through or around discontinuities and will hence result in a lower velocity.

Although very little work has been done in slope stability studies on relating material properties to attenuation of seismic signals (Luijk, 1998, Pyrak-Nolte & Shiau, 1998) it is expected that features such as discontinuities have a marked influence on the amplitude of the seismic signal of particular frequencies. Directly applicable methods are not yet established but are expected in the near future.

N ROCK MASS CHARACTERIZATION & CLASSIFICATION

N.1 Introduction

Rock masses have been described from the earliest geological maps onwards. The descriptions of the rocks were initially in lithological and in other geological terms. With increasing knowledge of geology, geological features and the influence of geology on engineering the amount of information to be included in a description for geotechnical purposes increased, leading to sets of rules for the description or characterization of a rock mass geotechnically. These are briefly reviewed in ch.N.2.

Parallel with this development, a movement took place in mining and engineering geology to combine the characterization of a rock mass with direct recommendations for tunnel support. This resulted in rock mass classification systems. Rock mass classification procedures were developed for underground excavations as an alternative to analytical analyses of a discontinuous rock mass. The systems were developed primarily empirically by establishing the parameters of importance, giving each parameter a numerical value and a weighting. This led, via empirical formulae, to a final rating for a rock mass. The final rating was related to the stability of the underground excavation used for the development of the classification system. In systems that are more elaborate, the rating was also related to the support installed in the excavation. Any other underground excavation made in a rock mass with a similar final classification rating is assumed to have the same stability appraisal or to require the same support as the excavations used for the development of the classification system. The reason for the development of classification systems is that analytical stability calculations for tunnels in discontinuous rock masses are nearly impossible. In the time before computers became generally available, a deterministic and even remotely realistic, analytical calculation was not really feasible. This brought some engineers onto the idea that empirical relations might be an alternative.

Various classification systems have been developed since 1946. A division is often made between so-called ‘early’ systems and ‘recent’ systems (Bieniawski, 1989). This division is also maintained in this description of existing characterization and classification systems. The main difference between the two groups of classification systems is the number of parameters used in the systems. The ‘early’ (ch.0) systems often depend on only one or two parameters and were developed for underground excavations. ‘Recent’ (ch.N.4) classification systems use more parameters. The ‘recent’ systems also have been primarily developed for underground use but in the last decades, some extensions to surface applications (e.g. slope stability, foundations, and rippability) have been published (ch.N.5). The New Austrian Tunneling Method (NATM) (Pacher et al., 1974) is also discussed in ch.N.4.6. This system includes legal and contractual parameters not found in any of the other systems, and is strictly related to tunneling. In ch. N.4.7 the Rock Engineering Systems (RES) methodology is briefly discussed. Although the RES methodology itself is not a classification system as the other systems discussed, applications of the RES to slope stability, such as presented in ch.N.5.9., resemble the application of a classification system. In ch. N.6 correlations between the different systems are discussed as well as calculation methods, the parameters used, and the influence of these parameters on the final classification result.

This chapter covers the characterization and classification systems, which are the main and (in the opinion of the author) most interesting systems with good published documentation. Most of these systems have been used in different geological and geotechnical environments for different projects. In many civil engineering or mining projects, systems have been developed or existing systems have been modified. Often these have been modified to the particular needs of a project and might not be applicable to other projects or other geological or geotechnical environments. Sometimes parameters or factors of different systems are combined (Japan, 1992). Only the main parameters and characteristics of the systems are described. All characterization and classification systems are accompanied by (extensive) tables for descriptions of parameters and, if appropriate, by tables with recommendations for civil or mining engineering applications. These tables have not been copied and the reader is referred for the details to the cited literature.

N.2 Descriptive and characterization systems

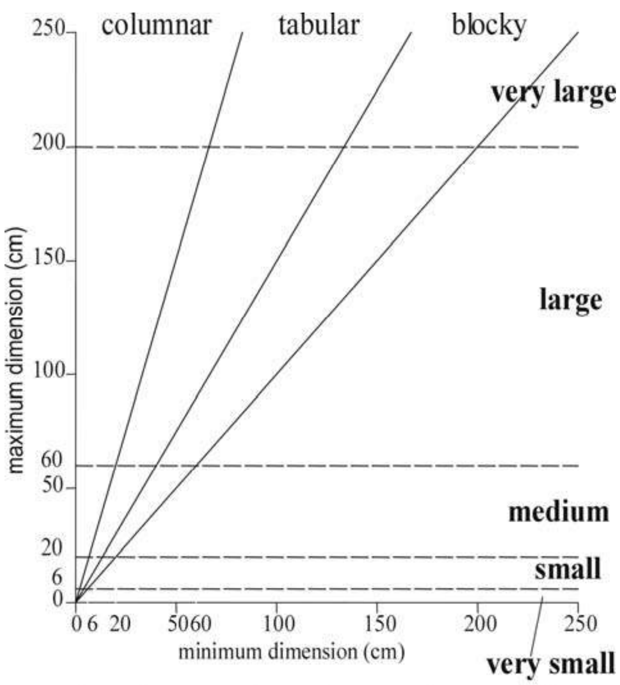
Two standard systems that characterize rock mass and express rock mass characteristics in standard terms are those in BS 5930 (1981) and the ISRM Basic Geotechnical Description (ISRM, 1981b). A third, mainly used in the USA, is the Unified Rock Classification System (URCS) (Williamson, 1980, 1984). The systems do not result in a numerical value or direct design recommendation. The systems facilitate communication on rock mass characteristics and are in use widely for various purposes.

Borehole core and exposure logging

Deere et al. (1964, 1967) and Moye (1967) published detailed instructions and recommendations for the description of rock masses and the presentation of rock mass data in the form of borehole core logs. This has been adapted by the working party of the Geological Society Engineering Group in the report ‘The logging of rock cores for engineering purposes’ (Anon., 1970).

British Standard BS 5930

The present version BS 5930 (1981) gives recommendations for a standard description of a rock mass. The characteristics are described according to a series of standard terms and phrases and lead to an extensive rock mass name. An interesting feature of the British Standard is the recognition of the importance of intact rock block size and form. Rock blocks are described as very large blocky, very small columnar, etc. (Fig. N-1). Although not quantified, the descriptive terms relating to block form are very useful in engineering geology.



note: the ratios separating columnar, tabular and blocky (continuous lines) are an interpretation from Price (1992); these are not quantified in BS 5930 (1981).

Fig. N-1. Block size and form description according to British Standard (BS 5930, 1981) with ratios for block form (Price, 1992)

ISRM Basic Geotechnical Description

ISRM (1981b) recommends the following geotechnical rock mass parameters to be described or measured:

1. Rock lithology, with geological description
2. Discontinuity spacing (bedding or layer thickness and joint/fracture spacing)
3. Unconfined compressive strength (UCS)
4. The friction angle of the discontinuities

The far more extensive ISRM ‘Suggested methods for rock and discontinuity characterization, testing and monitoring’ (1978b, 1981a) recommends the quantitative description of a very extensive and complete set of rock mass parameters for the characterization of a rock mass.

Unified Rock mass Classification System (URCS)

The Unified Rock mass Classification System (Williamson, 1980, 1984) has been specially designed to facilitate the communication on rock mass parameters. The parameters described are: 1) degree of weathering, 2) strength, 3) discontinuities, and 4) density. Hsein (1990) extended the system to give an overall ‘performance index’ for a rock mass.

Discussion

It is regrettable that the descriptive systems do not use the same descriptions for the same parameters. For example, Table N-1 shows the description of strength of intact rock for three systems published within a period of two years. The systems use different intervals and terms to describe the strength of intact rock. Similar differences are found for discontinuity spacing, degree of weathering, etc. The differences are often based on futile reasons that do not justify the differences. For example, in the ISRM system (1981b) interval

Table N-1. Characterization of intact rock strength according to BS 5930 (1981), ISRM (1981b) and URCS (1980)

strength of intact rock						
BS 5930 (1981)		ISRM (1981b)		URCS (1980)		
interval MPa	class	interval MPa	class	interval psi	interval MPa	class
> 200	extremely strong	> 200	very high	> 15,000	> 103	rebounds (elastic)
100 - 200	very strong	60 - 200	high			
50 - 100	strong			8,000 - 15,000	55 - 103	pits (tensional)
12.5 - 50	moderately strong	20 - 60	moderate	3,000 - 8,000	21 - 55	dents (compression)
5 - 12.5	moderately weak	6 - 20	low	1,000 - 3,000	7 - 21	craters (shears)
1.25 - 5	weak	< 6	very low	< 1,000	< 7	mouldable (friable)
< 1.25	very weak					

boundaries are used which resemble the particle size[20] boundaries, for which the philosophy is that it is easily remembered. Other differences are caused by cultural background (for example, the use of psi interval boundaries in the URCS system, 1980).

The British Standard, ISRM, and URCS systems are presented as basic description systems. These allow additional information to be provided with the basic description. However, standard guidelines for the additional information are not given. The only system, whether classification or characterization that includes the parameter of rock material density, is the URCS. Probably this parameter is included because a main user of the system is the Soil Conservation Service of the U.S. Department of Agriculture. Apart from applications for construction materials, the author is not aware of any application for which density is of major importance.

N.3 Early classification systems

Table N-2. Early classification systems (note the increasing recognition of the importance of discontinuities with time)

year	system	main characteristic
1946	Terzaghi's Rock Load Classification system	Continuum
1958	Lauffer's Stand-up Time Classification	Various standard rock mass types with increasing degree of brokenness
1967	Deere's RQD index classification	Recognition of importance of discontinuities
1972	Wickham's Rock Structure Rating (RSR)	First system to identify the importance of discontinuities as a separate item in classification

Table N-2 shows the most important so-called early classification systems. The increasing recognition of the importance of discontinuities is clear over the years; while Terzaghi calculates a solid piece of rock, the system of Wickham classifies discontinuities and discontinuity properties.

Terzaghi - rock load classification system

K. Terzaghi (1946) classified rock masses with the objective of predicting the load on steel arch support sets in tunneling. The parameters taken into account are the 'rock condition', the dimensions of the tunnel, the depth below the terrain surface and below the water table (Fig. N-2).

The assumption that the steel arch set has to support a certain volume of rock above the tunnel implies that the rock is allowed to deform until it can exert a force on the support. Terzaghi modeled deformation zones (a crack or shear zone) starting at the toes of the steel arch set in upward direction to allow the volume of

[20] Soil particle size intervals: 0.002, 0.06, 2, 60 mm, etc. (BS 5930, 1981).

rock above the tunnel to rest on the set. The load on the set is assumed to be the weight of the rock volume in-between the deformation zones up to a certain height above the tunnel (Hp) and the water load (W).

The 'rock condition' parameter describes the rock mass in various classes such as 'hard and intact', 'hard stratified or schistose', etc.. In addition, classes for crushed and swelling rock are distinguished. A table is provided which, based on the 'rock condition', gives the 'rock load (Hp)' parameter as a factor of the width and height of the tunnel. The table also includes estimates of the variation in pressure on the support (e.g. the presence or absence of side-pressure on the steel arch sets)[21].

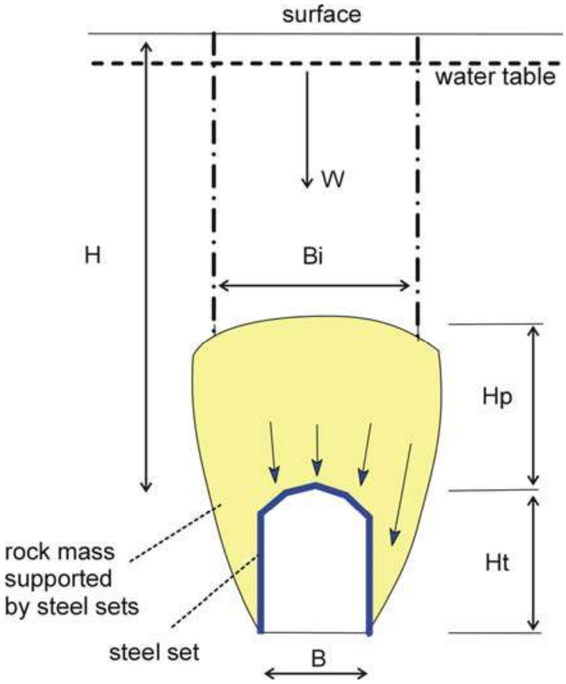


Fig. N-2. Terzaghi's rock load classification. The rock volume supposed to be supported by the steel arch set is hatched (after K. Terzaghi, 1946)

Lauffer - stand-up time classification

Lauffer (1958) related the stand-up time of an un-supported span to standard rock mass types. Compared to the Terzaghi approach this was a major improvement as discontinuities (structural defects) were considered. The characterization of the rock mass was, however, not done by describing different rock mass parameters but had to be selected from a number of characterizations of standard type rock masses prescribed by Lauffer. Later the Lauffer system became the basis for the New Austrian Tunneling Method (ch. N.4.6).

Deere - RQD index classification

Deere et al. (1967, 1988, 1989) introduced the Rock Quality Designation (RQD). The RQD index is measured on borehole cores, following eq.[N-1].

$$RQD = \frac{\sum length\ pieces\ of\ intact\ core\ with\ length > 10\ cm}{total\ length\ drilled} * 100\,\% \qquad [N\text{-}1]$$

The intact pieces of core (highly weathered pieces of rock or infill material should not be included) should be measured along the centerline of the core and the RQD values should be calculated separately for each lithostratigraphic unit. Core runs should preferably be not longer than 1 or 1.5 m. The RQD values provide a measure of the brokenness of the rock mass. Deere et al. (1967) related the RQD index to support types for tunnels. It is therefore the first classification system incorporating an index for the amount and quality of discontinuities in a rock mass. Recently 'rock quality charts (RQC)' have been based on RQD measurements by Şen et al. (1991, 1992).

Wickham - Rock Structure Rating (RSR)

Wickham et al. (1972, 1974) developed the Rock Structure Rating. The system is based on quantitative parameters for:

parameter A rock structure (origin, hardness, geological structure),

[21] Severe doubt has been expressed about the concept of a deformation zone starting at the toe of the steel support and developing in upward direction. The development of deformation zones as indicated is only likely in a massive, not jointed (thus continuous), rock mass. In a discontinuous rock mass the deformations will follow existing discontinuities and may well lead to a totally different volume of rock to be supported. Secondly, the deformation zones will develop in upward direction only under low horizontal stress. With a higher horizontal stress the normal stress on the proposed deformation zones will be too high to allow shearing or tension cracking, thus preventing the development of deformation zones, whereas if the horizontal stresses are considerably larger than the vertical stresses the deformation zones may well develop horizontally rather than vertically. The assumption that the water load has to be supported by the steel set over the full height up to the water table is also unlikely as this would only be the case for a tunnel with impermeable lining capped by a fully permeable waterlogged rock mass.

parameter B discontinuity pattern with respect to the direction of the tunnel (joint-spacing and orientation relative to direction of tunnel drive),

parameter C groundwater inflow (based on overall rock mass quality described by parameters A and B, joint condition, amount of water inflow in tunnel),

factor AF for type of excavation (drilling - blasting)

The final rating is:

$$\begin{gathered} RSR\ (rock\ structure\ rating) = A + B + C \\ RSA\ (adjusted\ RSR) = RSR * AF \end{gathered} \qquad [N\text{-}2]$$

The outcome of eq. [N-2] is used to design rib, bolt, and shotcrete support for tunnels via the support recommendations of the Terzaghi system. The RSR (or RSA) system is the first system that resembles the recent systems, which are based on a number of rock mass parameters.

N.4 Recent classification systems for underground excavations

N.4.1 Bieniawski's RMR

Bieniawski's RMR system is one of the oldest of the often so-called 'recent' systems (appendix A). The system has been developed in South Africa for underground mining. The system is based on a combination of six parameters (eq. [N-3]). Each parameter is expressed in a point rating and the final RMR ranges between 0 (very poor rock for tunneling) to 100 (very good rock for tunneling).

$$\begin{gathered} RMR\ (rock\ mass\ rating)\ = \\ (IRS + RQD + spacing + condition + groundwater) + reduction\ \ factor \\ IRS = \text{Intact Rock Strength} \quad RQD = \text{Rock Quality Designation} \\ spacing = \text{discontinuity spacing of one set (see text)} \\ condition = \text{expression for condition of (shear strength) of one set (see text)} \\ groundwater = \text{expression for groundwater inflow (pressure)} \\ reduction\ \ factor = \text{depending on orientation of engineering} \\ \text{structure relative to the main discontinuity set} \end{gathered} \qquad [N\text{-}3]$$

In the latest modification published by Bieniawski (1989) the 'condition of the discontinuity' parameter has been extended and more specified. In addition, the RMR has been related to the span and stand-up time of the excavation.

The spacing and condition parameters are determined by the weakest discontinuity set or by the discontinuity set with the most adverse influence on stability. Support of an underground excavation is determined by the RMR parameter and results in five different support classes.

N.4.2 Barton's Q-system

The Q-system of Barton et al. (1974, 1976a, 1988, appendix A) expresses the quality of the rock mass in the so-called Q-value. The Q-value is determined with eq. [N-4]. The first term RQD (rock quality designation) (ch. 0) divided by J_n (joint set number) is related to the size of the intact rock blocks in the rock mass. The second term J_r (joint roughness number) divided by J_a (joint alteration number) is related to the shear strength along the discontinuity planes and the third term J_w (joint water parameter) divided by SRF (stress reduction factor) is related to the stress environment for the discontinuities around the tunnel opening.

$$\begin{gathered} \mathrm{Q} = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} \\ Q = \text{rock mass quality} \\ RQD = \text{rock quality designation} \quad J_n = \text{joint set number} \quad J_r = \text{joint roughness number} \\ J_a = \text{joint alteration number} \quad J_w = \text{joint water reduction factor} \\ SRF = \text{stress reduction factor (depending on intact rock strength and stress environment)} \end{gathered} \qquad [N\text{-}4]$$

A multiplication of the three terms results in the 'Q' parameter, which can range between 0.00006 for an exceptionally poor rock mass to 2666 for an exceptionally good rock mass. The numerical values of the class boundaries for the different rock mass types are subdivisions of the Q range on a logarithmic scale.

Intact rock strength influences the result only when the intact rock strength is relatively low compared to the stress environment. J_r and J_a are the parameters for the discontinuity roughness and alteration of the weakest discontinuities (Barton et al., 1974) or the discontinuity most likely to allow failure to initiate (Barton, 1976a). The Q-value determines the quality of the rock mass, but the support of an underground excavation is based not only on the Q-value but is also determined by the different terms in eq.[N-4]. This leads to a very extensive list of classes for support recommendations.

N.4.3 Laubscher's MRMR

Laubscher (1977, 1981, 1984, 1990, appendix A) modified the RMR classification of Bieniawski. In his system, the stability and support are determined with eq.[N-5]. The main parameters are the same as for the Bieniawski system but the parameter for groundwater is included in the condition parameter. The number of classes for the parameters and the detail of the description of the parameters are more extensive than in the RMR system.

$$RMR = IRS + RQD + spacing + condition$$

RMR = Rock Mass Rating IRS = Intact Rock Strength
RQD = Rock Quality Designation
$spacing$ = expression for the spacing of discontinuities
$condition$ = condition of discontinuities (parameter also dependent on
groundwater presence or quantity of groundwater inflow in tunnel)
(parameters for RQD and spacing can be replaced by the fracture frequency) [N-5]

$$MRMR = RMR * adjustment\ factors$$

$MRMR$ = Mining Rock Mass Rating
$adjustment\ factors$ are compensation factors for : the method of excavation,
orientation of discontinuities and excavation, induced stresses and future weathering.

The resulting RMR parameter is multiplied by adjustment factors depending on future (susceptibility to) weathering, stress, orientation, method of excavation and the amount of free block faces that facilitate gravity fall, and then becomes the MRMR (Mining Rock Mass Rating). The values of RMR and MRMR determine the so-called 'reinforcement potential'. A rock mass with a high rock mass rating before the adjustment factors are applied has a particular reinforcement potential. A high RMR rated rock mass can be reinforced by for example rock bolts whatever the MRMR value might be after excavation. Contrariwise, rock bolts are not a suitable reinforcement for a rock mass with a low RMR (has a low potential for reinforcement) even if after excavation the MRMR is not much lower than the RMR.

Laubscher uses a graph for the spacing parameter. The parameter is dependent on a maximum of three discontinuity sets that determine the size and the form of the rock blocks. The condition parameter is determined by the discontinuity set with the most adverse influence on the stability. The Laubscher system specifies values for the discontinuity condition parameter depending on different situations with respect to water or water pressure and does not have a separate parameter for water in the RMR equation (eq. [N-5]).

The concept of adjustment factors for the rock mass before and after excavation is very attractive (Laubscher, 1990). This allows for compensation of local variations, which may be present at the location of the rock mass observed, but might not be present at the location of the proposed excavation or vice versa. In addition, this allows for quantification of the influence of excavation and excavation induced stresses, excavation methods and the influence of past and future weathering of the rock mass.

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

N.4.4 Franklin's Size Strength Classification

Franklin et al. (1970, 1974, 1975a, 1986) and Louis (1974) developed a classification system based on intact rock strength and the block size of intact rock blocks. The intact rock strength can be established by hammer and scratch tests or Point Load Strength (PLS) tests. Block size is defined as the diameter of a typical rock block. Block size is determined by observing an exposure or rock core from bore holes. The intact rock strength, the influence of rock block diameter and tunnel size have been related to tunnel stability and potential failure mechanisms.

N.4.5 Modified Hoek-Brown failure criterion for jointed rock masses

The Hoek-Brown failure criterion for rock masses has recently been adjusted. It now incorporates a simple rock mass classification system (Hoek et al., 1992). The failure criterion is formulated as follows:

$$\sigma_1' = \sigma_3' + \sigma_c * \left(m_b * \frac{\sigma_3'}{\sigma_c} \right)^a$$

σ_1' = major principal effective stress at failure [N-6]

σ_3' = minor principal effective stress at failure

σ_c = intact rock strength

m_b and a are parameters describing the rock mass structure and surface conditions

The rock mass parameter σ_c (intact rock strength) is derived from a field estimate that resembles the system for estimation of field intact rock strength by Burnett (1975), however, the classes, descriptions and class boundaries are different. The parameters m_b and a are derived from a matrix describing the 'structure' and the 'surface condition' of the rock mass. The 'structure' is related to the block size and the interlocking of rock blocks while the 'surface condition' is related to weathering, persistence, and condition of discontinuities. The parameter for rock mass 'structure' is divided in four classes, ranging from 'blocky' (well interlocked, undisturbed rock mass, large to very large block size) to 'crushed' (poorly interlocked, highly broken rock mass, very small blocks). The parameter for 'surface condition' is divided in five classes, ranging from 'very good' (unweathered, discontinuous, very tight aperture, very rough surface, no filling) to 'very poor' (highly weathered, continuous, narrowly spaced discontinuities, polished/slickensided surfaces, soft infilling).

N.4.6 NATM - New Austrian Tunneling Method

The New Austrian Tunneling Method (NATM) (Müller, 1978, Kovári, 1993, Pacher et al., 1974, Rabcewicz et al., 1964, 1972) comprises characterization and classification but also includes rock mass modeling, deformation monitoring, legal contract aspects and the construction of a tunnel. Various modifications, adjusted to local circumstances, have been developed worldwide, noticeably in Japan (Japan, 1992).

N.4.7 Hudson's RES - Rock Engineering Systems

The Rock Engineering Systems (RES) methodology developed by Hudson (1992), relates the interaction of parameters that have an influence on engineering in discontinuous rock masses. As well the influence of a parameter on the engineering structure as the influence of a parameter on other parameters is quantified and results in a rating for a parameter of the engineering structure. This last-named parameter can be, for instance, the stability or instability of a tunnel or slope. Parameters can be parameters describing properties of a rock mass, such as intact rock strength, discontinuity orientation, etc., but also parameters describing external influences on rock mass parameters or engineering structures, such as climate, geomorphologic processes, etc. The quantification of all the interactions results in a matrix with which the required parameter, for example, the stability of a tunnel, is determined. Quantification of the interactions or influences between parameters and between parameters and engineering structure can have any form. These can be, for example, differential equations, binary operations (0 or 1, for example, for features that are either present or not present), classifications, or numerical calculations. How these relations are established (e.g. by engineering judgment or actually proved by testing) is of no importance. The reliability and accuracy of the final result depend, however, on the reliability and accuracy of the relations (and obviously of the input data). The methodology resembles the working of a neural network as also pointed out by Hudson, however, the relations between in- and output parameters in a neural network are normally of a simpler form.

The methodology is not a classification system, but rather a methodology of thinking for engineering in or on discontinuous rock masses. Hudson gives no detailed applications or relations between parameters, however, suggestions are given for implementation of the methodology in various forms of engineering in or on discontinuous rock masses.

N.5 Rock mass classification systems for surface engineering applications

Some rock mass classification systems developed for underground excavations have been used for surface engineering structures such as slopes directly (Bieniawski, 1976, 1989, Barton et al., 1974) or in a modified form (Haines et al., 1991, Robertson, 1988, Romana, 1985, 1991, Selby, 1980, 1982). The systems developed by Shuk (1994) and Hack (1998) are specially designed for slope stability. In addition, systems have been designed specially for excavation, rippability, etc.

N.5.1 Barton's Q-system applied to slope stability

Barton et al. (1974) included in his system an estimate of the friction angle for the shear strength of discontinuities. This friction angle can be used in, for example, slope stability calculations.

N.5.2 Bieniawski's RMR applied to slope stability

Bieniawski (1976, 1989) included not only recommendations for underground excavations but also for foundations and slope stability. The author is not aware whether the system has actually been used for slope stability analyses in the form as presented by Bieniawski.

N.5.3 Vecchia - Terrain index for stability of hillsides and scarps

Vecchia (1978) designed a classification system to quantify the stability of a hillside or scarp, e.g. natural slopes, based on parameters for 'lithology' and 'attitude', and a 'friction' parameter that depends on the 'lithology' and 'attitude' parameters. The 'lithology' parameter is determined by the presence of clay and shale in the rock mass and by characteristics of the rock mass such as loose, coherent, or massive rock masses. This, combined with interbedded lithologies, results in a series of different standard classes for the lithology, e.g. from shale with a few coherent beds (rating 10 points) to massive rocks with few or no discontinuities (rating 90 points). The rock mass in the field is visually compared to the standard classes provided by Vecchia (1978), classified, and rated. The 'attitude' parameter assigns a rating ranging from 0 (unfavorable) to 12 (favorable) to the orientation of discontinuities with respect to the orientation of slope or scarp. The 'friction' parameter is a rating for the friction along the main discontinuity (set) allowing sliding. The 'friction' parameter with a rating between 2 and 10 is assigned on the bases of the classes determined for the 'lithology' and 'attitude' parameters. The 'friction' parameter is thus not a separate parameter established in the field. A terrain index (IT) is calculated as follows:

$$I_T = \text{terrain index} = \textit{lithology} + \textit{attitude} - \textit{friction} \qquad \text{[N-7]}$$

The simplicity of the system and the limited number of parameters, effectively only two, which have to be assessed in the field, are very attractive. This simplicity, however, may also be its largest drawback. The quantity of standard lithologies given is limited, will not always fit a rock mass in the field and the visual comparison may be ambiguous. The definition of standard lithologies resembles the approach of standard rock mass classes as used by Lauffer (1958, ch. 0) for underground excavations.

Other drawbacks are that there are no provisions for more than one discontinuity set and the limited options for the friction along the discontinuities. An interesting observation (Vecchia, 1978) is made that water in surface hillsides or scarps is generally limited to surface water. Water pressures in the rock mass are therefore not considered.

N.5.4 Selby - Geomorphic rock mass strength classification

Selby (1980, 1982) designed the Geomorphic Rock Mass Strength classification. The classification is designed with emphasis on geomorphology rather than engineering. The system resembles the Bieniawski system and includes for a large part the same parameters. Parameters assessed and rated are: intact rock strength (which can also be assessed by Schmidt hammer), degree of weathering, spacing of joints, joint orientations, widths (aperture) of joints, continuity (persistence) of joints combined with joint infill, and outflow of water (ratings are given in Table N-4, page N-134). The ratings obtained for each parameter are added and the total rating is an expression for the rock mass strength. The rock mass strength is divided in five classes ranging from very strong to very weak. The total rating is not directly related to slope stability but is used in the qualification and quantification of geomorphologic processes.

N.5.5 Robertson' RMR (modified Bieniawski)

Robertson (1988) modified the Bieniawski (RMR) system for use in slope stability analyses. The main distinction with the original system is that for RMR > 40 the stability of the slope is fully governed by the discontinuities whereas for an RMR < 40 the slope stability can be assessed by a modified Bieniawski system. In Table N-4 (page N-134) the parameters are listed that are used for determining the slope stability for an RMR < 40.

N.5.6 Romana's SMR (modified Bieniawski)

Romana (1985, 1991) extended the RMR classification system to slope stability problems expressed in the slope mass rating (SMR):

$$SMR = RMR - (F_1 * F_2 * F_3) + F_4$$

$$\begin{gathered} SMR = \text{Slope Mass Rating} \\ RMR = \text{Rock Mass Rating (same as Bieniawski's } RMR) \\ F_1 = \text{factor for parallelism of the strikes of discontinuities and slope face} \\ F_2 = \text{factor for discontinuity dip angle} \\ F_3 = \text{factor for relation between slope face and discontinuity dip} \\ F_4 = \text{factor for method of excavation} \end{gathered} \qquad [N-8]$$

The parameters F1, F2 and F3 are for one discontinuity only and therefore the SMR should be calculated for each discontinuity set and the lowest resulting SMR value gives an indication for the stability of the slope. The SMR value predicts the possibility of a 'soil-type' failure (normally for low values) and the amount of plane and wedge failures (normally for higher SMR values). The SMR value is also used to indicate the support measures to be taken for (partially) unstable slopes.

N.5.7 Haines (modified Laubscher)

The Laubscher (ch.N.4.3) system is used to forecast rock slope stability in open pits in South Africa (Haines et al., 1991). The adjustment ratings incorporated in the Laubscher system are reported to be of great benefit for slope stability estimation. The design chart to determine the slope dip related to slope height and factor of safety using the MRMR of the Laubscher classification is shown in Fig. N-3. Haines et al. point out that the system is designed in a mining environment where safety requirements are generally lower than in civil engineering. However, they also incorporated slope dips for slopes with a factor of safety equal to 1.5. These might be suitable for civil engineering. The system has been designed empirically based on existing slopes in

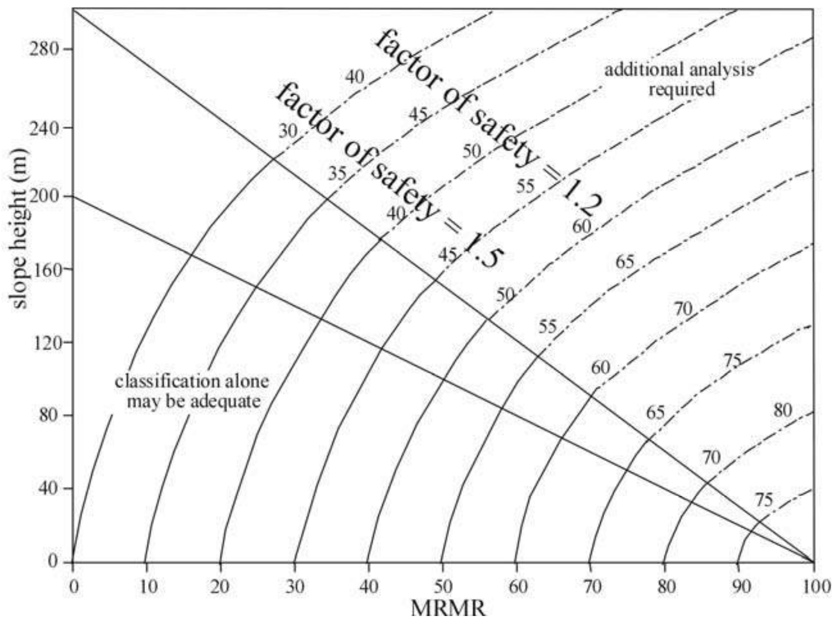


Fig. N-3. Design charts to determine slope dip and height using MRMR classification data (after Haines et al., 1991) open pit mines and analytical calculations.

The intact rock strength value necessary in the Laubscher system can be replaced by an estimate with Schmidt hammer values for soil and 'softer' rocks and by the density of the material for 'harder' rocks (Haines et al., 1991). The orientation of the slope with respect to the discontinuity orientations is incorporated in an adjustment percentage.

N.5.8 Shuk - Natural slope methodology (NSM)

Designing the inclination of a new slope based on slope dips measured on existing natural and artificial slopes is often used in the design of new slopes to be excavated. Normally no formal characterization or classification of the rock mass is applied.

The Natural Slope Methodology (NSM) (Shuk, 1994a, 1994b, 1994c, 1994d) is based on this principle. This method uses a statistical analysis of existing natural slopes to predict rock mass and soil parameters, and the probability of slope stability. The method is based on a presumed relation (eq.[N-9]) between the height and length of a natural slope.

$$Height_{slope} = a * \left(Length_{slope}\right)^{b}$$

$$a = f\left[\left(\frac{c}{\gamma}\right)^{(1-b)}\right] \qquad b = s_a + p_a \qquad s_a = \tan\varphi_r + \frac{c}{\gamma * Height_{slope}}$$

$$\begin{aligned} p_a &= \text{non-dimensional pressurization parameter} \\ &\quad \text{(related to tectonics, water pressures, etc.)} \\ \varphi_r, c &= \text{residual friction angle, residual cohesion of rock mass or soil} \\ \gamma &= \text{unit weight of rock mass or soil} \\ a \text{ and } b &= \text{weighting factors} \end{aligned} \qquad [N\text{-}9]$$

Equation [N-9] is only one of the possible relations. Shuk has not investigated other more complicated relations in depth at present. Back analyses of a large number of natural slopes and optimization of eq. [N-9] result in estimates for different rock (mass) or soil parameters. The method can also be combined with anisotropic behavior of rock masses and soils. The methodology is very attractive, as it does not require extensive field investigations.

A problem with the methodology as reported, is that not all relations, parameters, and especially the methods used to optimize the non-linear relations on the data are clear from the articles published. It is thus impossible to perceive the methodology, or comment on it in detail at present[22]. It is understood that the methodology has been still further developed and future versions and publications may show the full potential.

N.5.9 Hudson's RES - rock mass characterization applied to assess natural slope instability

Mazzoccola et al. (1996) presented an example for determining natural slope instability following the Rock Engineering Systems (RES) methodology (ch.N.4.7, Hudson, 1992). The rock mass characterization evaluates the interactions between and the influence of all parameters that may be of influence on slope stability. Twenty parameters are evaluated ranging from parameters as the geology, folding, etc. to parameters describing the rock mass such as weathering, the number of discontinuity sets, slope orientation, etc. In addition, external influences are included such as climate influences, as rainfall, freeze and thaw, etc. The instability of the slopes is determined following the Rock Engineering Systems (RES) methodology.

The publication shows that a good correlation is obtained with a predictability rating for slope instability based on indicators of potential instability of the natural slopes (Nathanail et al., 1992).

[22] Therefore, this system has not been included in Table N-4.

N.5.10 Slope stability probability classification (SSPC)

The Slope Stability Probability Classification (SSPC) system is based on a three-step approach and on the probabilistic assessment of independently different failure mechanisms in a slope (Hack, 1998, appendix A). First, the scheme classifies rock mass parameters in one or more exposures. In the second step, these are compensated for weathering and excavation disturbance in the exposures. This gives values to the parameters of importance to the mechanical behavior of a slope in an imaginary unweathered and undisturbed 'reference' rock mass. The third step is the assessment of the stability of the existing or any new slope in the reference rock mass, with allowance for the influence of excavation method and future weathering. The result of the system is probabilities for different failure mechanisms. The system has been used in Spain, Austria, South Africa, and the Dutch Antilles.

N.5.11 Excavatability, rippability and blasting assessment

Various classifications have been developed to assess the excavatability and rippability of rock masses at terrain surface (Franklin et al. 1971, Weaver, 1975, Kirsten, 1982). Franklin et al. based the excavatability on strength (unconfined compressive or point load strength) and discontinuity spacing in accordance with the Franklin size - strength classification (ch. N.4.4). Weaver based his rippability assessment on the Bieniawski classification for underground excavations (ch. N.4.1) while the approach of Kirsten is based on the Barton classification (ch. N.4.2). Most excavatability or rippability assessment systems are equipment specific, e.g. give recommendations for a particular type of excavation or ripping equipment. Some systems also include seismic velocities to assess rippability (Weaver, 1975).

N.6 Calculation methods and parameters in classification systems

N.6.1 Method of calculation

Addition, subtraction, multiplication, and division of logarithmic, linear, or non-linear parameters are used in the different existing classification systems. These are used either solely or in combination and no clear benefit from using a particular type of numeric representation or calculation method seems to exist. Some slope classification systems that use a method of calculation based on combining different parameters to give one single rating number, can give results difficult to perceive (for example: Robertson's RMR, and Romana's SMR. In these classification systems parameters have an influence on the stability rating for a slope which instability may be caused by a physical mechanism that is independent from those parameters. For example, intact rock strength is used to calculate the stability rating while a slope is unstable because of sliding on a discontinuity with a thick clay infill and hence intact rock strength is of no importance for the stability or instability of the slope.

N.6.2 Correlations between different classification systems

Various relationships have been established between the different existing classification systems (Cording et al., 1972, Rudledge et al., 1978, Yufu, 1995). An important correlation is that between the systems of Bieniawski and Barton. The existence of a correlation of the numerical rating values was already established in 1976 (Bieniawski, 1976, 1989) and is shown in Fig. N-4. It should be noted that the quality classes do not perfectly correlate (continuous lines) and the scatter allows for one to two classes difference between the two systems (dashed lines). This may be due to the definition of the classes. A more correct comparison between the two systems should be based on the recommended support for underground excavations. The recommended types of support are, however, different for the two systems and a comparison cannot be easily made. The two systems (Bieniawski and Barton) were developed in different parts of the world, in different types of mines, in different rock types and, above all, the systems use partly different parameters, and have defined differently the parameters included in both systems. That two so very different systems do correlate is strange but tentative reasons for this correlation might be:

1. Correlation between parameters; e.g. a rock mass with a low intact rock strength has often also a small discontinuity spacing or a low shear strength along discontinuities or both. A correlation between different classification systems is always obtained for the majority of possible rock masses.

2. Biased users: The parameter difference is compensated by adjusting parameter(s) to values which the experienced user considers to be appropriate for the rock mass. Thus, if the user knows from experience or by other means that the rock mass is poor, he unconsciously creates also a poor rock mass rating by taking lower values for the individual parameters of the system he uses.

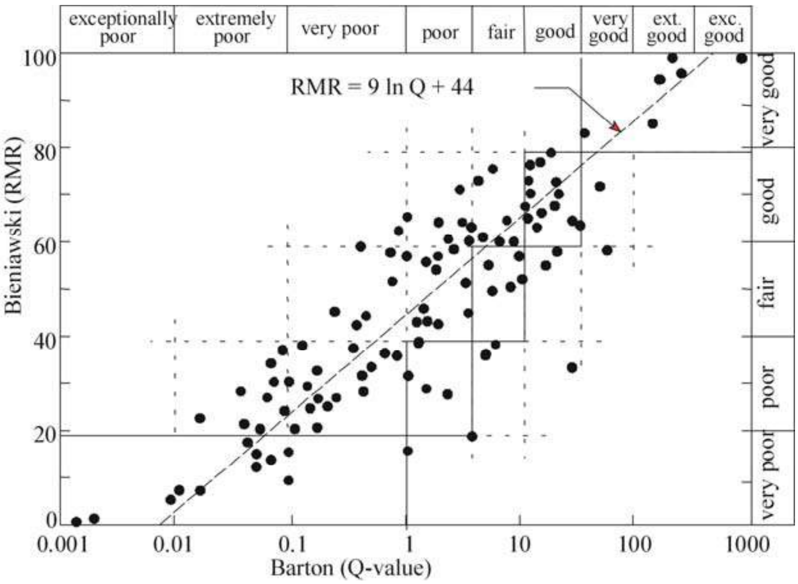


Fig. N-4. Correlation between Bieniawski (RMR) and Barton (Q). Data from case histories with RMR and Q-system (after Bieniawski, 1989). (Continuous lines indicate correlating classes of rock mass quality.)

N.6.3 Influence of parameters in existing classification systems

An inventory of the most important rock mass parameters of interest for engineering structures in or on a rock mass is presented in Table N-3. This table is based on the experience and intuition of the author and on the literature. The parameters listed are, in part, those occurring in some of the existing characterization and classification systems previously discussed. Many systems do, however, not contain one or more of the parameters and also the influence of parameters in the existing classification systems is not for all classification systems the same. Table N-4 presents the various parameters used in the existing rock mass classification systems and gives a crude indication of the maximum influence of each parameter on the final rating or recommendations for tunnel support or slope geometry. It is impossible for all systems to indicate the influence per parameter exactly because in some systems, parameters are dependent or parameters are not linear. The percentages indicate the reduction of the final rating when that parameter is given its minimum value and all other parameters have their maximum value, compared to the rating based on the maximum value of all parameters. If a parameter is linked to another parameter then the other parameter is also changed as required[23].

Noteworthy differences in the influence of parameters (Table N-4) are:

1. The absence of the intact rock strength (except for a low intact rock strength/environment stress ratio), in the Barton system.
2. The absence of discontinuity spacing in the Barton system.
3. The strong reduction in influence of the water parameter in the Laubscher and Haines systems as compared to the systems of Bieniawski and Barton.
4. The absence of a water/water pressure parameter in the Robertson modification for slopes of the Bieniawski system and in the slope stability system of Vecchia.
5. The strong influence of the susceptibility to weathering in the Laubscher system.
6. The strong increase in influence of orientation of discontinuities in relation to the orientation of the walls and roof of underground excavations in the Laubscher system compared to the Bieniawski system.
7. The systems (except for Haines) for surface applications do not include the height of the slope whereas the height of the slope likely has an influence on the stability.

[23] Take for example, the link between Jr and Ja in the Barton system; the lowest value for Ja is 20 but this cannot be combined with the maximum value (5) for Jr but only with Jr = 1.

Table N-3. Rock mass parameters of interest for engineering structures in or on rock

<table>
<tr><td rowspan="12">Rock mass</td><td colspan="4">Intact rock strength</td></tr>
<tr><td rowspan="9">Discontinuities</td><td rowspan="4">rock block size and form</td><td colspan="2">orientation (with respect to engineering structure)</td></tr>
<tr><td colspan="2">amount of sets</td></tr>
<tr><td colspan="2">spacing per set</td></tr>
<tr><td colspan="2">persistence per set</td></tr>
<tr><td rowspan="5">shear strength along discontinuity (condition of disconti-nuity)</td><td rowspan="4">surface characteristics of discontinuity wall</td><td>material friction</td></tr>
<tr><td>roughness (dilatancy)</td></tr>
<tr><td>strength</td></tr>
<tr><td>deformation</td></tr>
<tr><td colspan="2">infill material</td></tr>
<tr><td colspan="4">Susceptibility to weathering</td></tr>
<tr><td colspan="4">Deformation parameters of intact rock/rock mass</td></tr>
<tr><td>Engineering structure</td><td colspan="4">Geometry of engineering structure (size and orientation of a tunnel, height and orientation of a slope, etc.)</td></tr>
<tr><td rowspan="2">External Influences</td><td colspan="4">Water pressure/flow, snow and ice, stress relief, external stress, etc.</td></tr>
<tr><td colspan="4">Type of excavation</td></tr>
</table>

Since the systems are based on back calculation (regression analysis) of case histories that are mostly unpublished, an exact determination of the origin of the differences cannot be given. In this respect, it should also be mentioned that empirical systems are never ‘final’. In the last two decades the systems have continuously developed. Experience with the systems and subsequent changes in or fine-tuning of weighting factors and parameters cause some of the differences between the systems. It is also likely that the added experience with classification systems makes the latest systems the most reliable. In this respect the decrease of the influence of water in some of the newer systems and, in particular, in systems focused on slope stability should be noted.

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

Table N-4. Parameters and their influence in existing classification systems

MAXIMUM NEGATIVE INFLUENCE OF PARAMETERS (in percentage from final maximum rating)(1)(2)																	
classification system(2)	rating range	intact rock strength	RQD	discontinuities								future weather-ing	pressure or load		excavation		method of excavation
				number of sets	persis-tence	spacing	aper-ture	roughness (scale)		infill	altera-tion walls		rock	water	dimen-sion	orien-tation	
								large	small								
EARLY SYSTEMS (for underground excavations)																	
Deere (RQD)	0 - 100		100														
Wickham (RSR)	19 - 120	24 (general area geology parameter)				35	7							7		11	17
RECENT SYSTEMS (for underground excavations)																	
Bieniawski (RMR)	0 - 100	15	20		6	20	6	6		6	6		15		100(4)	12	
		(reductions are not enough for a change of class)															
Barton(3) (Q)	0.00006 - 2666	with rock load parame-ter(3)	90 extr. good	97 very good				90 extr. good		99 good			97 very good	95 extr. good	100(4)		
Laubscher	0 - 120	17 (no change of class)	13(5)	21(5)(6) Good				5 (no change in class)	9	15	5	70 poor	40 good	3(7) (no change)	100(4)	37 good	20 (no change)
SLOPE SYSTEMS																	
Selby	0 - 100	20			7	30	7			*(8)		10(9)		6		20	
Bieniawski (RMR)	0 - 100	15	20		6	20	6	6		6	6		15			60	
Vecchia	0 - 100	88														12	
Robertson (RMR)(10)	0 - 100	30	20		6	20	6	6		6	6					(100) (10)	
Romana (SMR)	0 - 115	13	17		5	17	5	5		6	6		13			52	13
Haines	0 - 100	17	13(5)	21(5)(6)				5	9	15	5	70	40	3(7)	(note 11)		20

Notes:

1 Influence percentages are only an approximate indication. Some systems are combinations of adding/subtracting, multiplier/divider, and/or logarithmic parameters, not independent and/or non-linear parameters (see text). Influence percentage = (maximum final rating - rating with the parameter minimum and all other parameters maximum) / maximum final rating x 100 %. For the recent classification systems also the class is indicated that results if the particular parameter has its minimum value. This allows comparison of classes between the logarithmic scale of the Q-system and the linear scales of the Bieniawski and Laubscher systems.

2 Terzaghi, Lauffer and NATM systems are not included as they do not use a rating for different parameters.

3 Intersections and portals are not considered. Intact rock strength is only of influence if low compared to stress environment.

4 Graphical (approximately logarithmic) relations between roof span or hydraulic radius, final rating and stand-up time.

5 Laubscher's system. Parameters for RQD and discontinuity spacing can be replaced by discontinuity frequency.

6 Amount of discontinuity sets, spacing, and persistence combined in logarithmic relation.

7 Water influence combined with discontinuity ratings.

8 Infill combined with persistence.

9 Selby rates present degree of weathering (thus not future weathering) for the whole rock mass following BS 5930 (1981).

10 Robertson: If RMR < 40 points slope stability governed by the RMR rating; if RMR > 40 points the stability is fully governed by the orientation and strength of the discontinuities.

11 Haines: Final result from graph relating slope height, dip, safety factor, and (MRMR) rating. Adjustment parameter for slope orientation in relation with orientation of discontinuities with maximum of 100%.

N.7 Problems with parameters in existing rock mass classification systems

In the previous chapter it is shown that not all systems use the same parameters, that not all systems include all parameters thought to be important for geotechnical purposes and that the influence of a parameter on the final classification result is not the same for all systems. Apart from these differences, the implementation of some parameters can also be questioned. A further discussion of the parameters thought to be important for a classification system for geotechnical engineering is therefore necessary.

N.7.1 Intact rock strength

Intact rock strength is defined, in most classification systems, as the strength of the rock material between the discontinuities. Strength values used are often from laboratory unconfined compressive strength (UCS) tests. Problems caused by the definition of intact rock strength and using strength values based on UCS laboratory tests are:

1. The UCS includes discontinuity strength for rock masses with small discontinuity spacing. The UCS test sample is most often about 10 cm long and if the discontinuity spacing is less than 10 cm the core may include discontinuities[24].
2. Samples tested in the laboratory tend to be of better quality than the average rock because poor rock is often disregarded when drill cores or samples break (Laubscher, 1990), and cannot be tested.
3. The intact rock strength measured depends on the sample orientation if the intact rock exhibits anisotropy.
4. UCS is not a valid parameter because, in reality, most rock will be stressed under circumstances resembling conditions of triaxial tests rather than UCS test conditions.

Some classification systems (Franklin et al.) use the Point Load Test solely or as alternative, for UCS or hammer tests as index test for the intact rock strength. The same problems applying to using the UCS test also apply to the PLS test. The inclusion of discontinuities in the rock will cause a PLS value tested parallel to this discontinuity to be considerably lower than if tested perpendicular. This effect is stronger for the PLS test than for a UCS test, as the PLS test is a splitting test.

The size-strength system of Franklin et al., the Unified Rock mass Classification System (URCS), the slope stability system of Haines et al., the geomorphic rock mass strength classification of Selby, and the modified Hoek-Brown failure criterion (Hoek et al., 1992) allow for an estimate or ‘engineering guess’ of intact rock strength using ‘simple means’ (geological hammer, Schmidt hammer, scratching, breaking by hand, etc.). Although Laubscher also recognizes the problems inherent to testing of intact rock strength he actually does not explicitly allow for an ‘engineering guess’ with ‘simple means’.

The disadvantage of using a Schmidt hammer for estimation of intact rock strength is the influence of discontinuities behind the tested surface. Schmidt hammer values may be influenced by a large and unquantifiable loss of rebound if a discontinuity is present inside the rock behind the tested surface.

N.7.2 Rock Quality Designation (RQD)

Rock quality designation (RQD)[25] is defined as eq. [N-10] (Deere et al., 1967).

$$RQD = \frac{\sum \text{length of pieces of intact rock core with length} > 10\,\text{cm}}{\text{total length drilled}} * 100\% \qquad \text{[N-10]}$$

The RQD is measured on the borehole core. Normally the RQD is determined for every meter length of borehole core per lithostratigraphic unit. The length of unbroken pieces of sound core that are of more than 10 cm (4 inches) length along the center line of the core (ISRM, 1978b, 1981a), are added and the ratio, as percentage, to the length drilled is the RQD. Recommended is a drilled length of 1 or 1.5 m. In principle, the

[24] With discontinuities are denoted mechanical discontinuities, see glossary, page 223.

[25] RQD is used as an indicator for rock mass quality directly (ch. N.3), but also included in many classification systems together with other rock mass parameters. The discussion in this chapter considers the RQD only as a parameter in a rock mass classification system and not as an indicator for rock mass quality itself.

RQD is a very simple test and used worldwide. However, the definition of the RQD and the day-to-day practice of determining the RQD introduce several severe disadvantages those cause the RQD often to be inaccurate or to result in totally misleading values. Many authors have commented on the disadvantages of RQD measurements (R.D. Terzaghi, 1965). Some major problems with RQD measurements are:

1. The value of 10 cm (4 inches) unbroken rock is arbitrary.
2. The value of 10 cm for unbroken pieces of rock core is an abrupt boundary. A rock mass with discontinuity spacing of 9 cm perpendicular to the borehole axis will result in an RQD value of 0 % while a discontinuity spacing of 11 cm will result in an RQD of 100 %. Although a (small) quality difference might result from the difference in spacings, this is certainly not such a large difference that it should result in a difference between minimum and maximum of the quality assignment. Obviously in a real rock mass the spacings between discontinuities are not all the same and therefore the 10 cm boundary effect is more or less abrupt depending on the distribution of the spacings.
3. The RQD is biased through orientation with respect to discontinuity orientation (Fig. N-5 - compare vertical borehole to horizontal borehole A). If a discontinuity is in the borehole core parallel to the borehole (borehole B) then ISRM (1978b, 1981a) recommends measuring the length of the core offset from the center line if sound pieces of > 10 cm length are present in that stretch of the core. Depending on the infill thickness of the discontinuity, this might solve the problem of borehole B (RQD = 0 %) in Fig. N-5.

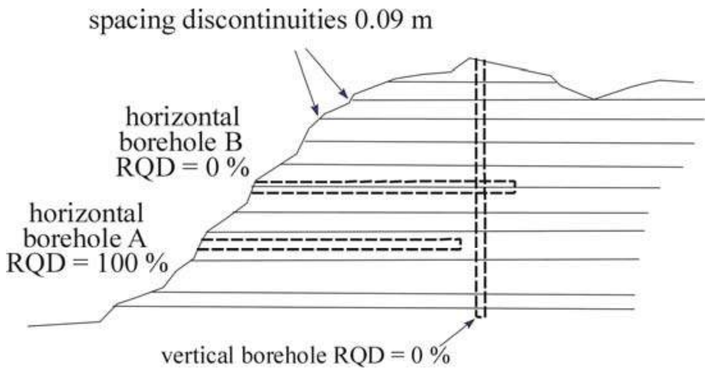


Fig. N-5. Bias of RQD due to orientation of borehole

4. Weak rock pieces (weathered pieces of rock or infill material) that are not sound should not be considered for determining the RQD (Deere et al., 1967, 1988). To exclude infill material will usually not be too difficult; however, excluding pieces of weathered, not sound rock is fairly arbitrary.
5. The RQD value is influenced by drilling equipment, drilling operators and core handling. Especially RQD values of weak rocks can be considerably reduced due to inexperienced operators or poor drilling equipment.
6. The equipment and especially the core barrels used for geotechnical rock drilling are not standard. It is obvious that the number of breaks caused by the drilling process will be strongly dependent on whether single-, double-, or triple-tube core barrels are used. ISRM recommends measuring RQD on cores drilled with a double-tube core barrel only. The borehole, however, is normally not only made to determine the RQD. Often triple-tube core barrels are used for weaker rock or fractured rock masses to obtain a decent core for test samples. The RQD measured on this core is overrated but the amount of overrating is not known. Alternatively, two boreholes should be drilled; one for the RQD with a double-tube core barrel and one for the samples with a triple-tube core barrel. The author does not know of any site where this has been the case. On the contrary the author has noticed many sites were the RQD was determined and compared from borehole to borehole irrespective of the core barrels used.
7. The diameter of the borehole core is not standard in geotechnical drilling. A core diameter of not less than 70 mm (H size) is recommended for geotechnical drilling. In massive rocks, however, a reduction is allowed to 55 mm (N size) and in very weak or fractured rock, the diameter should be increased between 100 and 150 mm (BS 5930, 1981). The author has noticed that in practice very often N or NQ sized boreholes (approximately 47 to 55 mm core diameter) are used independent of the quality of the rock. Bieniawski (1989) allows borehole diameters from BQ to PQ (36.5 to 85 mm) for RQD determination. A larger diameter will result in: 1) fewer breaks during drilling and core handling after drilling, 2) a larger chance that a parallel discontinuity is intersected and 3) a larger chance that pieces of sound rock will be present in the core if a (near-) parallel discontinuity is intersected. In general, smaller core diameters lead to lower values for the RQD and larger diameters to higher values for the RQD.

8. Pieces of rock that are clearly broken through drilling or transport are supposed to be fitted together and the length should be measured as unbroken (ISRM, 1978b, 1981a). If this is done properly it partly solves the problems mentioned in points 5, 6 and 7, however it is not always easy to distinguish between natural discontinuities and breaks from drilling or core handling. In particular, in a fresh rock mass this distinction is often almost impossible and a less experienced engineer or drilling master might make considerable errors.
9. Although the RQD should be established per lithology, many establish the RQD irrespective of the lithology. Partly because of inexperience, partly because lithological boundaries are often uncertain. This problem is emphasized if core loss occurs in interbedded lithologies where the weaker lithology is not present in the borehole core.

The above leads to the conclusion that the RQD is not very strictly defined, that the definition is not very logical, that the result may not express the rock mass quality and that comparison of RQD values might be deceptive. Thus the incorporation of the RQD in rock mass classification systems can be questioned.

In many classification systems the RQD is incorporated as a parameter while the classification system also contains a parameter for discontinuity spacing. This seems not very logical. It effectively doubles the influence of the spacing of discontinuities on the final rating.

RQD values determined without a borehole

Various methods have been proposed to determine the RQD value for situations where no borehole core is available. Palmstrøm (Barton, 1976a, Bieniawski, 1989, Palmstrøm, 1975) recommends measuring all discontinuities along a scanline on an exposure and to calculate the RQD following eq. [N-11].

$$
\begin{array}{c}
IF \;\; J_v \geq 4.5 \quad ===> \quad RQD = \left(115 - 3.3 * J_v \right)\% \\
IF \;\; J_v < 4.5 \quad ===> \quad RQD = 100\,\% \\
J_v = total\ number\ of\ discontinuities\ per\ m^3 \\
(= sum\ of\ number\ of\ discontinuities\ per\ metre \\
length\ of\ all\ discontinuity\ sets)
\end{array}
\qquad [N\text{-}11]
$$

A more sophisticated approach is a three-dimensional model to calculate the RQD from discontinuity spacing and orientation (Eissa et al., 1991, _en et al., 1991). The methods are vulnerable to criticism because 1) the relations are only approximate, 2) an exposure might show more discontinuities than a borehole in the same rock mass (certainly when the exposure has been created by blasting), 3) weak rock pieces (highly weathered pieces of rock or infill material) that should be excluded in the determination of RQD cannot be excluded in these theoretical models and 4) influences of drilling and core handling are completely excluded, whereas the RQD measured in a borehole is always influenced by the drilling and core handling. A more fundamental error might be caused by the orientation of the measurement. A borehole is nearly always vertical and a scanline nearly always horizontal. As classification systems are empirical, the orientation of the measurement might well have an influence although this is not quantified (or known) in the existing classification systems that use RQD.

N.7.3 Spacing of discontinuity sets

In many classification systems, the spacing of discontinuities is used as a parameter. However, often the spacing of only one discontinuity set can be incorporated (except for Laubscher and Franklin and modifications, and the 'modified Hoek-Brown failure criterion'). This is no problem if only one discontinuity set is present in the rock mass or if one discontinuity set has a considerably smaller spacing than the other discontinuity sets. The mechanical behavior of the rock mass with respect to discontinuity spacing is, in such rock masses, mainly governed by one discontinuity set. However, these classification systems do not describe what should be done if the mechanical behavior of the rock mass is governed by more than one discontinuity set, for example, if more sets with a similar discontinuity spacing are present.

N.7.4 Persistence of discontinuities

Non-persistent discontinuity sets do not have the same influence on the stability of a rock mass as persistent discontinuities have (ch. C.1). How to deal with persistence is described in detail in the Q-system (Barton et al., 1974, 1976a, 1988) and the geomorphic rock mass strength classification of Selby (1980, 1982). These

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

systems combine persistence with the description of the shear friction parameters of the discontinuity. In the RMR and Laubscher systems and modifications discontinuities are only considered if: 1) the discontinuity is larger than visible; thus the discontinuity can be followed for a distance equal to or larger than, for example, the dimensions of a tunnel or exposure, or 2) the discontinuity abuts against another discontinuity. Discontinuities that do not comply with 1 or 2 are not considered as discontinuities in these classification systems.

N.7.5 Condition of discontinuities

The condition of the discontinuities (material friction, roughness, discontinuity wall strength and infill material) determines the shear and tensile strength characteristics of the discontinuities. It is a problematic parameter in all-existing systems that use the condition of discontinuities. Most systems separate the condition of discontinuities in different parameters (for example: Barton, Bieniawski, Laubscher and modifications) that are independently rated in the classification system. The Laubscher system uses four parameters (large and small scale roughness, alteration of discontinuity walls and infill), to establish the quality of the discontinuity. The Barton system uses only two parameters (discontinuity roughness number and discontinuity alteration number), but the number of options for these parameters is so large that most discontinuity conditions can be described.

A major problem with the existing systems is that these use an expression for the condition of the discontinuities for one discontinuity set only. Obviously there is no problem if all discontinuity sets have the same characteristic condition but for a rock mass with discontinuity sets with different characteristics it is often difficult to decide which discontinuity set should be considered in the determination of the rock mass quality. Some authors (Bieniawski, 1989, Barton, 1976a, Laubscher, 1990 and modifications) indicate that: 1) the condition of the discontinuity set with the poorest condition should be included or 2) the condition of the discontinuity set that has the most adverse influence on the rock mass quality or engineering application should be included. Romana (1985, 1991) recommends that the rating should be calculated for each discontinuity set and the lowest resulting rating be used to determine the slope stability.

In Bieniawski (RMR) and modifications and the slope classification by Romana the problem is more pronounced because also the spacing parameter is defined for one discontinuity set only. According to Bieniawski the discontinuity set with the most adverse influence on the stability should be taken into account. A discontinuity set with a large spacing but with a bad condition could, however, have a worse influence on stability than a discontinuity set with a small spacing but with a good condition. It is not clear how the worst discontinuity set should be selected in such a situation. The problem is illustrated in Fig. N-6.

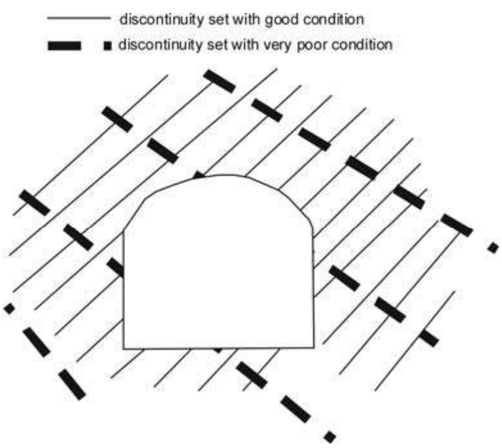


Fig. N-6. Influence of discontinuity condition. It is not clear which discontinuity set has the worst influence on the stability of the tunnel

N.7.6 Anisotropic discontinuity roughness

The roughness of a discontinuity can be anisotropic, e.g. ripple marks, striations, etc.. The shear strength resulting from anisotropic discontinuity roughness will also be anisotropic. Thus roughness should be assessed in relation with the orientation of the discontinuity and the roughness used in a classification system should be the roughness in the direction that is most important for the stability of a slope.

None of the existing classification systems incorporates anisotropic roughness. Robertson (1988) and Hack (1998) recommends assessing the roughness in the direction where possible sliding can occur. Systems that do not include the influence of discontinuity and excavation orientation can obviously also not include anisotropic roughness.

N.7.7 Discontinuity karst features

Karst features have been found to be of importance in slope and excavation stability. The open holes considerably weaken the rock mass. Karst features are nearly always found to originate from solution along discontinuities. Solution leaves cavities supported by points of contact across opened discontinuities. A diminished contact area reduces the shear strength if (apparent) cohesion is present, and points of contact may break due to overstressing. The presence of karst holes during excavation has also an adverse effect on the slope stability. During blasting the blasting gasses will force their way out of the rock mass via the karstic discontinuities rather than by breaking intact rock or by following discontinuities in the direction of the next borehole. Only the SSPC system incorporates a parameter that allows for an influence of karst features.

N.7.8 Susceptibility to weathering

Susceptibility to weathering is only considered, to a certain extent, in the classification system by Laubscher (1990) and in the modifications of this classification system. Susceptibility to weathering is an important factor in slope stability. Within the life span of a civil engineering structure future weathering of discontinuities and rock material may well lead to instability.

N.7.9 Deformation of intact rock and rock mass, stress relief

Deformation of intact rock is not considered in any of the existing systems, however, it is used for an indirect estimation of the intact rock strength by impact methods. Deformation of intact rock is likely not important for engineering structures which cause low stresses on the rock, e.g. slopes of relatively small heights. Deformation of a rock mass is considered in the Q-system (e.g. Barton et al., 1974, 1976a, 1988) in relation to stress relief due to weak or sheared zones in the rock mass. Deformation of a rock mass in relation to stress relief, not particularly related to weak or sheared zones, may, however, be of importance for slopes. Stress relief and related deformation may cause movements along discontinuities, increase of slope dips, etc., which influence the stability of a slope. A problem with deformation of a rock mass and with stress relief is that these cannot be tested, otherwise than with costly tests.

N.7.10 Relative orientation of slope and discontinuities

The orientation of discontinuities in relation with the orientation of the slope (and often also of an underground excavation) has a marked and often decisive effect on the stability of a slope (sliding, toppling failure, etc.). However, not all classification systems used for slope stability assessment incorporate a parameter that allows for this influence (for example, Robertson, 1988 for an RMR of less than 40). In the other systems the parameter is fairly crude or not fully decisive or both. For example Bieniawski allows for a reduction of the final RMR rating by 60 % if the slope is unfavorably oriented, and Romana allows a reduction of 52 %. In some systems (for example, Bieniawski and Romana) only the major discontinuity set or the discontinuity set with the most adverse influence on the slope stability has an influence on the final ratings, with respect to orientation of discontinuities and slope. This results in the same problem as outlined above for the condition of the discontinuity.

N.7.11 Slope height

The height of the slope has a direct influence on the stress levels in the rock mass of the slope. High stress levels, comparatively to the intact rock strength, may cause failure of the slope due to intact rock failure (Gama, 1989). A high slope may also present more opportunities for discontinuity related failure as the quantity of discontinuities intersected by the slope is larger. Hence, although slope height is likely to be of importance in a slope stability system, only the SSPC system, Haines, and Shuk incorporate the slope height.

N.7.12 Water

The presence, or the pressure of water in discontinuities, is a parameter incorporated in most systems. Water pressures and water flow in discontinuities may exercise pressures on rock blocks. The shear strength along discontinuities is unfavorably influenced because water pressure reduces the normal pressure on the discontinuity and therefore reduces the shear strength, while the presence of water may lower the shear strength of the infill material and of the discontinuity wall. Weathering of discontinuities through the passage of water can also strongly reduce the shear strength.

The incorporation of a ‘water’ parameter in classification systems to allow for an influence of water pressure on the stability of an engineering structure is questionable for the following reasons:

1. Establishing the value for a parameter for the influence of water determined by the amount of water flowing out of the rock mass can cause some problems. Mostly they are defined by a certain quantity of water flowing out of the rock mass per time unit over a certain length of tunnel. Discontinuities in virtually all rock masses will be the major conduits for water discharge. In the classification systems, the size or the form of the tunnel is not considered in relation with the parameter for water, whereas it can easily be seen that the number of water discharging discontinuities and thus the quantity of water discharged is dependent on the form and size of the tunnel.
2. An important shortcoming in the existing water class determination is that the quantity of water is not necessarily related to the pressure of the water in the discontinuities. A small quantity of water discharged by a low permeability rock mass might be related to a higher water pressure in the discontinuities than a large quantity of water discharged by a (free draining) rock mass with high permeability.
3. The discharge of water is often not constant over the slope height. In the rock mass of the lower part of the slope the water pressure and consequently water discharge will be higher than in the higher part of a slope. Whether an average of the water discharged should be used in a single classification or whether this should lead to two or more different classifications applicable to different levels of the slope is not described in the existing slope stability classification systems.
4. In underground excavations, the stress configuration around the opening will generally result in a higher compressive stress on discontinuities perpendicular to the wall of the opening and near to the underground opening than the compressive stress on discontinuities further away from the opening. Higher compressive stress causes a closing of the discontinuities in the direction of the underground opening. Water pressures are therefore present in the discontinuities adjacent to the opening. In slopes stress relief causes the discontinuities nearest to the slope face to open and the storage capacity increases in the direction of the slope face, resulting in a decrease of water pressures. The pressure decrease in the direction of a slope face can be large; in most slopes the discontinuities at the slope surface are free draining. This difference in water pressures between underground openings and slopes is likely to cause that water should be treated in a different way in slope than in underground excavation classification systems[26].
5. It has been shown that the water flow through discontinuities is often restricted to channels in the discontinuity (Abelin et al., 1990, Bear et al., 1993, Genske et al., 1995, Hakami, 1995, Neretnieks et al., 1982, 1985, Rasmussen et al., 1987). Probably this can be extended to water pressures. Water pressure acting on a plane only at the location of a channel would result in a total water pressure on the plane considerably smaller than if the water pressure would act over the full discontinuity plane[27].
6. Water run-off over the slope can lead to instability, but such run-off is not related to water seepage.
7. Water presence in slopes is not a continuous feature in time. During and shortly after rain high water pressures may build up in a slope or, alternatively, there may be no water at all after a dry period.
8. During rain it will be virtually impossible to distinguish between water discharged by discontinuities in the rock mass of the slope and surface run-off water over the slope.
9. Drains will normally be present in a wet tunnel, in which the quantity of water flowing in and out a section can be simply measured with, for example, a weir. The difference between the quantities of water flowing in and out of the section is the amount of water discharged by the rock mass surrounding the tunnel. Slopes, however, will usually not have a drain at the toe and measuring the quantity of water will be a practical problem.
10. In the existing classification systems for underground excavations the water parameter is normally expressed in classes such as: ‘dry’, ‘moist’, ‘dripping’, ‘wet’ or in classes that are directly related to an amount of water flowing out of the rock mass into the excavation. Classes such as ‘dry’ and ‘moist’ are not very difficult to establish but classes such as ‘dripping’ or ‘wet’ are subjective.

[26] This applies to flowing - dynamic - water; the water pressures of static water are independent of the storage capacity. The slope face and often also the walls, roof and floor in an underground excavation, are, however, always free draining, except if the rock mass is covered by an impermeable material, such as shotcrete, without draining facilities, and an underground opening mostly, and thus there is a flow of water in the direction of the slope face or underground opening.

[27] Water flow may be restricted to channels while the whole discontinuity is filled by static, not flowing, water, then the water pressure still acts over the whole surface of the discontinuity. In underground excavations has, however, been found that in some rock masses the majority of the discontinuities is not water bearing while the rock mass is water bearing (Neretnieks et al., 1985).

The above leads to the conclusion that the methodology used in the existing classification systems that incorporate the influence of water pressures on the mechanical behavior of a rock mass, should be reconsidered.

N.7.13 Ice and snow influence

Ice and snow can have a severe influence on the stability of a slope. Freezing of water leads to an expansion in volume. Water frozen in a discontinuity will exert a very high pressure on the discontinuity walls. In underground applications, this virtually will never be a problem as temperatures underground are normally not below zero. In surface applications and certainly in slope stability applications freezing of water in discontinuities can, however, be a major factor for the stability of a slope. Freezing of water may lead to opening and widening of discontinuities, displacements of rock blocks out of the slope face, but also to closure of discontinuities, blocking the discharge of seepage water that may lead to water pressure build-up in the slope. Snow may cause a problem for slope stability because of the additional weight of snow on the slope face. The influence of ice and snow is also dependent on the orientation of the slope with respect to the direction of the sun as daily temperature changes, especially a regular variation between freezing and thawing, has a negative influence on the quality of the rock mass. The problem of ice and snow influence is not addressed in any of the systems for slope stability.

N.7.14 Method of excavation

The way the exposure has been established has a considerable influence on the parameters measured or observed in the exposure. For example, an exposure in a riverbed created by slow scouring of the river over probably hundreds to thousands of years creates an exposure with a relatively small amount of visible discontinuities. Stress concentrations have not occurred or were minimal during the creation of the exposure due to the slow process. The tendency for discontinuities to open is minimal and therefore a larger part of the discontinuities is not clearly visible. Contrariwise a blasted excavation shows considerably more discontinuities because partly intact rock has been cracked due to the blasting, but also, and often more important, existing internal planes of incipient weakness, which before blasting were not visible, have opened or widened due to the pressure of the blasting gasses and the shock wave, and therefore become visible and thus will be measured as mechanical discontinuities.

Some existing classification systems consider this effect (Haines, Laubscher, Romana, Wickham, and Hack). These systems reduce the rock mass rating with a parameter to compensate for the damage that will be caused by the method of excavation.

N.7.15 Seismic velocity in a discontinuous rock mass

Some systems include seismic parameters, usually the velocity or apparent velocity of the wave, to assess the quality of the rock or rock mass (Japan, 1992, Weaver, 1975). For rippability, excavation and blasting assessment this is a fairly standard procedure, but assessments are often specific for types and brands of (excavation) equipment, for blasting procedures or for types and brands of explosives. In excavation or blasting assessment the interpretation is in general simpler than for other applications. The influence of intact rock strength and spacing and orientation of discontinuities (the main rock mass parameters defining excavatability) on seismic waves is comparatively straightforward. To relate seismic velocities to other rock mass or discontinuity parameters (for example, shear strength) is far more complicated. The behavior of a seismic wave in a rock mass and the relationships between the rock mass parameters and the seismic parameters are not known in all details and consequently the interpretation is often ambiguous (Cervantes, 1995, Hack et al., 1982, 1990).

N.7.16 Operator experience and familiarity with a classification

Assigning values to some of the parameters in the systems discussed is often subjective and depends upon the operator’s experience and the familiarity of the operator with the system. Examples for which this is of major importance are: ‘the discontinuity set with the most adverse influence on the rock mass or for the engineering application’ and classes such as ‘wet’, ‘dripping’ for water influence. The merits of a system are clearly reduced if a system depends on the operator’s experience or familiarity with the system.

O NUMERICAL CALCULATION

O.1 Introduction

Numerical calculation programs for rock mechanics have been around for some time. The basic principle is straightforward; a computer program simulates the mechanical behavior of the rock mass and the engineering structure on or in the rock mass. There are two approaches possible: 1) a discontinuous model that models the rock mass including all discontinuities, and 2) a continuum model that simplifies the rock mass by using larger blocks of material that include discontinuities. The problem with a discontinuous model is that too many items are involved for any larger size rock mass (too many different blocks, with too many discontinuities). If the rock mass, or part of the rock mass is modeled as a continuum the mechanical behavior of the blocks is very difficult or impossible to define. This chapter does not include basics on numerical modeling, for this is referred to the appropriate literature.

Continuum calculations for engineering structures in or on a rock mass cannot be appropriate, as the simplifications needed to present the rock mass as a continuum are so substantial that it is nearly always highly questionable to what extent the final calculation model still represents reality. Discontinuous ‘distinct block’ numerical calculations can model the discontinuities and calculate the behavior of a rock mass in all detail, provided that property data are available. Apart from the need to have powerful computers to do the large number of calculations required by the vast quantity of discontinuities, the test data needed for a detailed numerical discontinuous calculation are never available. An often-applied practice to avoid these problems is to simplify the discontinuity model, and estimate or guess the properties or to use literature values. To what extent the result is still representative for the real situation is a question that often remains unanswered. Analytical or numerical calculations should be performed in three dimensions because discontinuities usually make a rock mass three-dimensionally anisotropic. Calculations are, however, usually in two dimensions because of the amount of data needed and the number of calculations required for three-dimensional analyses.

Alternatively, numerical methods can be used not as a deterministic method but to produce sensitivity analyses that will give the most likely and worst-case scenarios for a rock mass calculation. This, however, may result in a colossal quantity of calculations. The same applies to the various methods of stochastic calculations incorporated in analytical or numerical calculations. The near infinite number of parameters for which values and distributions of values are not or only partly known, prohibits acceptable and fast calculations

O.2 Parameters to simulate continuum rock

O.2.1 Stress-strain of a block including discontinuities

The stress-strain relationships for a block of rock including discontinuities can be defined a s follows for a block of rock including one set of discontinuities (Fig. O-1). The simulated elastic modulus of the block with discontinuities is defined as:

$$\frac{1}{E_n} = \frac{1}{E} + \frac{1}{k_n * S} \qquad \text{[O-1]}$$

E_n = simulated elastic modulus in direction n $\quad E$ = intact rock elastic modulus

k_n = normal stiffness of discontinuity $\quad S$ = distance between discontinuities

E_n is the simulated elastic modulus perpendicular to the discontinuities. The elastic modulus parallel to the discontinuities is equal to E; the intact rock elastic modulus. The Poisson's ratio for the strain in direction n as result of a stress in direction v equals (if the deformation of the discontinuities in direction n can be neglected):

$$\begin{aligned} \nu_{tn} &= \nu \\ \nu &= \text{Poisson's ratio of intact rock} \\ \nu_{tn} &= \text{strain in direction } n \text{ by} \\ & \text{normal stress in direction } t \end{aligned} \qquad [O\text{-}2]$$

The Poisson's ratio for the strain in direction t because of a stress in direction n is:

$$\begin{aligned} \nu_{nt} &= \frac{E_n}{E} * \nu \\ \nu &= \text{Poisson's ratio of intact rock} \\ E_n &= \text{simulated elastic modulus} \\ & \text{in direction } n \\ \nu_{nt} &= \text{strain in direction } t \text{ by} \\ & \text{normal stress in direction } n \end{aligned} \qquad [O\text{-}3]$$

The simulated shear modulus of the block is:

$$\begin{aligned} \frac{1}{G_{nt}} &= \frac{1}{G} + \frac{1}{k_s * S} \\ G_{nt} &= \text{simulated shear modulus} \\ G &= \text{intact rock shear modulus} \\ k_s &= \text{shear stiffness of discontinuity} \\ S &= \text{distance between discontinuities} \end{aligned} \qquad [O\text{-}4]$$

The simulation of stress-strain moduli for a rock mass with more than one discontinuity set is difficult and cannot be easily done analytically.

O.2.2 Strength and failure of a block of rock including discontinuities

The strength and failure criteria of rock mass including discontinuities are highly dependent on the discontinuities. The influence of discontinuities on strength has been shown before. The only feasible methodology to establish the strength failure criteria for a rock mass is based on classification. Bieniawski's RMR and Hack's SSPC define friction and cohesion parameters to be used in the Mohr-Coulomb failure criterion. Hoek-Brown modified failure criterion defines a non-linear failure criterion for rock masses.

Although the statement above is true occasionally, it may be possible for some rock masses to define a rock mass strength criterion in which discontinuities are not explicitly modeled. The criteria may be based on laboratory and field-testing, back calculations of similar case histories, and on experience. Fig. O-2 shows a failure criterion for a rock mass of good quality gneiss (Hoek et al., 1980) based

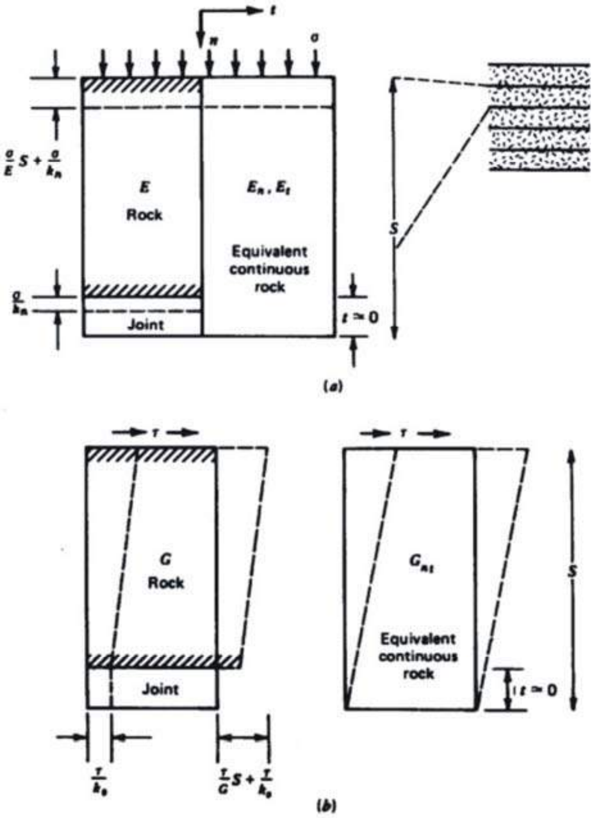


Fig. O-1. Representation of a regularly jointed rock by an "equivalent" transversely isotropic material (after Goodman, 1980)

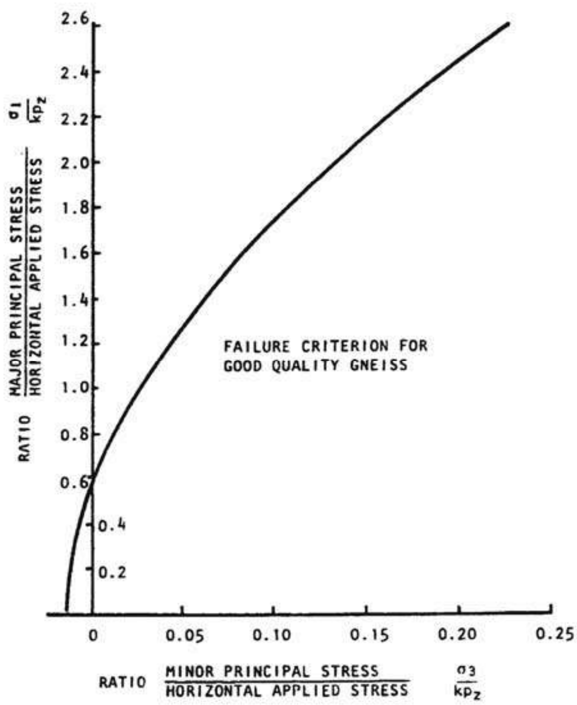


Fig. O-2. Failure criterion for good quality gneiss expressed in terms of the ratios of principal stresses to applied horizontal stress kp_z (after Hoek et al., 1980) on this principle. Note that the failure criterion is modeled isotropic.

O.3 Examples of continuum calculations

O.3.1 Stress failure of tunnel wall

Fig. O-3 shows the strength/stress ratio contours around a tunnel. The gray zone bordering the tunnel wall indicates that the stress exceeds the strength of the rock. For the modeling a continuum, numerical model has been used. The strength of the rock is calculated following a criterion similar to that shown in Fig. O-3.

O.3.2 Arching around three parallel tunnels

Fig. O-4 shows a continuum calculation of the stress around three parallel tunnels. In the right figure are indicated the strength to stress ratios. Note the stress relief in-between the tunnels just above and below the center line between the tunnels. The combined ‘arch’ effects of the three tunnels form an elliptic arch around all tunnels together. In-between the tunnels the stresses are relatively relieved. Note also that along the tunnel wall the strength/stress ratio will be below 1 and failure is likely of the rock at the tunnel wall.

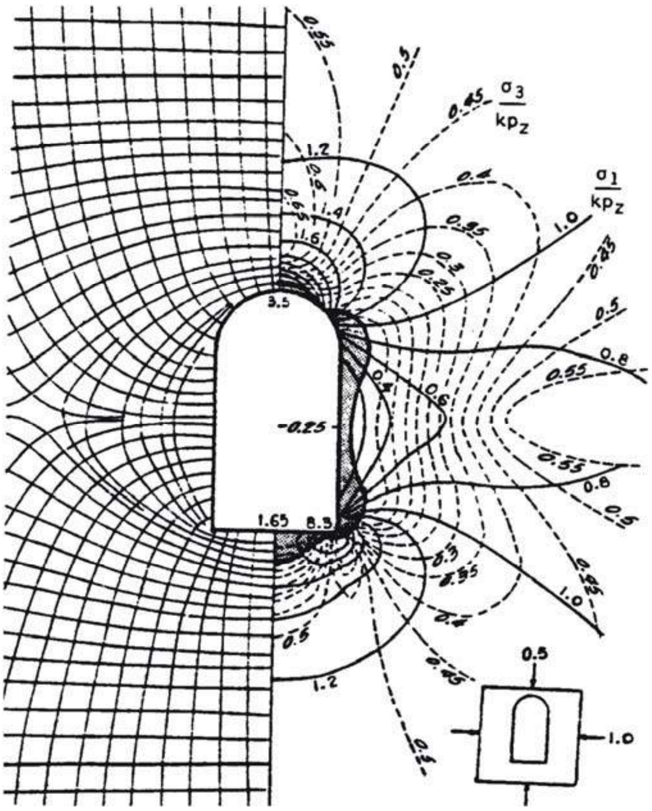


Fig. O-3. Principal stress trajectories (left) and Strength/stress ratio contours (right) around a tunnel (after Hoek et al., 1980)

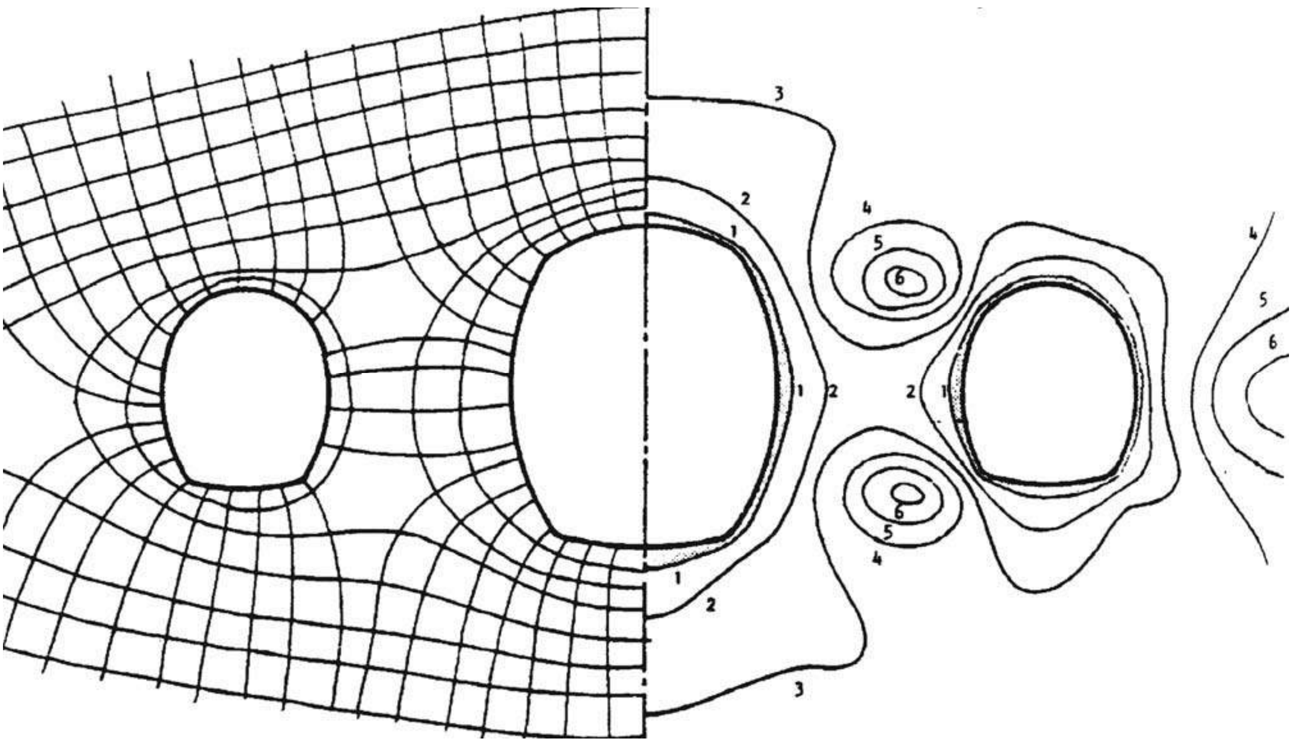


Fig. O-4. Principal stress trajectories (left) and strength/stress contours (right) around three parallel tunnels (after Hoek et al., 1980)

O.4 Examples of discontinuous calculations

Three examples are given of discontinuous calculations done with UDEC (UDEC, 1996). Note the difference in behavior with the continuum calculations before. Blocks of intact rock can move and be fully detached from neighboring blocks with result open space in the rock mass and loss of shear strength along discontinuities. In continuum, calculations open space does not exist, other than at pre-defined locations. The last example shows a comparison between analytical, classification, and discontinuous numerical assessment of a slope.

O.4.1 Rock bolt as tunnel support

Fig. O-5, Fig. O-6, and Fig. O-7 show how a rock bolt supports an instable roof.

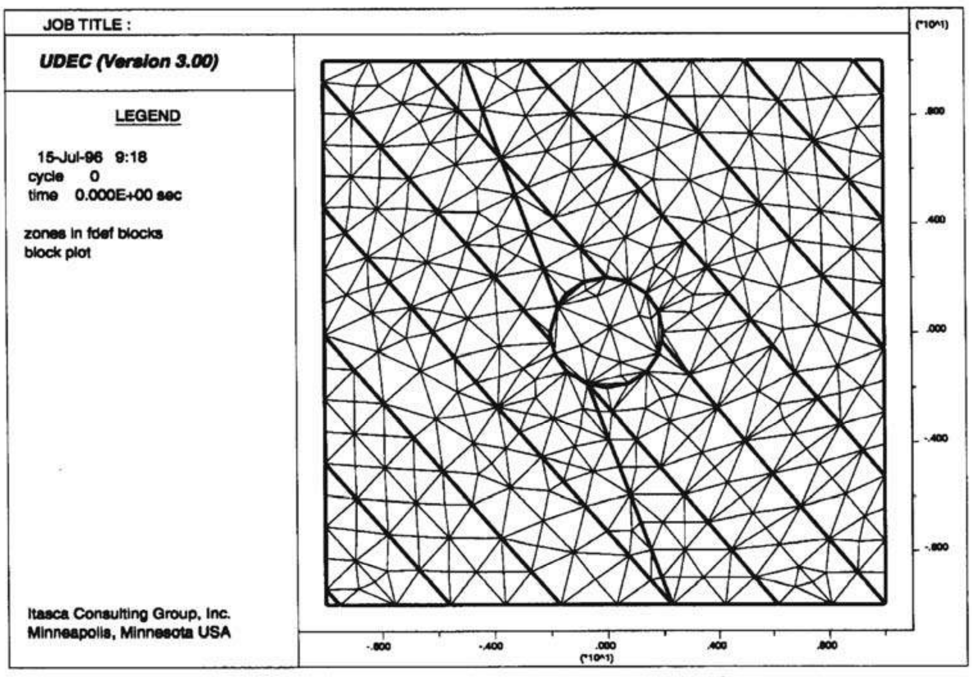


Fig. O-5. Tunnel in rock mass with fault and discontinuity set (after UDEC, 1996)

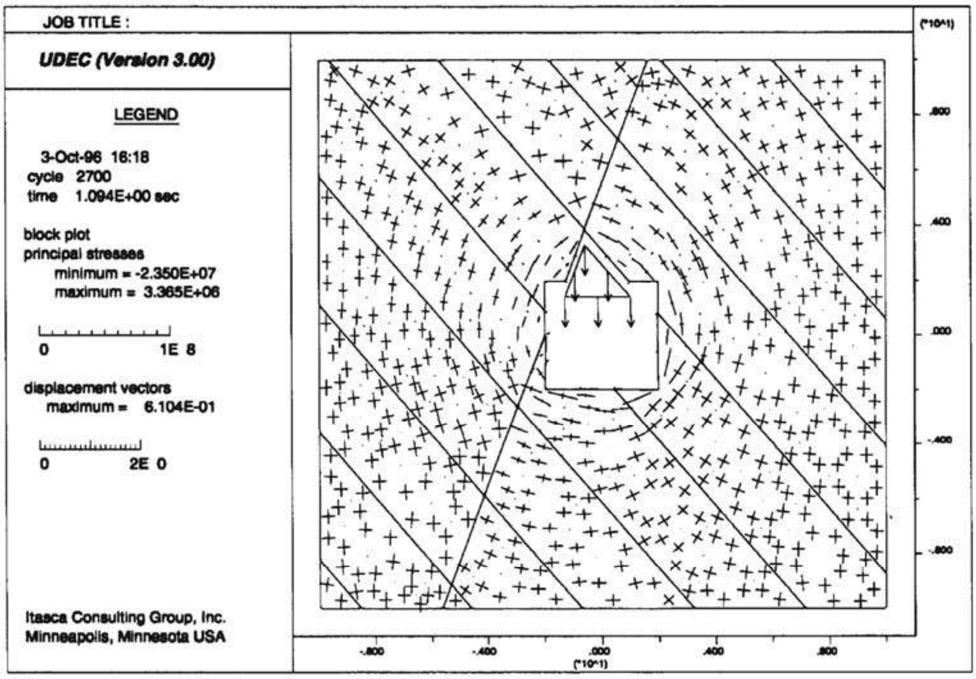


Fig. O-6. Roof instability (after UDEC, 1996)

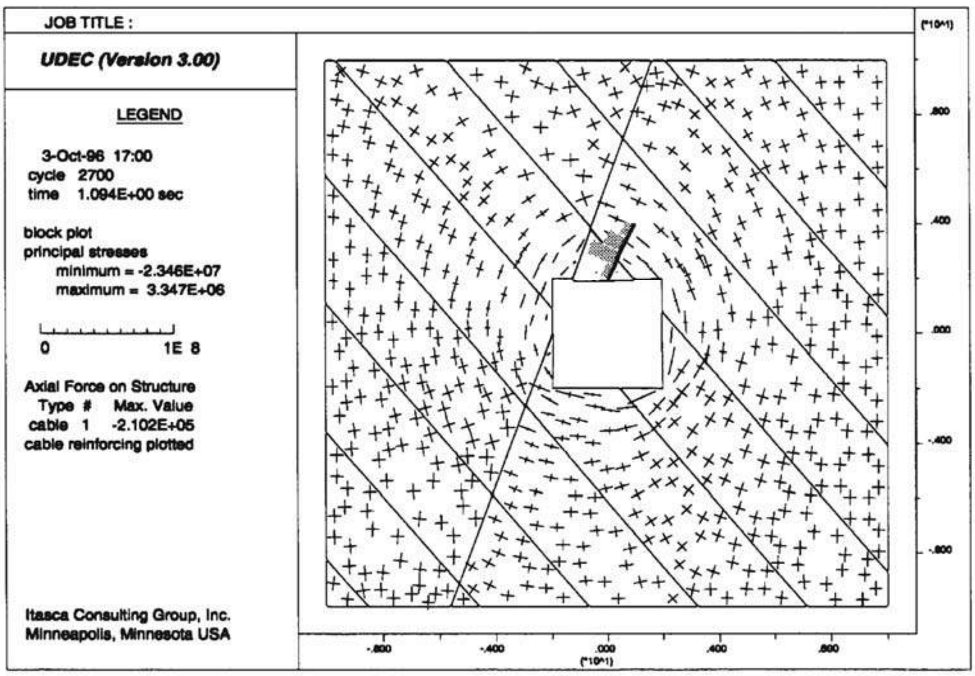


Fig. O-7. Rock bolt to stabilize block (after UDEC, 1996)

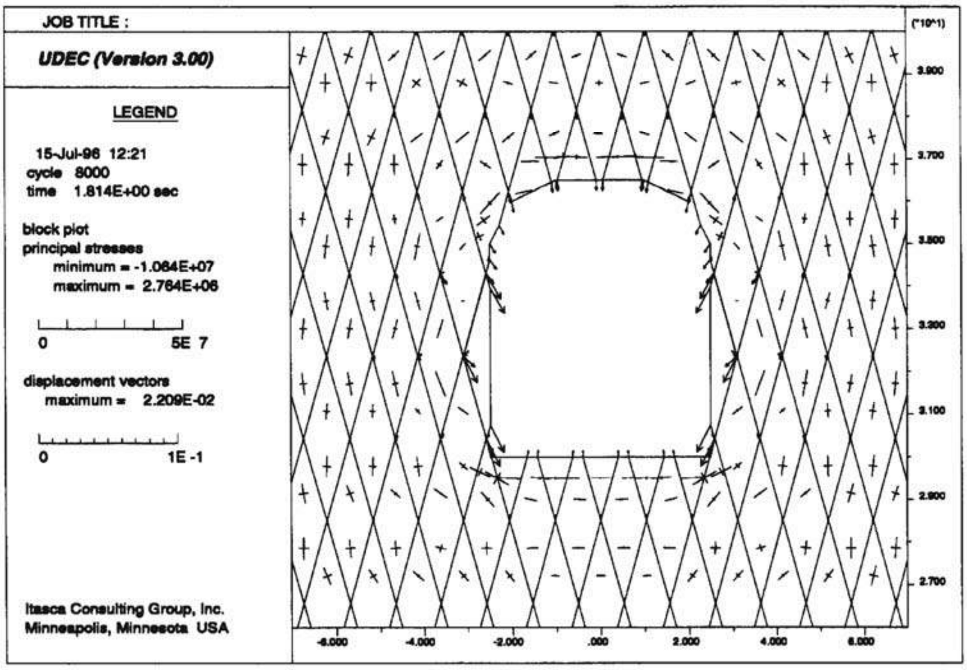


Fig. O-8. Stress-strain equilibrium after excavating a tunnel (after UDEC, 1996)

O.4.2 Tunnel under seismic loading

Fig. O-8 and Fig. O-9 show how a stable tunnel can become instable during seismic (earthquake) loading. The vibrations from the earthquake cause that the contact between discontinuity walls is lost. This causes that the shear strength along the discontinuities is lost and the blocks will fall. The blocks in the floor move upward due to the earthquake vibrations.

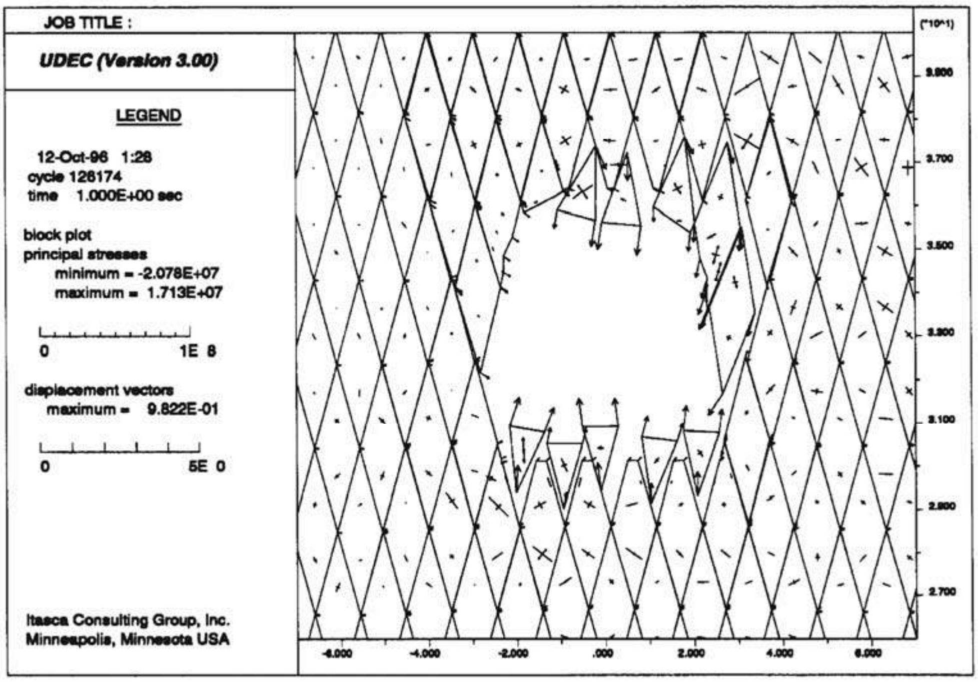


Fig. O-9. Instability due to seismic loading (after UDEC, 1996)

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

O.4.3 Plane sliding failure in a 40 year old slope in Upper Muschelkalk

This example shows the application of the SSPC system to predict the stability of an existing slope, and the usefulness of analytical and numerical computer calculations compared to using the SSPC system. The 40-year-old road cut is situated in Upper Muschelkalk (Tg23) limestone and dolomite at about 2.5 km from Marçá along the road from Marçá to La Torre de Fontaubella.

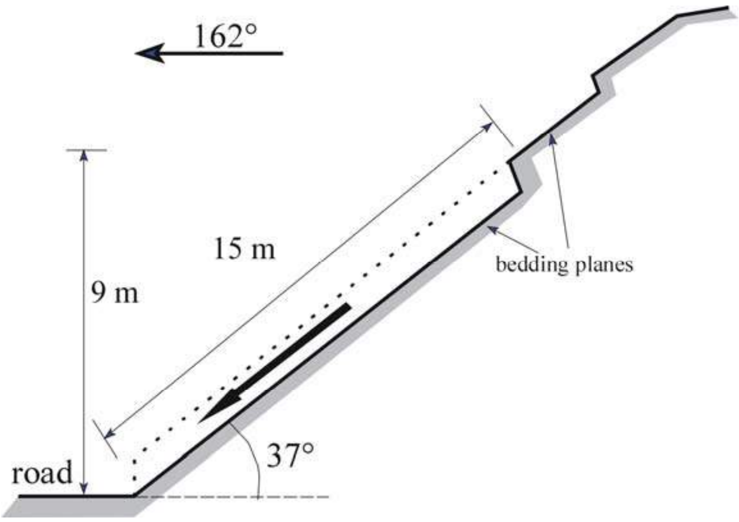


Fig. O-10. Geometrical cross section of the slope

The slope (photo: Fig. A-2; cross-section: Fig. O-10) is cut in Upper Muschelkalk light gray calcisiltite (Tg23), medium bedded, very widely jointed, slightly weathered, strong, and impermeable except along the joints and bedding planes. The bedding (dip-direction/dip = 162/37) strikes parallel to the slope of the terrain (dip slope) and dips 36 - 37° towards the road excavation. Two joint sets are present. One set is vertical (265/85) and strikes approximately perpendicular to the road cut. A second joint set (337/48) is striking parallel to the slope face and bedding, but dips ≈ 50° against the slope face direction and thus approximately perpendicular to the bedding plane. The spacing of this discontinuity set is approximately 15 m. However, this discontinuity set showed a far smaller spacing of about 5 m in parts of the slope directly below the part that had slid and in parts directly adjacent to the sliding plane. This is likely due to the slope geometry that caused existing joints to open and new cracks to form because of tensile stresses. These additional joints with orientation 337/48 (at a spacing of about 5 m) are further called ‘internal joints’. The bedding plane and the vertical jointing contain some clay infill. The clay in the bedding planes is likely to result from weathering as the bedding planes often contain some minor contents of clay. Clay infill in the vertical discontinuity set (265/85) is likely topsoil flushed in from the terrain surface above. Some karstic solution was observed along the vertical discontinuity set.

The slope has originally been cut at an angle of 80 - 90° at least some 40 years ago. The slope was stable until April 1990 but already showed the ‘internal joints’, likely caused by tensile stresses, and it was found to have failed in March 1991. Because the slope is such a good example of plane sliding, the slope has been investigated in more detail. Samples for UCS and shearbox testing have been sawn out of the rock and a detailed survey of the topography of the slope has been executed. The slope has also been subject to analytical and numerical modeling with the distinct element method (UDEC, 1993). Numerical modeling was possible because the slope could be modeled for the two-dimensional UDEC program without too many simplifications (Cindarto, 1992).

O.4.3.1 Slope stability by classification

SSPC classification has been done on the slope. The exposure characterization has been done in 1990 before the slope failed. The slope calculation shows that for a slope dip (road cut) of 90° the stability probability for sliding along the bedding plane was 55 % before the sliding happened. This indicates that the slope stability against sliding was almost unity, and for example, a very slight decrease of the condition of the bedding plane due to weathering was sufficient to cause failure. The fact that tension cracks developed during the lifetime of the slope also indicates that the slid block above the bedding plane was not fully supported by shear strength along the lower parts of the bedding plane. The clay infill in the vertical joint set (265/85) has not been included in the calculations of the reference rock mass and in the slope stability probability, as the infill was expected to be flushed into the discontinuities from the terrain surface and not to be present deeper in the rock mass. Whether the spacing of the second joint set (337/48) is taken as 15 or 5 m does not make a difference for the calculation of the reference rock mass nor for the probability of the slope stability.

O.4.3.2 Laboratory tests

Shear tests samples of the discontinuity planes have been done with the Golder shearbox (Hencher et al., 1989). Samples have been obtained from the debris of the failed slope and have been sawn out of still standing parts of the road cut. The samples from the debris were used for shear tests on non-fitting surfaces whereas the samples sawn out of the rock-mass were used to test fitting discontinuity surfaces. Only samples could be tested which did not contain steps. No significant differences were found between tests on the bedding planes and on the other discontinuities. The shearbox friction angle from these tests is 45° (this is the average of six tests which are not corrected for dilatancy, standard deviation 1°). The clay infill on the bedding surface as observed in the field has not been present on the surfaces of the samples for testing. For the debris samples this is obvious but also for the sawn samples the clay infill (which is very thin; 1 - 2 mm) was lost during the sawing and preparation of the sample.

The laboratory shearbox friction values for the bedding plane are representative for a rough planar surface (the sample with steps could not be tested) without infill and a large-scale roughness equal to straight. This results in a friction angle of about 43° according to the 'sliding criterion'[28]. The description of the bedding plane in the field is, however, straight, rough stepped with fine soft sheared infill and equivalent to about a 'sliding angle' of 35° along the plane ('sliding criterion')[28]. The value from the laboratory shearbox test of 45° is thus in agreement with the sliding criterion for the sample tested, however, is not representative for the bedding plane in reality. That the difference between the test result and reality is not larger is pure coincidence. The absence of steps on the surface of the samples is compensated by the absence of the infill material in the laboratory tests. This illustrates the limited usefulness of shearbox testing, even for discontinuities that have no large-scale roughness.

O.4.3.3 Slope stability by limiting-equilibrium back calculation

A traditional limiting-equilibrium back analysis was made of the slope of example II (Fig. O-11). The cohesion along the sliding plane is taken as zero. The length of the sliding block is defined by the second joint set (337/48) approximately perpendicular to the failure plane, the so-called 'internal joint'. In the calculations, the spacing of this joint set and thus the length of the sliding block are varied between 3 and 15 m.

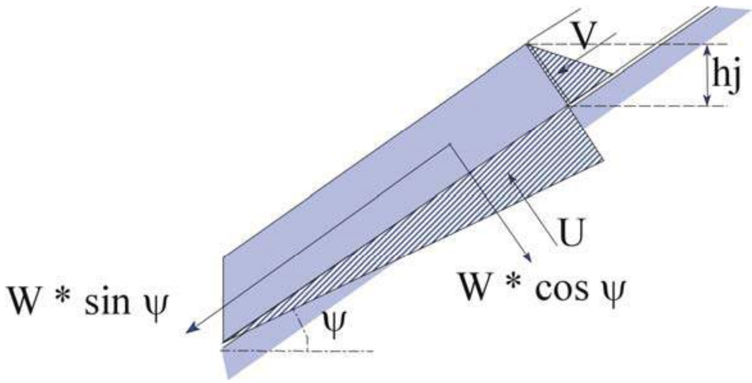


Fig. O-11. Limiting-equilibrium analysis

Whether the failure occurred under the influence of water pressures in the discontinuities was also investigated. Three different levels of water in the 'internal joint' were used in the calculation: h_w = 100 %, 50 % and 25 % (h_w = the height of the water as percentage of h_j, the height of the joint above the bedding plane). The friction angle along the sliding plane is calculated with:

$$\varphi = \arctan\left(\frac{W \sin\psi + V}{W \cos\psi - U}\right) \qquad \text{[O-5]}$$

φ = friction along sliding plane W = weight of block ψ = dip of sliding plane

U = water force at bottom of block V = water force at rear of block

[28] Condition of discontinuity for the laboratory samples: TC = 0.75 (straight) * 0.65 (rough planar) * 1.00 (no infill) * 1.00 (no karst) = 0.49. 'Sliding criterion': = TC / 0.0113 = 43°.

Condition of discontinuity for the bedding plane in the field: TC = 0.75 (straight) * 0.95 (rough stepped) * 0.55 (fine soft sheared infill) * 1.00 (no karst) = 0.39. 'Sliding criterion': = TC / 0.0113 = 35°.

Fig. O-12 shows the relation between the length of the sliding block along the sliding plane and the friction angle for different water heights (hw) in the 'internal joint'. The friction angle decreases if the length of the sliding block increases. This relation is less pronounced if the water level in the joints decreases. For a friction angle of 45° (shear test result) along the sliding plane, sliding should not have occurred for a full 15 m length block, even not if the 'internal joint' would have been completely filled with water (hw = 100 %). However, for a block length of 5 m sliding would just have been possible if the 'internal joint' was completely filled with water. For a friction angle of 35° (SSPC - 'sliding criterion', ch. C.2.7.3) sliding would have been possible without the influence of water and independent from the block size.

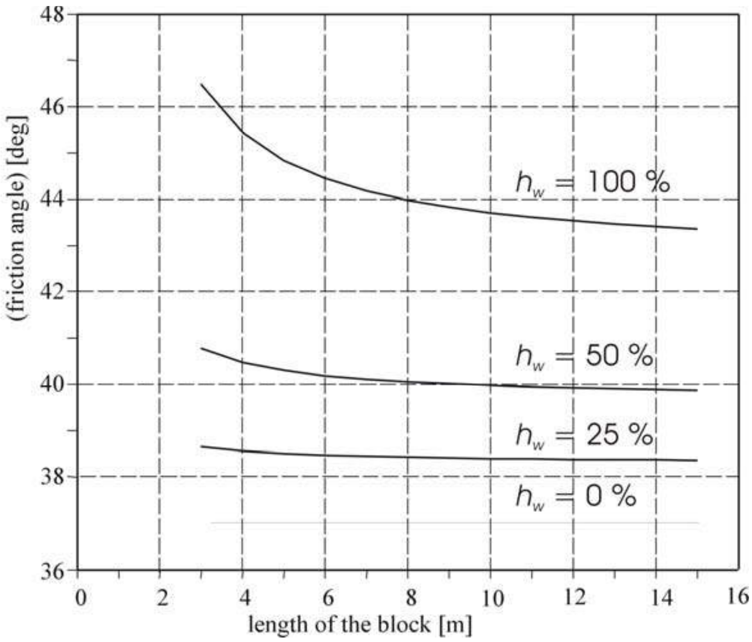


Fig. O-12. The friction angle as function of block length and the height of the water in the second joint set (337/48)

O.4.3.4 Slope stability by numerical analysis - UDEC simulation

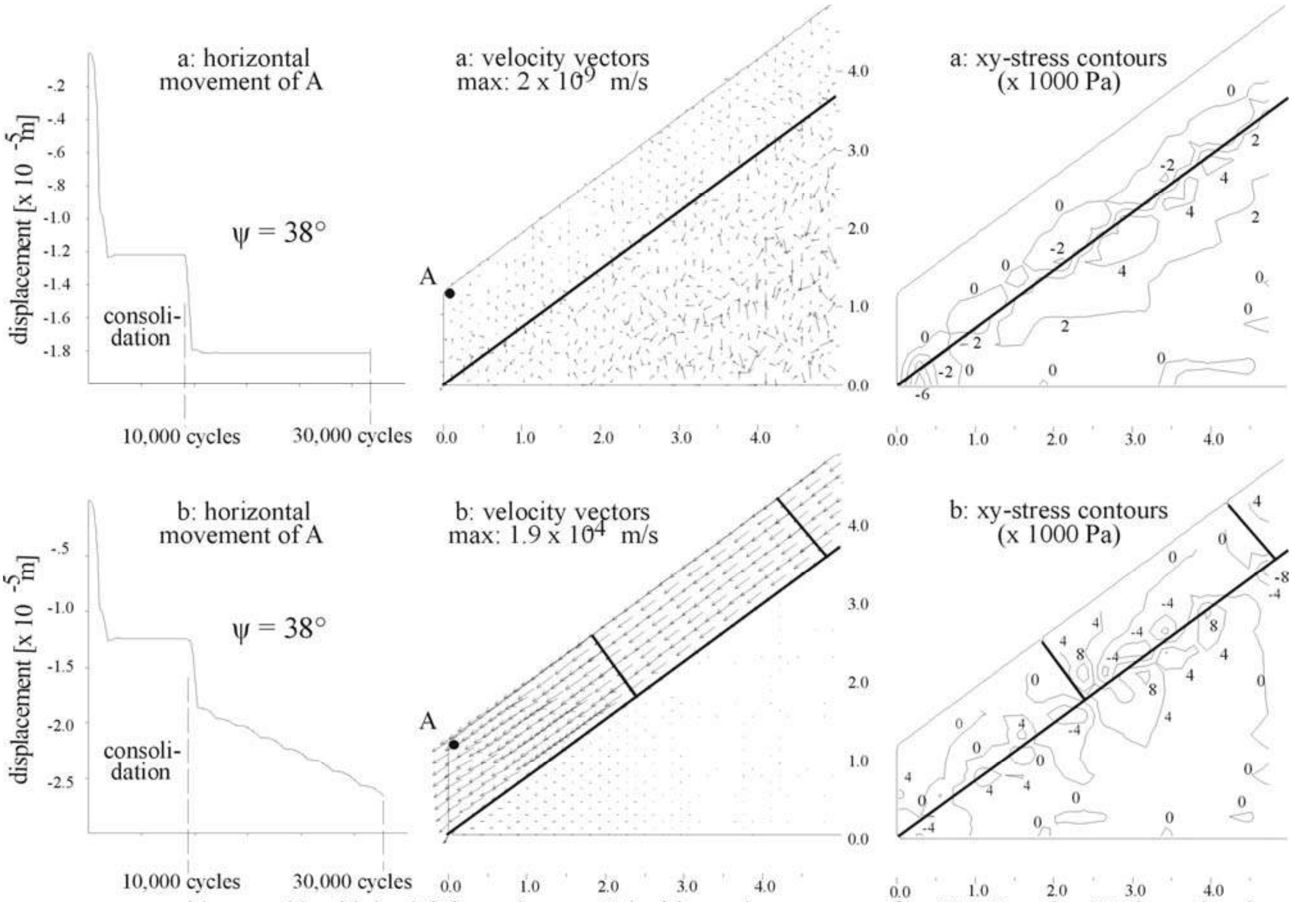
The UDEC simulations were made with fully deformable intact rock blocks[29]. The intact blocks behave elasto-plastic with a Mohr-Coulomb failure criterion and the discontinuities behave as an elasto-plastic area contact with Mohr-Coulomb failure criterion. The program allows for fluid flow and thus for investigating the influence of water pressures. Permeability parameters of discontinuities are, however, extremely difficult to obtain, and the parameters used in the modeling are studied guesses. The parameters have been chosen such that the 'internal joint' fills up with water.

The internal discontinuities striking parallel to the sliding plane are modeled as tension joints. The angle between sliding plane and internal discontinuity is perpendicular. The joints are modeled with tension strength of 7.5 MPa perpendicular to the plane (Brazilian indirect tensile strength for intact rock). The UDEC program allows for setting a so-called 'flag' that causes the tensile strength to become zero after opening of the joint. The friction angle along the sliding plane is varied. The displacement of the block at the toe of the slope versus the number of calculation cycles (representing time) is shown in Fig. O-13a and b; Fig. O-13a for a slope without tension joints and Fig. O-13b with tension joints. The result of the modeling with internal joints shows that the internal joints open and that the lowest block at the toe of the slope is moving down the slope and causing the slope to be unstable for friction angles smaller than 38° without any water being present. Fig. O-13b shows the displacement and the stress distribution in the slope.

O.4.3.5 Conclusions

The classification, and the limiting-equilibrium and numerical analyses show that this slope was prone to failure. The limiting-equilibrium and numerical analyses become unstable if a friction angle along the bedding plane is less than about 37° to 38° without water pressures. This is the friction angle resulting from the SSPC system. The slope would have been stable if the friction angle from the shearbox testing was used in these analyses, except if high water pressures in the discontinuities had been assumed. Completely water filled discontinuities, as an explanation for instability, are unlikely. Even if the bedding plane was completely

[29] Only the main parameters for the modeling are included here. Parameters not mentioned are at a default value as suggested by the manual of the program and have no or minor influence on the modeling results. For more detailed descriptions the reader is referred to the literature (UDEC, 1993).



a: without and b with back joint and water. Velocities and xy-stresses after 30,000 cycles. Horizontal and vertical dimensions in metres from the toe of the slope. Sections are in the direction of the slope (162).

Fig. O-13. UDEC simulation. Enlarged part of the toe of the slope showing displacements, velocity, and xy-stresses along sliding plane

blocked at the toe of the slope, then in this type of rock mass water must have been able to flow out the bedding plane sideways via the vertical joint set.

The most likely explanation for this failure after 40 years, is therefore that the slope always had a stability almost unity. However, the weathering over the years caused a very slight decrease in the shear strength of the bedding plane, reducing its shear strength slightly. The final trigger for failure may have been water, but with water pressures considerably smaller than associated with a fully filled up discontinuity. In addition, it is likely that water caused a softening of the infill material in the bedding plane and acted as lubricating agent in the discontinuity.

This example clearly shows the relative uselessness of taking samples and testing these samples for shear strength. Apart of the size of the samples, which only represent a very small part of the rock mass, all problems with taking samples and testing these are encountered. If results of such tests are not very carefully scrutinized, the conclusions based on subsequent analyses might be incorrect. Classification does not involve difficult sampling and testing and, more importantly, it does describe a large part of the rock mass. The analytical and numerical calculations are only possible because the slope is relatively simple, so that the degree of simplification is minimal. Even then, these calculation methods including the testing take many hours or even days to execute, while a classification probability study is carried out in less than a quarter of an hour.

P SITE INVESTIGATION FOR UNDERGROUND EXCAVATIONS

P.1 Introduction

These notes do not deal with site investigation or engineering geological modeling in general. However, underground excavation require some special methodology for site investigation, and therefore in this chapter briefly some methodologies for site investigations for underground excavations are reviewed. The object of site investigations for underground excavations must be to recover information adequate to anticipate problems of excavation, water inflow, support a future behavior of the materials. Extensive use is to be made of conventional geological survey methods applied to a large area surrounding the site in order to understand the geology as far as possible and to view the strata likely to be encountered in the excavation in the mass rather than only in samples taken from boreholes. The use of geophysical methods may be useful; particularly cross-hole shooting between investigation boreholes and, for excavations underwater, continuous seismic profiling methods (TAble ??. Considerable attention must be given to portal conditions for both tunnels and shafts for both pass through the weakest and most permeable ground conditions likely to be encountered. Many tunnels begin in surface excavation and such an excavation must be made with regard to conditions of slope stability, which apply to any surface excavation.

It is extremely useful if possible, to visit other underground workings nearby, talk to miners or consultants and contractors involved in the workings, and read publications. An underground work nearby is in fact a large true-scale test of what is going to be done and can give priceless information on possible virgin stress field problems, types of support and excavation methods, problems with support and stability, etc. In addition, practical data may be available as addresses of suppliers of support and excavation equipment. The author has noted that such visits are seldom done, probably because, in particular junior engineering geologists are afraid to show their lack of experience or are afraid that the information will be refused because of company confidentiality. In general, however, the atmosphere of openness and non-secrecy is remarkably larger in underground works than in surface works. Obviously, it may not be a good idea to visit potential competitors in the feasibility or tender phase of a project.

P.2 Site investigation components

The factors to be studied in a site investigation may be summarized as follows:

1. Geological conditions with particular reference to lithology and structure (including faulting and the orientation and spacing of discontinuities).
2. The engineering properties of the materials to be excavated and supported (with particular reference to: strength and deformability of intact material and discontinuities, abrasivity to cutting tools, and squeezing and swelling).
3. Hydrogeology.
4. Stability of portals and access shafts.
5. Natural hazards (such as earthquakes, temperature underground, dangerous gasses, avalanches over portal areas, etc.).

The groundmass should be divided in geotechnical units, based on the geotechnical properties and the cost per unit of the work done in that unit should be approximately calculated, if required for different alternative sites or routings. Making underground excavations, choice of excavation method and type of support is for a large part depending on experience, but choices should be sustained by other sources. Rock mass classification systems (chapter M) and supplier details on suitable excavation and support depending on the groundmass conditions. The keyblock theory (Goodman, 1995) may be used to establish bolt and anchor loads and lengths. It is advisable not to rely on just one system or methodology, as these do certainly not always come to the same recommendations. The differences, however, may indicate which system or methodology is the

Table P-1. Tentative indication of suitability of various geophysical methods. It is assumed that basic boundary conditions have been fulfilled; for example, no high- conductive materials present above where a ground radar survey is to be done.

method		artifacts, pipes,foun-dations, etc.	property determination for geotechnical purposes	structure			
				low contrast(*)		high contrast(*)	
				simple (**)	complex (**)	simple (**)	complex (**)
Seimic	refraction	--	+	-	--	++	-
	reflection	-	+	+	-	++	+
	tomography	-	+	++	++	++	++
Electro-magnetic	low frequency	++	--	-	--	-	--
	groundradar	++	--	++	+	++	++
Geo-electrical	normal	-	--	-	--	++	+
	2/3D imaging	-	--	++	++	++	++
Self-potential		--	--	--	--	-	-
Gravity		-	++	-	--	++	+

-- = not suitable, - marginal, + = good, ++ = very good

(*) low and high contrast refer to the contrast in property values measured between the different materials that define the structure

(**) simple and complex structure refer to the complexity of the structure to be measured, for example, simple should be something like two horizontal or slightly inclined layers, e.g. a topsoil layer on a rock slope, complex should be a series of irregular layers and objects, e.g. a debris flow deposit.

All methods can theoretically be used on- and off-shore, however, offshore it should be noted that conductive water layers will mask detail and will reduce depth penetration in electro-magnetic and geo-electrical methods, and may make surveys cumbersome to execute. In common practice only seismic reflection surveys and tomography are done off-shore.

most suitable for a particular project and groundmass. Some of the rock mass classification systems are linked to a 'stand up' time that depends upon rock class and tunnel span and will indicate how quickly support has to be applied after excavation. A first approach of the support requirements for tunnels in soil can be based on relatively simple calculations as found in most textbooks on soil mechanics and from these the appropriate properties to be collected in the site investigation, can be determined. More elaborate numerical calculations can refine the design at later stages in the project.

P.3 Geophysical surveys

None of the geophysical methods is better than another method. The success with which a method is applied fully depends on the circumstances at the site and on the sub-surface materials. Unsuccessful surveys are nearly always due to a lack of proper preparation. Often a survey is done only based on a vague article describing a similar type of survey, or on just the recommendation of an assumed 'expert'. To avoid disappointing results it is therefore important to establish on forehand whether it is likely that the required structures or properties can be measured with success in a particular situation. This should not be done only qualitative by estimation based on experience, but calculations should be done that simulate reality at the actual site as best as possible. Many surveys are unsuccessful because this has not been done properly.

A combination of different methods for the same site leads often to successful geophysical surveys because different features of the sub-surface structure are detected by different methods (Anon, 1995b, Bruno et al, 1998, Williams et al., 1996). Secondly, different methods may confirm the existence of a vague feature that may be missed if only one method is used. Table P-1 lists the various methods and whether a method is more or less suitable for a particular task. It should be realized that the table is very crude and is in no-way conclusive.

P.4 Excavation and support details

A first idea on the type of excavation method can be based on the graphs published in the literature, but should be refined by data from the manufacturers of excavation equipment and explosives for project design and construction phases. Choice of explosives is often limited to the explosives manufactured locally as long transport or importing explosives is cumbersome, time consuming, costly, and often just simple impossible. Ground properties required for the choice of a type of TBM have been lined out briefly in the chapters before. Development in TBM technology cause detailed groundmass criteria for application of a particular type of TBM, which are often manufacturer specific, to change so rapidly that it is not very useful to provide such criteria as these are likely outdated by the time of publication. The matching of a particular TBM to the subsurface conditions, its production estimates (Fig. K-11), and characteristics of TBM installed support can best be derived from the various manufacturers of TBMs. (which all have websites showing the latest developments and are quit prepared to provide additional information). An estimation of production rates in rock may also be made with the QTBM system, which is based on Barton's Q-system rock classification system (Barton, 2000). The International Tunnel Association (ITA) provides useful publications on TBMs (ITA, 2002).

P.5 Data gathering and engineering geological modeling

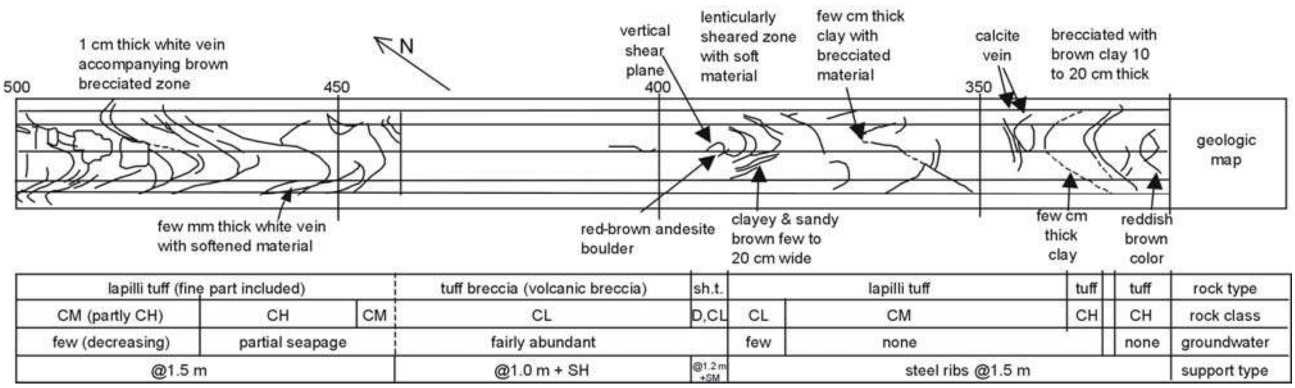


Fig. P-1. Example of a tunnel routing with geology, rock type and class, hydrogeology, and support details (simplified after Mitani, 1998)

Site investigations do not stop once tunnel construction is begun. It is now almost standard good practice to have continuous recording of geological conditions encountered in the progressing tunnel. Information is continually re-interpreted to give a better idea of what is yet to come in the tunnel to be dug. In some tunnels, 'probing ahead' is undertaken to establish the nature of the ground to be excavated in the next phase of tunneling. Fig. P-1 shows an example of the routing for a tunnel in a more traditional way. The top part shows the geotechnical units and the lower part shows the main properties for the tunnel support design. This type of section gives a fast overview of the whole tunnel; however, it is only a section and provides only details along one particular line. More sophisticated means for visualizing a tunnel, the properties, and calculating and visualizing costs are possible with the use of 3D-GIS programs (Fig. P-2), which provide possibilities to visualize the distributions of properties throughout space together with the evaluation of different routing and design options in 3D.

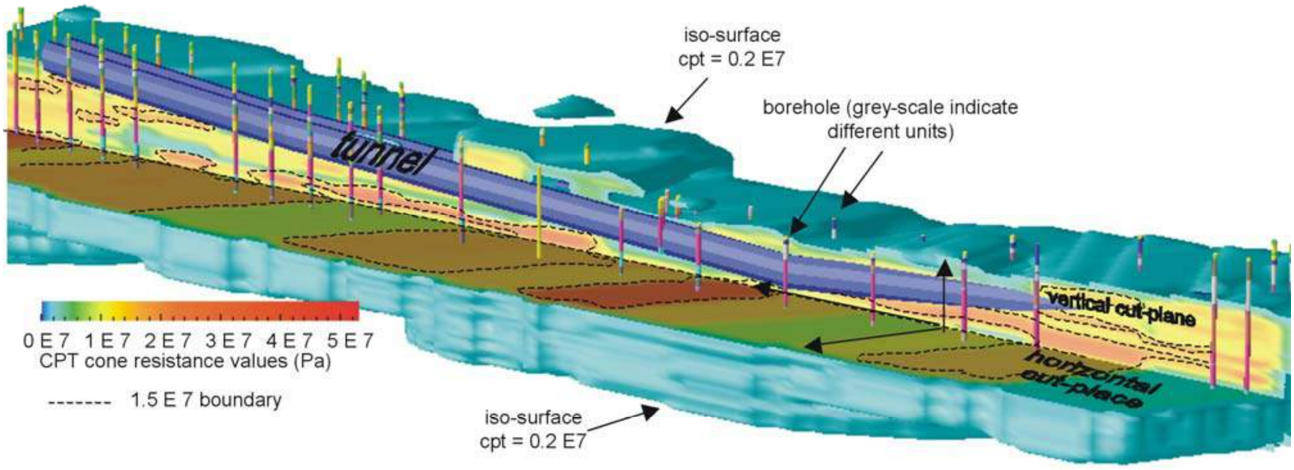


Fig. P-2. Example of 3D-GIS visualization of proposed tunnel alignment in a solid volume model of distribution of CPT cone resistance values, with boreholes showing geotechnical units and two cut-planes to show the distribution of CPT values (Heinenoord Tunnel, Netherlands; after Ozmutlu, 2002)

P.6 New Austrian Tunneling Method (NATM)

Special attention should be given to the New Austrian Tunneling Method (NATM). This methodology (or probably better a philosophy) has become very popular in recent years and has been applied for parts of the Channel Tunnel between France and Britain, and for many tunnels in Germany and Austria. The NATM (Müller, 1978, Pacher et al., 1974, Rabcewicz et al., 1964, 1972) (see also chapter M) comprises characterization and classification but also includes rock mass modeling, deformation monitoring, and the design and construction of a tunnel. Legal contract aspects between client, consultants, and contractors may also be included. A main feature is the use of a semi-flexible support with a closed invert, which may consist of steel arches or rock bolts and wire mesh with sprayed concrete installed direct after excavation to minimize movements in the rock mass, however, the moment of installation may be delayed to allow maximum arching effects to develop. Another main aspect is that the support should not be established in advance, but be adjusted to the circumstances encountered while progressing. Therefore, the system requires contracts to be drafted such that contractors are able to immediately install adjusted support (e.g. without time-consuming contract negotiations with the client and consultants) while the work is ongoing. Various modifications, adjusted to local circumstances, have been developed worldwide (Japan, 1992). The NATM is actually a combination of methods and methodologies that already existed, but have been grouped together as a 'method' (Kovári, 1994) and been extended with extensive options to calculate stability and support requirements based on monitoring. Remarkably is that many claim to follow the NATM system without exactly knowing what the system incorporates, for example, NATM is claimed to be used because shotcrete or the RMR classification system are applied. This is nonsense as shotcrete support and the RMR were developed completely independent of NATM. A recent less successful application of NATM was for the Heathrow Express Link underground railway line that partially collapsed in 1994. The investigation after the collapse showed that support installed was at some locations considerably less than pre-scribed in the design. This has raised some questions on the correctness of the NATM philosophy to arrange support installation and quality control such that the client has no direct control during the work.

P.7 Mobilization costs

A factor often forgotten is the mobilization of equipment and material to the project site and the availability of appropriate skilled labor. It may be satisfying to choose a state-of-the-art method, but if it and its labor have to come from far or if a new road has to be built to transport large-sized parts of a TBM, transport, and labor costs may well rule out the application of the method. It is therefore not a bad idea that the engineering geologist before any site investigation is done, checks what local resources are available; a telephone directory, a few telephone calls, and Internet may be very informative.

P.8 Site investigation for surface effects of tunneling

Tunnels in soft ground will give rise to surface subsidence if crown settlement is allowed to proceed unchecked. Accidental inflows of water bearing non-cohesive ground into the tunnel can cause surface subsidence. In hard ground, blasting in shallow tunnels may cause damage to buildings at surface. Explosive tests and vibration measurements should be carried out before the tunnel is too far advanced to obtain the best blasting system and pattern possible for the least surface disturbance compatible with good driving progress. It is prudent to have an inspection (with photographic records) of surface property before driving commences, to obtain evidence that may be required to counter future litigation. Good relations should be established with local residents and politicians. This later aspect may contribute enormously towards the success of a project in times where the average citizen knows how to find legal and political ways to delay or stop a project.

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

Appendix A Barton's Q-system, Bieniawski's RMR, Laubscher's MRMR, and Hack's SSPC

Barton's Q-system

(Copy from the book: "Engineering Rock Mass Classifications", by Z.T. Bieniawski, 1989. Publ. John Wiley & Sons, Inc. New York. pages: 73-90.)

Bieniawski's RMR system

(Copy from the book: "Engineering Rock Mass Classifications", by Z.T. Bieniawski, 1989. Publ. John Wiley & Sons, Inc. New York. pages: 51-69.)

Laubscher's MRMR system

(Copy from. "A geomechanics classification system for rating of rock mass in mine design" by Laubscher D.H., 1990. Publ. in Journal South African Inst. of Mining and Metallurgy. 90, No. 10, pp. 257 - 273.)

Hack's SSPC

Slope Stability Probability Classification (SSPC) - System to assess slope stability in terms of probability.

Including Erata of SSPC system

The copies of the systems are included to provide a concise overview of some of the classification systems in use. For comments and discussion is referred to the main text of the lecture notes.

Important: Before using any of the systems in real projects, it is advised to check whether any new versions of the system have been published. A classification system is empirical and the quality of the system depends on the number of case histories it is based on. In time, more case histories will be available and, hence, the system may (have to) be adjusted.

Appendix B Barton's Q-system

5

Q-System

Few things are created and perfected at the same time.
—Thomas Edison

The Q-system of rock mass classification was developed in Norway in 1974 by Barton, Lien, and Lunde, all of the Norwegian Geotechnical Institute. Its development represented a major contribution to the subject of rock mass classification for a number of reasons: the system was proposed on the basis of an analysis of 212 tunnel case histories from Scandinavia, it is a quantitative classification system, and it is an engineering system facilitating the design of tunnel supports.

The Q-system is based on a numerical assessment of the rock mass quality using six different parameters:

1. RQD.
2. Number of joint sets.
3. Roughness of the most unfavorable joint or discontinuity.
4. Degree of alteration or filling along the weakest joint.
5. Water inflow.
6. Stress condition.

These six parameters are grouped into three quotients to give the overall rock mass quality Q as follows:

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$$Q = \frac{\text{RQD}}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{\text{SRF}} \tag{5.1}$$

where RQD = rock quality designation,
J_n = joint set number,
J_r = joint roughness number,
J_a = joint alteration number,
J_w = joint water reduction number,
SRF = stress reduction factor.

The rock quality can range from $Q = 0.001$ to $Q = 1000$ on a logarithmic rock mass quality scale.

5.1 CLASSIFICATION PROCEDURES

Table 5.1 gives the numerical values of each of the classification parameters. They are interpreted as follows: The first two parameters represent the overall structure of the rock mass, and their quotient is a relative measure of the block size. The quotient of the third and the fourth parameters is said to be an indicator of the interblock shear strength (of the joints). The fifth parameter is a measure of water pressure, while the sixth parameter is a measure of a) loosening load in the case of shear zones and clay bearing rock, b) rock stress in competent rock, and c) squeezing and swelling loads in plastic incompetent rock. This sixth parameter is regarded as the "total stress" parameter. The quotient of the fifth and the sixth parameters describes the "active stress."

Barton et al. (1974) consider the parameters J_n, J_r, and J_a as playing a more important role than joint orientation, and if joint orientation had been included, the classification would have been less general. However, orientation is implicit in parameters J_r and J_a because they apply to the most unfavorable joints.

The Q value is related to tunnel support requirements by defining the equivalent dimensions of the excavation. This equivalent dimension, which is a function of both the size and the purpose of the excavation, is obtained by dividing the span, diameter, or the wall height of the excavation by a quantity called the excavation support ratio (ESR). Thus

$$\text{Equivalent dimension} = \frac{\text{span or height (m)}}{\text{ESR}} \tag{5.2}$$

The ESR is related to the use for which the excavation is intended and the degree of safety demanded, as shown below:

Excavation Category	ESR	No. of Cases
A. Temporary mine openings	3–5	2
B. Vertical shafts:		
Circular section	2.5	
Rectangular/square section	2.0	
C. Permanent mine openings, water tunnels for hydropower (excluding high-pressure penstocks), pilot tunnels, drifts, and headings for large excavations	1.6	83
D. Storage caverns, water treatment plants, minor highway and railroad tunnels, surge chambers, access tunnels	1.3	25
E. Power stations, major highway or railroad tunnels, civil defense chambers, portals, intersections	1.0	73
F. Underground nuclear power stations, railroad stations, factories	0.8	2

The relationship between the index Q and the equivalent dimension of an excavation determines the appropriate support measures, as depicted in Figure 5.1. Barton et al. (1974) provided the corresponding 38 support categories specifying the estimates of permanent support, as given in Tables 5.2–5.6. For temporary support determination, either Q is increased to $5Q$ or ESR is increased to 1.5 ESR.

It should be noted that the length of bolts is not specified in the support tables, but the bolt length L is determined from the equation

$$L = \frac{2 + 0.15B}{\text{ESR}} \tag{5.3}$$

where B is the excavation width.

The maximum unsupported span can be obtained as follows:

$$\text{Maximum span (unsupported)} = 2(\text{ESR})\, Q^{0.4} \tag{5.4}$$

The relationship between the Q value and the permanent support pressure P_{roof} is calculated from the following equation:

TABLE 5.1 Q-System Description and Ratings: Parameters RQD, J_n, J_r, J_a, SRF, and J_w[a]

Rock Quality Designation (RQD)

Very poor	0–25	*Note:*
Poor	25–50	(i) Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q in equation (5.1).
Fair	50–75	
Good	75–90	
Excellent	90–100	(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate

Joint Set Number J_n

Massive, none or few joints	0.5–1.0	*Note:*
One joint set	2	(i) For intersections, use $(3.0 \times J_n)$
One joint set plus random	3	
Two joint sets	4	(ii) For portals, use $(2.0 \times J_n)$
Two joint sets plus random	6	
Three joint sets	9	
Three joint sets plus random	12	
Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15	
Crushed rock, earthlike	20	

Joint Roughness Number J_r

(a) Rock wall contact and		*Note:*
(b) Rock wall contact before 10-cm shear		(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
Discontinuous joint	4	
Rough or irregular, undulating	3	
Smooth, undulating	2.0	*Note:*
Slickensided, undulating	1.5	(ii) J_r = 0.5 can be used for planar slickensided joints having lineation, provided the lineations are favorably oriented
Rough or irregular, planar	1.5	
Smooth, planar	1.0[b]	
Slickensided, planar	0.5	(iii) Descriptions B to G refer to small-scale features and intermediate-scale features, in that order
(c) No rock wall contact when sheared		
Zone containing clay minerals thick enough to prevent rock wall contact	1.0[b]	
Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	1.0[b]	

Joint Alteration Number J_a

(a) Rock wall contact	J_a	ϕ_r (approx)
A. Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote	0.75	
B. Unaltered joint walls, surface staining only	1.0	25–35°
C. Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25–30°
D. Silty or sandy clay coatings, small clay fraction (nonsoftening)	3.0	20–25°
E. Softening or low-friction clay mineral coatings, i.e., kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less in thickness)	4.0	8–16°
(b) Rock wall contact before 10-cm shear		
F. Sandy particles, clay-free disintegrated rock, etc.	4.0	25–30°

(Table continues on p. 78.)

TABLE 5.1 (*Continued*)

Joint Alteration Number J_a

G. Strongly over-consolidated, nonsoftening clay mineral fillings (continuous, <5 mm in thickness)	6.0	16–24°
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous, <5 mm in thickness)	8.0	12–16°
J. Swelling clay fillings, i.e., montmorillonite (continuous, < mm in thickness). Value of J_a depends on percentage of swelling clay-sized particles, and access to water, etc.	8.0–12.0	6–12°
(c) No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed rock and clay (see G., H., J. for description of clay condition)	6.0, 8.0 or 8.0–12.0	6–24°
L. Zones or bands of silty or sandy clay, small clay fraction (nonsoftening)	5.0	
M. Thick, continuous zones or bands of clay (see G., H., J. for description of clay condition)	10.0, 13.0 or 13.0–20.0	6–24°
Note: (i) Values of ϕ_r are intended as an approximate guide to the mineralogical properties of the alteration products, if present		

Stress Reduction Factor (SRF)

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				*Note:* (i) Reduce these SRF values by 25–50% if the relevant shear zones only influence but do not intersect the excavation
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0	
B. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤50 m)			5.0	
C. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)			2.5	
D. Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5	
E. Single-shear zones in competent rock (clay-free) (depth of excavation ≤50 m)			5.0	
F. Single-shear zones in competent rock (clay-free) (depth of excavation >50 m)			2.5	
G. Loose open joints, heavily jointed or "sugar cube," etc. (any depth)			5.0	
(b) Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		
H. Low stress, near surface	>200	>13	2.5	(ii) For strongly anisotropic stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\ \sigma_c$ and $0.8\ \sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to
J. Medium stress	200–10	13–0.66	1.0	

(*Table continues on p. 80.*

TABLE 5.1 (*Continued*)

Stress Reduction Factor (SRF)

K. High-stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability	10–5	0.66–0.33	0.5–2.0	0.6 σ_c and 0.6 σ_t (where σ_c = unconfined compressive strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses)
L. Mild rock burst (massive rock)	5–2.5	0.33–0.16	5–10	
M. Heavy rock burst (massive rock)	<2.5	<0.16	10–20	
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
N. Mild squeezing rock pressure			5–10	
O. Heavy squeezing rock pressure			10–20	(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)
(d) Swelling rock; chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5–10	
R. Heavy swelling rock pressure			10–15	

Joint Water Reduction Factor J_w

	J_w	Approximate water pressure (kg/cm^2)	
A. Dry excavations or minor inflow, i.e., 5 L/min locally	1.0	<1	*Note:* (i) Factors C–F are crude estimates. Increase J_w if drainage measures are installed
B. Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0–2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0	(ii) Special problems caused by ice formation are not considered
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5–10.0	
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2–0.1	>10.0	
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1–0.05	>10.0	

[a] After Barton et al. (1974).
[b] Nominal.

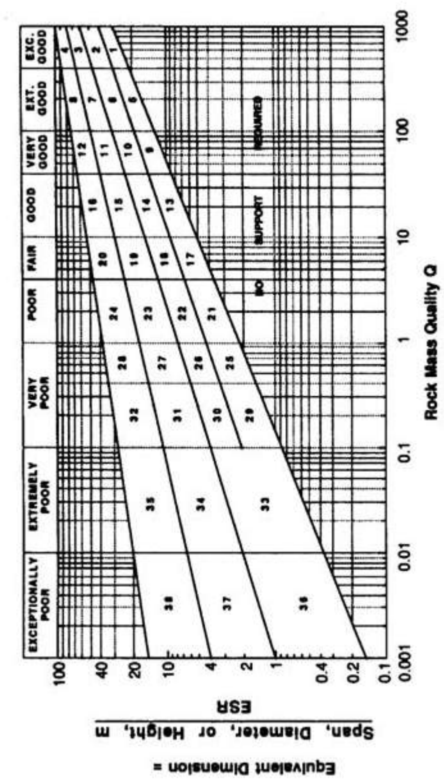

Figure 5.1 *Q-system: equivalent dimension versus rock mass quality. (After Barton et al., 1974.)*

$$P_{\mathrm{roof}} = \frac{2.0}{J_r} Q^{-1/3} \tag{5.5}$$

If the number of joint sets is less than three, the equation is expressed as

$$P_{\mathrm{roof}} = \frac{2}{3} J_n^{1/2} J_r^{-1} Q^{-1/3} \tag{5.6}$$

Although the Q-system involves 9 rock mass classes and 38 support categories, it is not necessarily too complicated. Some users of the Q-system have pointed out that the open logarithmic scale of Q varying from 0.001 to 1000 can be a source of difficulty; it is easier to get a feeling for a quoted rock mass quality using a linear scale of up to 100. The numerical procedure may also give some users a misplaced sense of numerical precision—for example, when reporting Q values such as "11.53."

5.2 CORRELATIONS

As stated in Section 4.4, a correlation was developed between the Q-index and the RMR (Bieniawski, 1976) as well as between the Q-index and the RSR (Rutledge and Preston, 1978). A total of 111 case histories were analyzed for this purpose: 62 Scandinavian cases, 28 South African cases,

TABLE 5.2 Q-System: Support Measures for Q Range 10 to 1000[a]

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	Span/ESR (m)	P^b (kg/cm²)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
1[c]	1000–400				<0.01	20–40	sb (utg)	
2[c]	1000–400				<0.01	30–60	sb (utg)	
3[c]	1000–400				<0.01	46–80	sb (utg)	
4[c]	1000–400				<0.01	65–100	sb (utg)	
5[c]	400–100				0.05	12–30	sb (utg)	
6[c]	400–100				0.05	19–45	sb (utg)	
7[c]	400–100				0.05	30–65	sb (utg)	
8[c]	400–100				0.05	48–88	sb (utg)	
9	100–40	⩾20			0.25	8.5–19	sb (utg)	
		<20					B (utg) 2.5–3 m	
10	100–40	⩾30			0.25	14–30	B (utg) 2–3 m	
		<30					B (utg) 1.5–2 m + clm	
11[c]	100–40	⩾30			0.25	23–48	B (tg) 2–3m	
		<30					B (tg) 1.5–2 m + clm	
12[c]	100–40	⩾30			0.25	40–72	B (tg) 2–3m	
		<30					B (tg) 1.5–2 m + clm	

(Table continues on p. 84.)

TABLE 5.2 (*Continued*)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	Span/ESR (m)	P^b (kg/cm²)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
13	40–10	≥10	≥1.5		0.5	5–14	sb (utg)	I
		≥10	<1.5				B (utg) 1.5–2 m	I
		<10	≥1.5				B (utg) 1.5–2 m	I
		<10	<1.5				B (utg) 1.5–2 m + S 2–3 cm	I
14	40–10	≥10		≥15	0.5	9–23	B (tg) 1.5–2 m + clm	I, II
		<10		≥15			B (tg) 1.5–2 m + S (mr) 5–10 cm	I, II
				<15			B (utg) 1.5–2 m + clm	I, III
15	40–10	>10			0.5	15–40	B (tg) 1.5–2 m + clm	I, II, IV
		≤10					B (tg) 1.5–2 m + S (mr) 5–10 cm	I, II, IV
16[c,d]	40–10	>15			0.5	30–65	B (tg) 1.5–2 m + clm	I, V, VI
		≤15					B (tg) 1.5–2 m + S (mr) 10–15 cm	I, V, VI

[a] After Barton et al. (1974).
[b] Approx.
[c] Original authors' estimates of support. Insufficient case records available for reliable estimation of support requirements. The type of support to be used in categories 1–8 will depend on the blasting technique. Smooth-wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1–8. Key: sb = spot bolting; B = systematic bolting; (utg) = untensioned, grouted; (tg) tensioned (expanding-shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; S = shotcrete; (mr) = mesh-reinforced; clm = chain-link mesh; CCA = cast concrete arch; (sr) steel-reinforced. Bolt spacings are given in meters (m). Shotcrete or cast concrete arch thickness is given in centimeters (cm).
[d] See note XII in Table 5.6.

TABLE 5.3 Q-System: Support Measures for Q Range 1 to 10[a]

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	Span/ESR (m)	P^b (kg/cm²)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
17	10–4	>30			1.0	3.5–9	sb (utg)	I
		≥10, ≤30					B (utg) 1–1.5 m	I
		<10		≥6			B (utg) 1–1.5 m + S 2–3 cm	I
		<10		<6			S 2–3 cm	I
18	10–4	>5		≥10	1.0	7–15	B (tg) 1–1.5 m + clm	I, III
		>5		<10			B (utg) 1–1.5 m + clm	I
		≤5		≥10			B (tg) 1–1.5 m + S 2–3 cm	I, III
		≤5		<10			B (utg) 1–1.5 m + S 2–3 cm	I
19	10–4			≥20	1.0	12–29	B (tg) 1–2 m + S (mr) 10–15 cm	I, II, IV
				<20			B (tg) 1–1.5 m + S (mr) 5–10 cm	I, II
20[c]	10–4			≥35	1.0	24–52	B (tg) 1–2 m + S (mr) 20–25 cm	I, V, VI
				<35			B (tg) 1–2 m + S (mr) 10–20 cm	I, II, IV
21	4–1	≥12.5	≤0.75		1.5	2.1–6.5	B (utg) 1m + S 2–3 cm	I
		<12.5	<0.75				S 2.5–5 cm	I
			>0.75				B (utg) 1m	I
22	4–1	>10, <30	>1.0		1.5	4.5–11.5	B (utg) 1m + clm	I
		≤10	>1.0				S 2.5–7.5 cm	I
		<30	≤1.0				B (utg) 1 m + S (mr) 2.5–5 cm	I
		≥30					B (utg) 1 m	I
23	4–1			≥15	1.5	8–24	B (tg) 1–1.5 m + S (mr) 10–15 cm	I, II, IV, VII
				<15			B (utg) 1–1.5 m + S (mr) 5–10 m	I
24[c,d]	4–1			≥30	1.5	18–46	B (tg) 1–1.5 m + S (mr) 15–30 cm	I, V, VI
				<30			B (tg) 1–1.5 m + S (mr) 10–15 cm	I, II, IV

[a] After Barton et al. (1974).
[b] Approx.
[c] See note XII in Table 5.6.
[d] See footnote *c* in Table 5.2.

TABLE 5.4 Q-System: Support Measures for Q Range 0.1 to 1.0[a]

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	Span/ESR (m)	P[b] (kg/cm²)	Span/ESR (m)	Type of Support[c]	Notes (Table 5.6)
25	1.0–0.4	>10	>0.5		2.25	1.5–4.2	B (utg) 1 m + mr or clm	I
		≤10	>0.5				B (utg) 1 m + S (mr) 5 cm	I
			≤0.5				B (tg) 1 m + S (Mr) 5 cm	I
26	1.0–0.4				2.25	3.2–7.5	B (tg) 1 m + S (mr) 5–7.5 cm	VIII, X, XI
							B (utg) 1 m + S 2.5–5 cm	I, IX
27	1.0–0.4			≥12	2.25	6–18	B (tg) 1 m + S (mr) 7.5–10 cm	I, IX
				<12			B (utg) 1 m + S (mr) 5–7.5 cm	I, IX
				>12			CCA 20–40 cm + B (tg) 1 m	VIII, X, XI
				<12			S (mr) 10–20 cm + B (tg) 1 m	VIII, X, XI
28[d]	1.0–0.4			≥30	2.25	15–38	B (tg) 1 m + S (mr) 30–40 cm	I, IV, V, IX
				≥20, <30			B (tg) 1 m + S (mr) 20–30 cm	I, II, IV, IX
				<20			B (tg) 1m + S (mr) 15–20 cm	I, II, IX
							CCA (sr) 30–100 cm + B (tg) 1 m	IV, VIII, X, XI
29	0.4–0.1	>5	>0.25		3.0	1.0–3.1	B (utg) 1 m + S 2–3 cm	
		≤5	>0.25				B (utg) 1 m + S (mr) 5 cm	
			≤0.25				B (tg) 1 m + S (Mr) 5 cm	
30	0.4–0.1	≥5			3.0	2.2–6	B (tg) 1 m + S 2.5–5 cm	IX
		<5					S (mr) 5–7.5 cm	IX
							B (tg) 1 m + S (mr) 5–7.5 cm	VIII, X, XI
31	0.4–0.1	>4			3.0	4–14.5	B (tg) 1 m + S (mr) 5–12.5 cm	IX
		≤4, ≥1.5					S (mr) 7.5–25 cm	IX
		<1.5					CCA 20–40 cm + B (tg) 1 m	IX, XI
							CCA (sr) 30–50 cm + B (tg) 1 m	VIII, X, XI
32[d]	0.4–0.1			≥20	3.0	11–34	B (tg) 1 m + S (mr) 40–60 cm	II, IV, IX, XI
				<20			B (tg) 1 m + S (mr) 20–40 cm	III, IV, IX, XI

[a] After Barton et al. (1974).
[b] Approx.
[c] For key, refer to Table 5.2, footnote *c*.
[d] See note XII in Table 5.6.

TABLE 5.5 Q-System: Support Measures for Q Range 0.001 to 0.1[a]

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	Span/ESR (m)	P[b] (kg/cm²)	Span/ESR (m)	Type of Support[c]	Notes (Table 5.6)
33	0.1–0.01	≥2			6	1.0–3.9	B (tg) 1 m + S (mr) 2.5–5 cm	IX
		<2					S (mr) 5–10 cm	IX
							S (mr) 7.5–15 cm	VIII, X
34	0.1–0.01	≥2	≥0.25		6	2.0–11	B (tg) 1 m + S (mr) 5–7.5 cm	IX
		<2	≥0.25				S (mr) 7.5–15 cm	IX
			<0.25				S (mr) 15–25 cm	IX
							CCA (sr) 20–60 cm + B (tg) 1 m	VIII, X, XI
35[d]	0.1–0.01			≥15	6	6.2–28	B (tg) 1 m + S (mr) 30–100 cm	II, IX, XI
				≥15			CCA (sr) 60–200 cm + B (tg) 1 m	VIII, X, XI, II
				<15			B (tg) 1 m + S (mr) 20–75 cm	IX, XI, III
				<15			CCA (sr) 40–150 cm + B (tg) 1 m	VIII, X, XI, III
36	0.01–0.001				12	1.0–2.0	S (mr) 10–20 cm	IX
							S (mr) 10–20 cm + B (tg) 0.5–1.0 m	VIII, X, XI
37	0.01–0.001				12	1.0–6.5	S (mr) 20–60 cm	IX
							S (mr) 20–60 cm + B (tg) 0.5–1.0 m	VIII, X, XI
38[e]	0.01–0.001			≥10	12	4.0–20	CCA (sr) 100–300 cm	IX
				≥10			CCA (sr) 100–300 cm + B (tg) 1 m	VIII, X, II, XI
				<10			S (mr) 70–200 cm	IX
				<10			S (mr) 70–200 cm	VIII, X, III, XI

[a] After Barton et al. (1974).
[b] Approx.
[c] For key, refer to Table 5.2, footnote *c*.
[d] See note XII in Table 5.6.
[e] See note XIII in Table 5.6.

TABLE 5.6 Q-System: Support Measures—Supplementary Notes[a]

I. For cases of heavy rock bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.
II. Several bolt lengths often used in same excavation, i.e., 3, 5, and 7 m.
III. Several bolt lengths often used in same excavation, i.e., 2, 3, and 4 m.
IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2–4 m.
V. Several bolt lengths often used in same excavation, i.e., 6, 8, and 10 m.
VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4–6 m.
VII. Several of the older-generation power stations in this category employ systematic or spot bolting with areas of chain-link mesh, and a free-span concrete arch roof (25–40 cm) as permanent support.
VIII. Cases involving swelling, e.g., montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.
IX. Cases not involving swelling clay or squeezing rock.
X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.
XI. According to the authors' [Barton et al.] experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., RQD/J_n < 1.5, for example, a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when RQD/J_n < 1.5, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick-setting resin anchors in these extremely poor-quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.
XII. For reasons of safety, the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (span/ESR > 15 m only).
XIII. Multiple drift method usually needed during excavation and support of arch, walls, and floor in cases of heavy squeezing. Category 38 (span/ESR > 10 m only).

[a] After Barton et al. (1974).

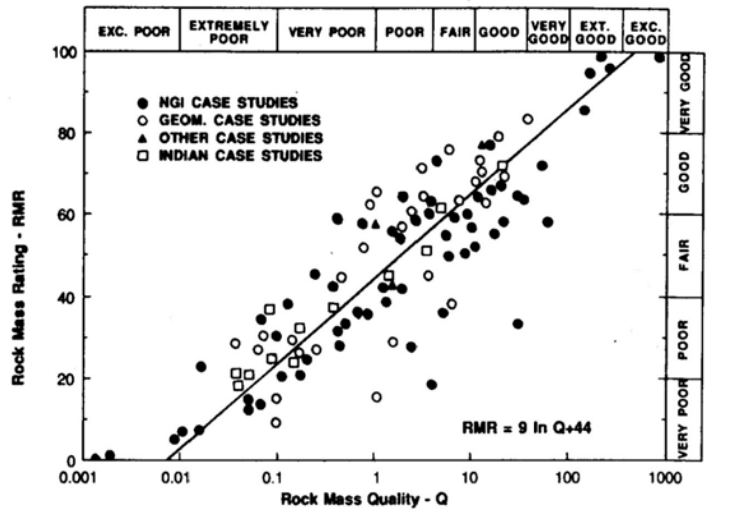

Figure 5.2 *Correlation between the RMR and the Q-index. (After Bieniawski, 1976 and Jethwa et al., 1982.)*

and 21 other case histories from the United States, Canada, Australia, and Europe. The results are plotted in Figure 5.2, from which it can be seen that the following relationship is applicable:

$$\text{RMR} = 9 \ln Q + 44 \tag{5.7}$$

The above correlation was further substantiated by Jethwa et al. (1982), whose case studies are also included in Figure 5.2. Further comparisons between the Q and the RMR systems are given by Barton (1988).

5.3 DATA BASE

Barton (1988) presented histograms of the 212 case records used to develop the Q-system. The majority of the cases are from Scandinavia (Sweden and Norway), including 97 cases reported by Cecil (1970).

The distribution of the rock types was as follows: 13 types of igneous rock, 26 types of metamorphic rock, and 11 types of sedimentary rocks. Hard rock was predominant, involving 48 cases of granite and 21 cases of gneiss.

The Q values covered the whole range of rock mass qualities; there were 40 cases with $Q = 10 - 40$, 45 cases with $Q = 4 - 10$, 36 cases with $Q = 1 - 4$, and 40 cases with $Q = 0.1 - 1.0$.

The predominant tunnel spans or diameters were 5–10 m (78 cases), and 10–15 m (59 cases). There were 40 cases of large caverns from hydroelectric projects, with spans of 15–30 m and wall heights of 30–60 m.

The excavation depths were commonly in the range of 50 to 250 m. However, 20 cases were in the range 250 to 500 m, and 51 cases involved depths less than 50 m.

Most case histories (180) were supported excavations; 32 of the 212 cases were permanently unsupported excavations. The predominant form of support was rock bolts, or combinations of rock bolts and shotcrete often mesh-reinforced.

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Appendix C Bieniawski's RMR-system

4

Geomechanics Classification (Rock Mass Rating System)

If you can measure what you are speaking about and express it in numbers, you know something about it.
—Lord Kelvin

The Rock Mass Rating (RMR) system, otherwise known as the Geomechanics Classification, was developed by the author during 1972–1973 (Bieniawski, 1973). It was modified over the years as more case histories became available and to conform with international standards and procedures (Bieniawski, 1979). Over the past 15 years, the RMR system has stood the test of time and benefited from extensions and applications by many authors throughout the world. These varied applications, amounting to 351 case histories (see Chap. 10), point to the acceptance of the system and its inherent ease of use and versatility in engineering practice, involving tunnels, chambers, mines, slopes, and foundations. Nevertheless, it is important that the RMR system is used for the purpose for which it was developed and not as the answer to all design problems.

Definition of the System Due to the RMR system having been modified several times, and since the method is interchangeably known as the Geomechanics Classification or the Rock Mass Rating system, it is important to state that the system has remained essentially the same in principle despite the changes. Thus, any modifications and extensions were the outgrowth of the same basic method and should not be misconstrued as new systems. To avoid any confusion, the following extensions of the system were valuable new applications but still a part of the same overall RMR system: mining applications, Laubscher (1977, 1984); rippability, Weaver (1975); hard rock mining, Kendorski et al. (1983); coal mining, Unal (1983), Newman and Bieniawski (1986); dam foundations, Serafim and Pereira (1983); tunneling, Gonzalez de Vallejo (1983); slope stability, Romana (1985); and Indian coal mines (Venkateswarlu, 1986).

Moreover, some users of the RMR system list their results as "CSIR rating" or talk of the "CSIR Geomechanical" system. This is incorrect and has never been used or endorsed by the author. The correct expressions are "Rock Mass Rating system" or the "RMR system," or the "Geomechanics Classification." While it is true that the author has worked for an organization whose initials are "CSIR," that organization did not develop the system, and indeed, most of the work on this system was performed after he left the CSIR some 12 years ago.

4.1 CLASSIFICATION PROCEDURES

The following six parameters are used to classify a rock mass using the RMR system (Geomechanics Classification):

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

To apply the Geomechanics Classification, the rock mass is divided into a number of structural regions such that certain features are more or less uniform within each region. Although rock masses are discontinuous in nature, they may nevertheless be uniform in regions when, for example, the type of rock or the discontinuity spacings are the same throughout the region. In most cases, the boundaries of structural regions will coincide with major geological features such as faults, dykes, shear zones, and so on. After the structural regions have been identified, the classification parameters for each structural region are determined from measurements in the field and entered onto the input data sheet given in Figure 2.6.

The Geomechanics Classification is presented in Table 4.1.

In Section A of Table 4.1, five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. The importance ratings are assigned to each parameter according to Section A of Table 4.1. In this respect, the average typical conditions are evaluated for each discontinuity set and the ratings are interpolated, using Classification Charts A–E. The charts are helpful for borderline cases and also remove an impression that abrupt changes in ratings occur between categories. Chart D is used if either RQD or discontinuity data are lacking. Based on the correlation data from Priest and Hudson (1976), the chart enables an estimate of the missing parameter. Furthermore, it should be noted that the importance ratings given for discontinuity spacings apply to rock masses having three sets of discontinuities. Thus, when only two sets of discontinuities are present, a conservative assessment is obtained. In this way, the number of discontinuity sets is considered indirectly. Laubscher (1977) presented a rating concept (see Chap. 8) for discontinuity spacings as a function of the number of joint sets. It can be shown that when less than three sets of discontinuities are present, the rating for discontinuity spacing may be increased by 30%.

After the importance ratings of the classification parameters are established, the ratings for the five parameters listed in Section A of Table 4.1 are summed to yield the basic (unadjusted for discontinuity orientations) RMR for the structural region under consideration.

The next step is to include the sixth parameter, namely, the influence of strike and dip orientation of discontinuities by adjusting the basic RMR according to Section B of Table 4.1. This step is treated separately because the influence of discontinuity orientations depends on the engineering applications, such as a tunnel, mine, slope, or foundation. It will be noted that the "value" of the parameter "discontinuity orientation" is not given in quantitative terms but by qualitative descriptions such as "favorable." To help decide whether strike and dip orientations are favorable or not in tunneling, reference should be made to Table 4.2, which is based on studies by Wickham et al. (1972). For slopes and foundations, the reader is referred to papers by Romana (1985) and by Bieniawski and Orr (1976), respectively.

The parameter "discontinuity orientation" reflects on the significance of the various discontinuity sets present in a rock mass. The main set, usually designated as set No. 1, controls the stability of an excavation; for example, in tunneling it will be the set whose strike is parallel to the tunnel axis. The

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TABLE 4.1 The Rock Mass Rating System (Geomechanics Classification of Rock Masses)[a]

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100–250	50–100	25–50	5–25	1–5	<1
	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	0.6–2 m	200–600 mm	60–200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1–5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10–25	25–125	>125		
			or	or	or	or	or		
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	<0.1	0.1–0.2	0.2–0.5	>0.5		
			or	or	or	or	or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and 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	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15-m span	1 yr for 10-m span	1 wk for 5-m span	10 h for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass (kPa)	>400	300–400	200–300	100–200	<100
Friction angle of the rock mass (deg)	>45	35–45	25–35	15–25	<15

[a] After Bieniawski (1979).

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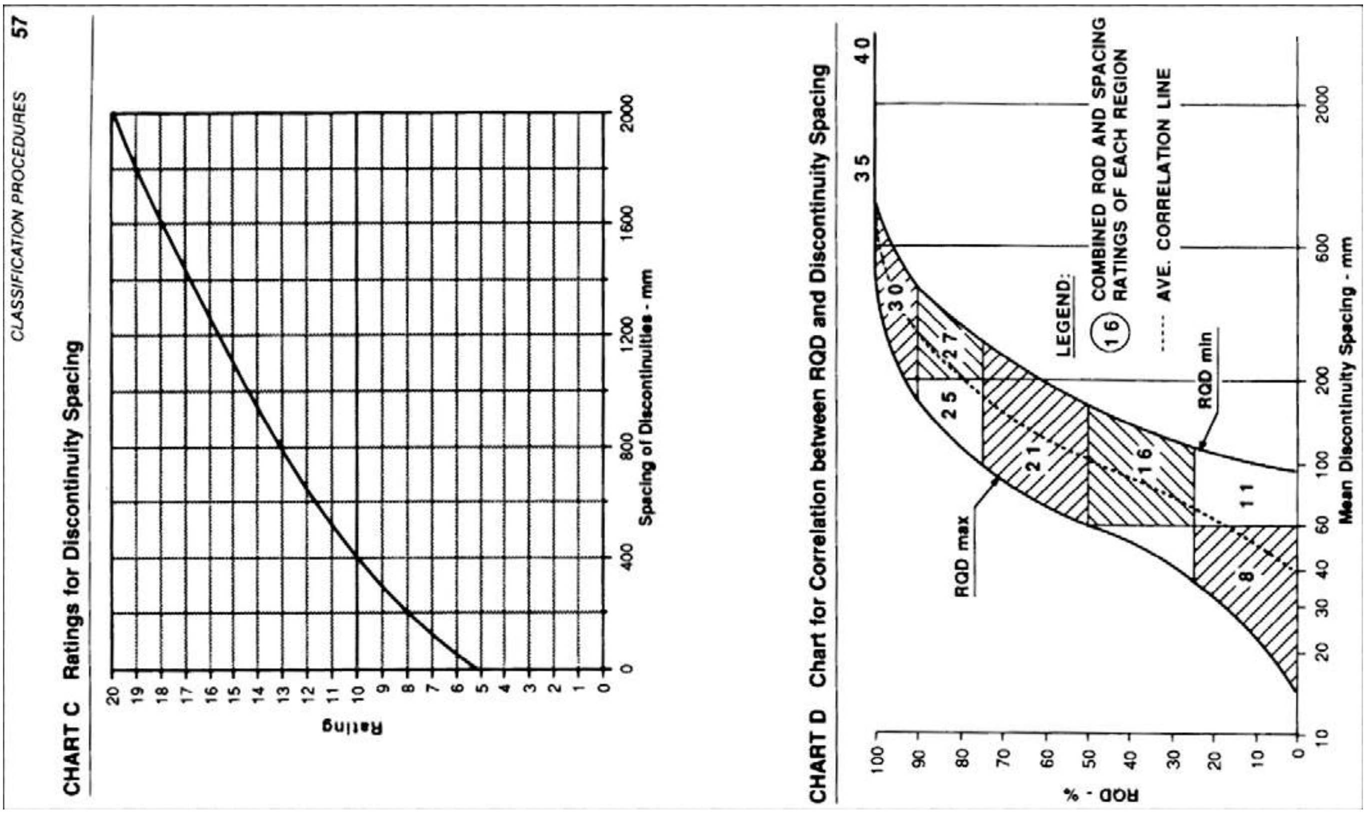
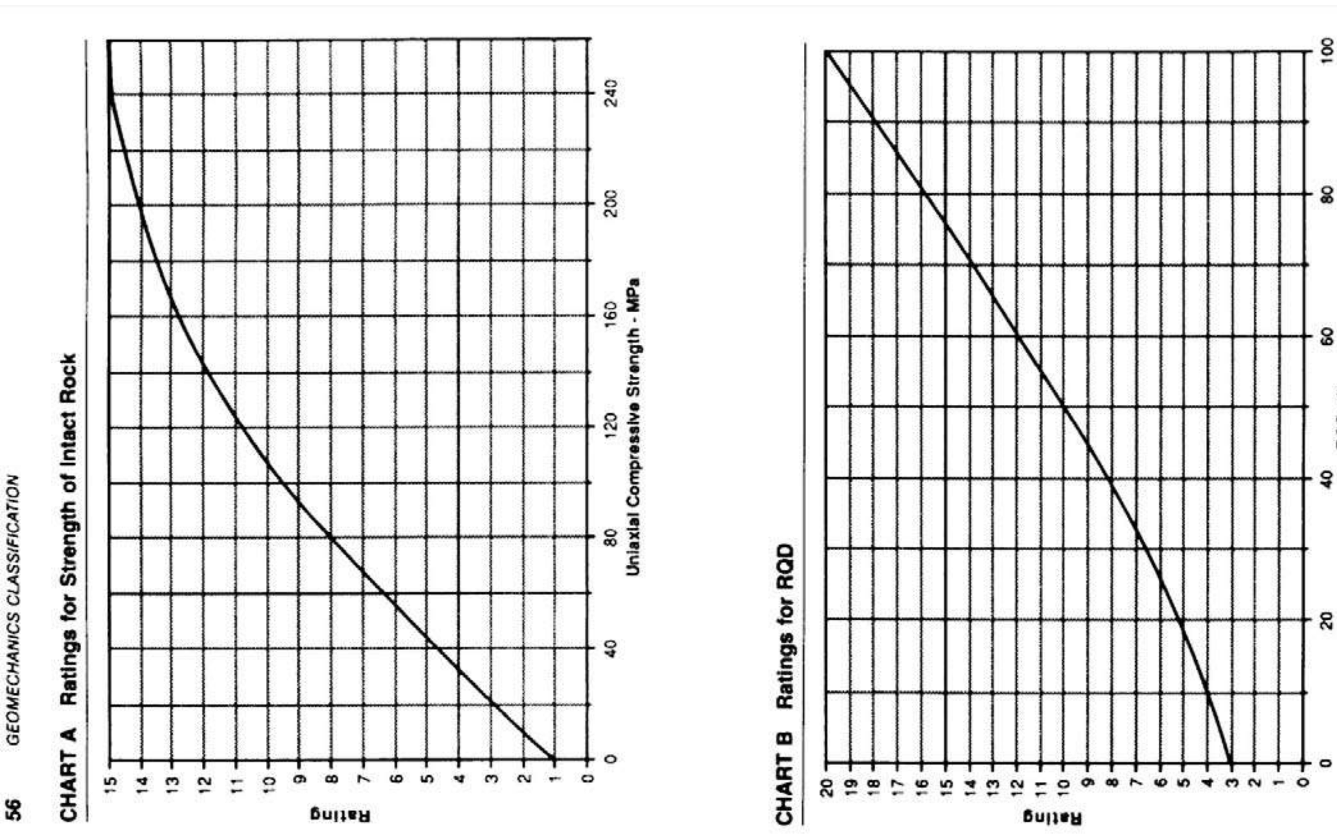

57
CLASSIFICATION PROCEDURES
CHART C Ratings for Discontinuity Spacing
Rating
Spacing of Discontinuities - mm
CHART D Chart for Correlation between RQD and Discontinuity Spacing
RQD - %
Mean Discontinuity Spacing - mm
RQD max
RQD min
LEGEND:
16 COMBINED RQD AND SPACING RATINGS OF EACH REGION
AVE. CORRELATION LINE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GEOMECHANICS CLASSIFICATION
CHART A Ratings for Strength of Intact Rock
Rating
Uniaxial Compressive Strength - MPa
CHART B Ratings for RQD
Rating
RQD - %

TABLE 4.2 Effect of Discontinuity Strike and Dip Orientations in Tunneling[a]

Strike Perpendicular to Tunnel Axis			
Drive with Dip		Drive against Dip	
Dip 45–90	Dip 20–45	Dip 45–90	Dip 20–45
Very favorable	Favorable	Fair	Unfavorable

Strike Parallel to Tunnel Axis		Irrespective of Strike
Dip 20–45	Dip 45–90	Dip 0–20
Fair	Very unfavorable	Fair

[a] Modified after Wickham et al. (1972).

summed-up ratings of the classification parameters for this discontinuity set will constitute the overall RMR. On the other hand, in situations where no one discontinuity set is dominant and of critical importance, or when estimating rock mass strength and deformability, the ratings from each discontinuity set are averaged for the appropriate individual classification parameter.

In the case of civil engineering projects, an adjustment for discontinuity orientations will generally suffice. For mining applications, other adjustments may be called for, such as the stress at depth or a change in stress; these have been discussed by Laubscher (1977) and by Kendorski et al. (1983). The procedure for these adjustments is depicted in Table 4.3.

After the adjustment for discontinuity orientations, the rock mass is classified according to Section C of Table 4.1, which groups the final (adjusted) RMR into five rock mass classes, the full range of the possible RMR values varying from zero to 100. Note that the rock mass classes are in groups of 20 ratings each. This concept of rating a rock mass out of a maximum value of 100 has a distinct advantage over an open-ended system in that it allows the engineer or geologist to get the sense of a relative quality, or the lack of it, of a given rock mass in terms of its maximum potential.

Next, Section D of Table 4.1 gives the practical meaning of each rock mass class by relating it to specific engineering problems. In the case of tunnels, chambers, and mines, the output from the Geomechanics Classification is the stand-up time and the maximum stable rock span for a given RMR, as shown in Figure 4.1.

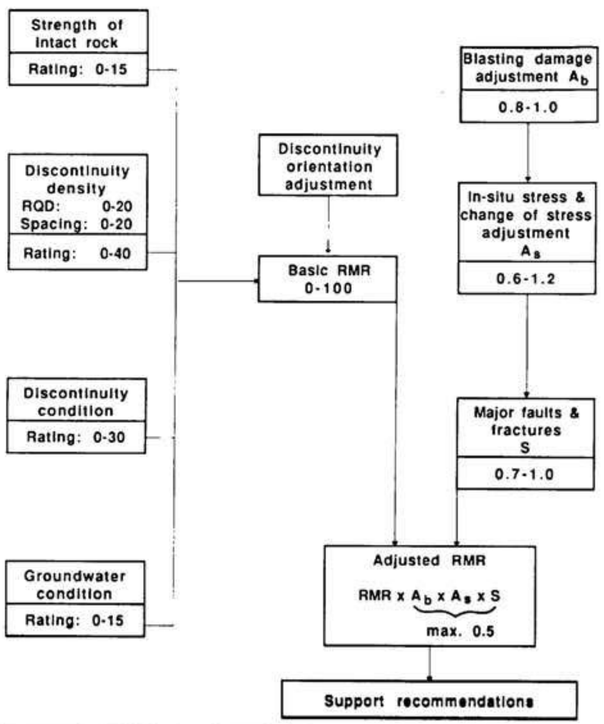
When mixed-quality rock conditions are encountered at the excavated rock face, such as good-quality and poor-quality rock being present in one exposed area, it is essential to identify the "most critical condition" for the assessment of the rock strata. This means that the geological features most significant for stability purposes will have an overriding influence. For example, a fault or shear in high-quality rock face will play a dominant role irrespective of the high rock material strength in the surrounding strata.

CHART E Guidelines for Classification of Discontinuity Conditions[a]

Parameter	Ratings				
Discontinuity length (persistence/continuity)	<1 m 6	1–3 m 4	3–10 m 2	10–20 m 1	>20 m 0
Separation (aperture)	None 6	<0.1 mm 5	0.1–1.0 mm 4	1–5 mm 1	>5 mm 0
Roughness	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
		Hard filling		Soft filling	
Infilling (gouge)	None 6	<5 mm 4	>5 mm 2	<5 mm 2	>5 mm 0
Weathering	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

[a] *Note:* Some conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge. In such cases, use Table 4.1 directly.

TABLE 4.3 Adjustments to the Rock Mass Rating System for Mining Applications



It is recommended that when there are two or more clearly different zones in one rock face, one approach to adopt is to obtain RMR values for each zone and then compute the overall weighted value by the surface area corresponding to each zone in relation to the whole area, as well as by the influence that each zone has on the stability of the whole excavation.

The Geomechanics Classification provides guidelines for the selection of rock reinforcement for tunnels in accordance with Table 4.4. These guidelines depend on such factors as the depth below surface (in-situ stress), tunnel size and shape, and the method of excavation. Note that the support measures given in Table 4.4 represent the permanent and not the primary support. Table 4.4 is applicable to rock masses excavated using conventional drilling and blasting procedures.

Most recently, Lauffer (1988) presented a revised stand-up time diagram specifically for tunnel boring machine (TBM) excavation and superimposed

it on the RMR diagram given in Figure 4.1. This is depicted in Figure 4.2, which is most useful because it demonstrates how the boundaries of RMR classes are shifted for TBM applications. Thus, an RMR adjustment can be made for machine-excavated rock masses.

Support load can be determined from the RMR system as proposed by Unal (1983):

$$P = \frac{100 - \mathrm{RMR}}{100} \gamma B \qquad (4.1)$$

where P = the support load, kN;
B = the tunnel width, m;
γ = the rock density, kg/m^3.

It must be emphasized that for all applications such as those involving the selection of rock reinforcement and determination of rock loads or rock mass strength and deformability, it is the actual RMR value that must be used and not the rock mass class, within which this RMR value falls. In

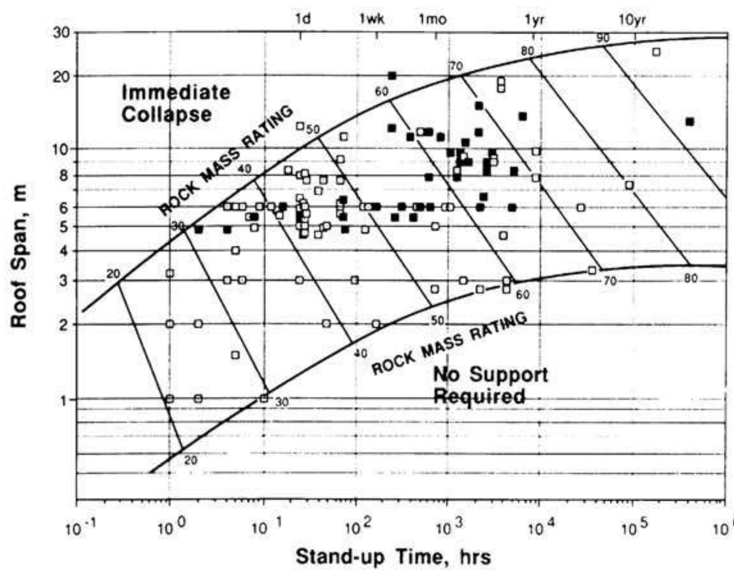


Figure 4.1 Relationship between the stand-up time and span for various rock mass classes, according to the Geomechanics Classification: output for tunneling and mining. The plotted data points represent roof falls studied: filled squares for mines, open squares for tunnels. The contour lines are limits of applicability.

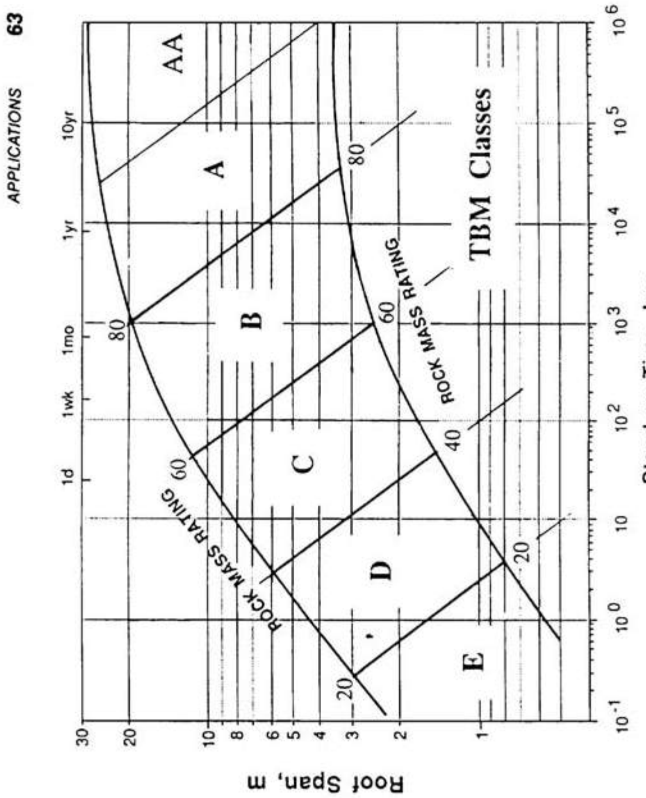


Figure 4.2 Modified 1988 Lauffer diagram depicting boundaries of rock mass classes for TBM applications. (After Lauffer, 1988.)

this way, the RMR system is very sensitive to individual parameters, because within one rock mass class, such as "good rock," there is much difference between RMR = 80 and RMR = 61.

Finally, note that the ranges in Table 4.1 follow the recommendations of the International Society of Rock Mechanics (ISRM) Commissions on Standardization and on Classification. The interested reader is referred to a document entitled *Suggested Methods for Quantitative Description of Discontinuities in Rock Masses* (ISRM, 1982).

4.2 APPLICATIONS

The Geomechanics Classification has found wide applications in various types of engineering projects, such as tunnels, slopes, foundations, and mines. Most of the applications have been in the field of tunneling (Bieniawski, 1984).

This classification system has been also used widely in mining, particularly in the United States, India, and Australia. Initially, Laubscher (1977) applied

TABLE 4.4 Guidelines for Excavation and Support of Rock Tunnels in Accordance with the Rock Mass Rating System[a]

Rock Mass Class	Excavation	Support: Rock Bolts (20-mm Dia, Fully Grouted)	Support: Shotcrete	Support: Steel Sets
Very good rock I RMR:81–100	Full face 3-m advance	Generally, no support required except for occasional spot bolting		
Good rock II RMR:61–80	Full face 1.0–1.5-m advance Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
Fair rock III RMR: 41–60	Top heading and bench 1.5–3-m advance in top heading Commence support after each blast Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
Poor rock IV RMR: 21–40	Top heading and bench 1.0–1.5-m advance in top heading. Install support concurrently with excavation 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock V RMR: <20	Multiple drifts 0.5–1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert

[a] Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

the Geomechanics Classification to asbestos mines in Africa. Most recently, the RMR system was applied to coal mining as well as to hard rock mining (Ghose and Raju, 1981; Abad et al., 1983; Unal, 1983; Kendorski et al., 1983; Newman, 1986; Venkateswarlu, 1986).

The Geomechanics Classification is also applicable to slopes (Romana, 1985) and to rock foundations (Bieniawski and Orr, 1976). This is a useful feature that can assist with the design of slopes near the tunnel portals as well as allow estimates of the deformability of rock foundations for such structures as bridges and dams.

In the case of rock foundations, knowledge of the modulus of deformability of rock masses is of prime importance. The Geomechanics Classification proved a useful method for estimating in-situ deformability of rock masses (Bieniawski, 1978). As shown in Figure 4.3, the following correlation was obtained:

$$E_M = 2\ \mathrm{RMR} - 100 \tag{4.2}$$

where E_M is the in-situ modulus of deformation in GPa and RMR is >50.

More recently, Serafim and Pereira (1983) provided many results in the range RMR < 50 and proposed a new correlation:

$$E_M = 10^{(\mathrm{RMR}-10)/40} \tag{4.3}$$

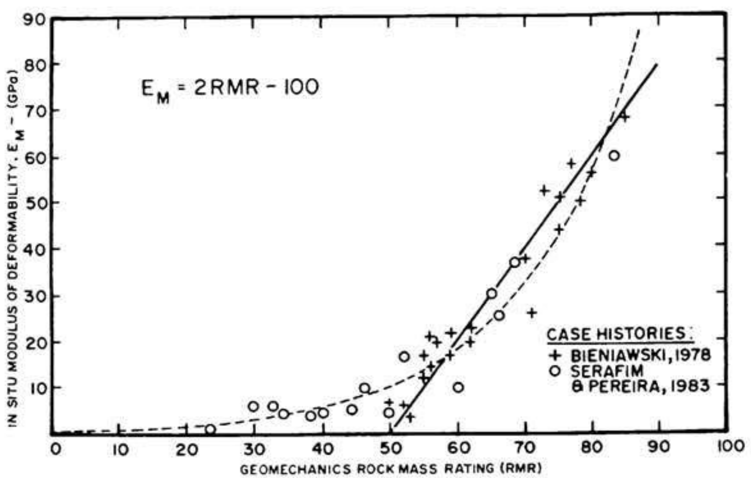


Figure 4.3 *Correlation between the in-situ modulus of deformation and RMR.*

In the case of slopes, the output is given in Section D of Table 4.1 as the cohesion and friction of the rock mass. Romana (1985) has applied the RMR system extensively for determination of rock slope stability.

Recently, Hoek and Brown (1980) proposed a method for estimating rock mass strength which makes use of the RMR classification. The criterion for rock mass strength is as follows:

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\frac{\sigma_3}{\sigma_c} + s} \tag{4.4}$$

where σ_1 = the major principal stress at failure,
σ_3 = the applied minor principal stress,
σ_c = the uniaxial compressive strength of the rock material,
m and s = constants dependent on the properties of the rock and the extent to which it has been fractured by being subjected to σ_1 and σ_3.

For intact rock, $m = m_i$, which is determined from a fit of the above equation to triaxial test data from laboratory specimens, taking $s = 1$ for rock material. For rock masses, the constants m and s are related to the basic (unadjusted) RMR as follows (Hoek and Brown, 1988):

For Undisturbed Rock Masses (smooth-blasted or machine-bored excavations):

$$m = m_i \exp\left(\frac{\mathrm{RMR} - 100}{28}\right) \tag{4.5}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{9}\right) \tag{4.6}$$

For Disturbed Rock Masses (slopes or blast-damaged excavations):

$$m = m_i \exp\left(\frac{\mathrm{RMR} - 100}{14}\right) \tag{4.7}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{6}\right) \tag{4.8}$$

Moreno Tallon (1982) developed a series of correlations between tunnel deformation, RMR, and time, based on a case history in Spain. Unal (1983) proposed an "integrated approach" to roof stability assessment in coal mines by incorporating RMR with roof span, support pressure, time, and deformation. This is diagrammatically depicted in Figure 4.4. Finally, recent research by Nicholson and Bieniawski (1986), incorporating the RMR system, proposed an empirical constitutive relationship for rock masses.

4.2.1 Strengths and Limitations

The RMR system is very simple to use, and the classification parameters are easily obtained from either borehole data or underground mapping (Gonzalez de Vallejo, 1983; Cameron-Clark and Budavari, 1981; Nakao et al., 1983).

This classification method is applicable and adaptable to many different situations, including coal mining, hard rock mining, slope stability, foundation stability, and tunneling.

The RMR system is capable of being incorporated into theoretical concepts, as is evident in the work of Unal (1983), Moreno Tallon (1982), Hoek and Brown (1980), and Nicholson and Bieniawski (1986).

The Geomechanics Classification is adaptable for use in knowledge-based expert systems. With the introduction of fuzzy-set methodology applied to the Geomechanics Classification by Nguyen and Ashworth (1985) and by Fairhurst and Lin (1985), the subjectiveness, or fuzziness, inherent in a classification can be considered and incorporated into the expert system.

The output from the RMR classification method tends to be rather conservative, which can lead to overdesign of support systems. This aspect is best overcome by monitoring rock behavior during tunnel construction and adjusting rock classification predictions to local conditions. An example of this approach is the work of Kaiser et al. (1986), who found that the no-support limit given in Figure 4.1 was too conservative and proposed the following correction to adjust RMR (No Support) at the no-support limit for opening size effects:

$$\text{RMR (NS)} = 22 \ln \text{ED} + 25 \qquad (4.9)$$

where ED is the equivalent dimension as defined by equation (5.2).

For the convenience of the user, a microcomputer program is listed in the Appendix for determination of the RMR and the resulting rock mass properties. An example of the output is included.

4.3 DATA BASE

The data base used for the development of a rock mass classification may indicate the range of its applicability. For example, the RMR system originally

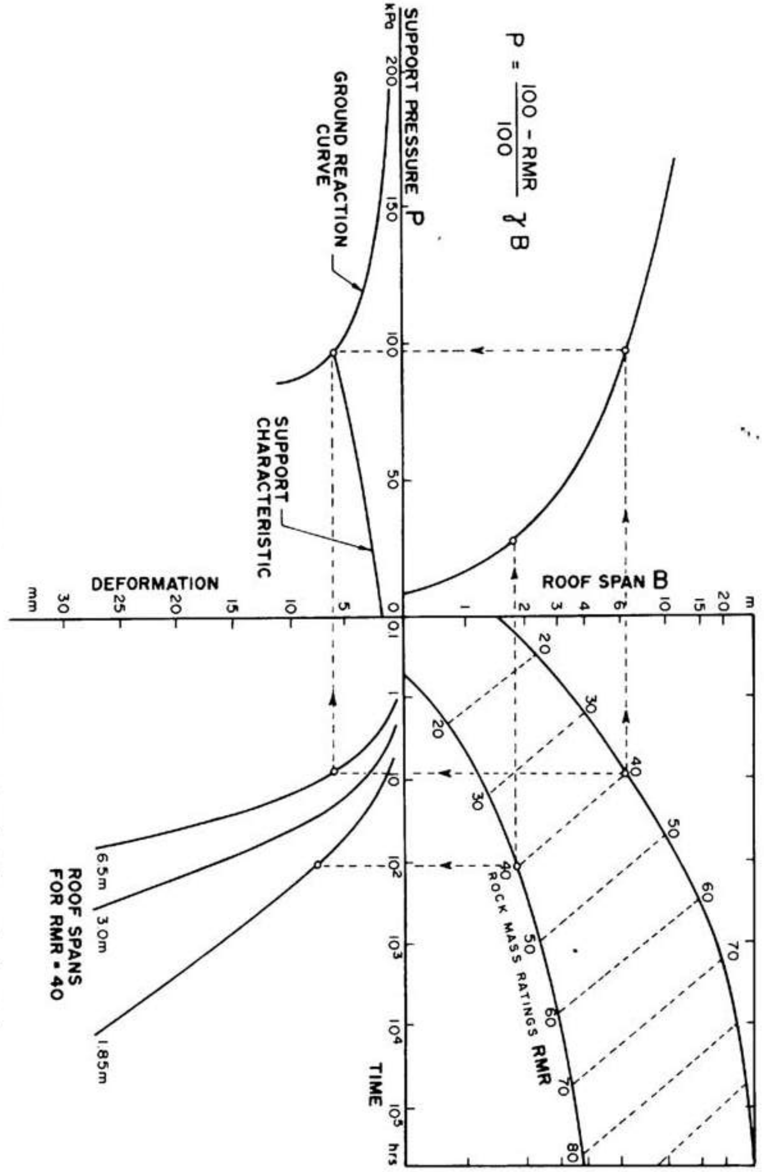


Figure 4.4 *Integration of RMR with support characteristics and roof deformation in coal mines. (After Unal, 1983.)*

involved 49 case histories (which were reanalyzed by Unal, 1983), followed by 62 case histories added by Newman and Bieniawski (1986) and a further 78 tunneling and mining case histories collected between 1984 and 1987. To date, according to the author's files, the RMR system has been used in 351 case histories (see Chap. 10). It was found that the system could be successfully used in rock formations not featured in the original case histories (Fowell and Johnson, 1982; Sandbak, 1985; Smith, 1986; Singh et al., 1986). At the same time, in some cases the system did not provide realistic results (Kaiser et al., 1986).

Nakao et al. (1983) made a significant contribution by performing a statistical reconsideration of the parameters for the Geomechanics Classification in order to apply the RMR system to Japanese geological conditions. In total, 152 tunnel cases were studied. It was found that the results of the parameter rating analysis "virtually agreed with that of the RMR concept."

Finally, the RMR classification—as any other—is not to be taken as a substitute for engineering design. This classification is only a part of the empirical design approach, one of the three main design approaches in rock engineering (empirical, observational, and analytical). It should be applied intelligently and used in conjunction with observational and analytical methods to formulate an overall design rationale compatible with the design objectives and site geology.

4.4 CORRELATIONS

A correlation was proposed between the RMR and the Q-index (Bieniawski, 1976) as well as between the RMR and the RSR (Rutledge and Preston, 1978). Based on 111 case histories analyzed for this purpose (involving 62 Scandinavian cases, 28 South African cases, and 21 case histories from the United States, Canada, Australia, and Europe), the following relationship was found for civil engineering tunnels (Bieniawski, 1976):

$$\mathrm{RMR} = 9 \ln Q + 44 \tag{4.10}$$

For mining tunnels, Abad et al. (1983) analyzed 187 coal mine roadways in Spain, arriving at this correlation:

$$\mathrm{RMR} = 10.5 \ln Q + 42 \tag{4.11}$$

Rutledge and Preston (1978) determined the following correlation from seven tunneling projects in New Zealand:

$$\mathrm{RSR} = 0.77\ \mathrm{RMR} + 12.4 \tag{4.12}$$

Moreno Tallon (1982) confirmed the above relationships on the basis of four tunneling projects in Spain. Jethwa et al. (1982) further substantiated the correlation by Bieniawski (1976) on the basis of 12 projects in India, whereas Trunk and Hönisch (1989) found an almost identical correlation to that given in equation (4.10) based on their study of tunnels in West Germany. For further discussion of these and other correlations, see Section 5.2.

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Appendix D Laubscher's MRMR-system

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A geomechanics classification system for the rating of rock mass in mine design

by D.H. LAUBSCHER*

SYNOPSIS

The mining rock-mass rating (MRMR) classification system was introduced in 1974 as a development of the CSIR geomechanics classification system to cater for diverse mining situations. The fundamental difference was the recognition that *in situ* rock-mass ratings (RMR) had to be adjusted according to the mining environment so that the final ratings (MRMR) could be used for mine design. The adjustment parameters are weathering, mining-induced stresses, joint orientation, and blasting effects.

It is also possible to use the ratings (RMR) in the determination of empirical rock-mass strength (RMS) and then in the application of the adjustments to arrive at a design rock-mass strength (DRMS). This classification system is versatile, and the rock-mass rating (RMR), the mining rock-mass rating (MRMR), and the design rock-mass strength (DRMS) provide good guidelines for the purposes of mine design. However, in some cases a more detailed investigation may be required, in which case greater attention is paid to specific parameters of the system.

Narrow and weak geological features that are continuous within and beyond the stope or pillar must be identified and rated separately.

The paper describes the procedure required to arrive at the ratings, and presents practical examples of the application of the system to mine design.

SAMEVATTING

Die mynrotsmassa-aanslag-klassifikasiestelsel (MRMR) is in 1974 ingevoer as 'n ontwikkeling van die WNNR se geomeganikaklassifikasiestelsel om vir uiteenlopende mynboutoestande voorsiening te maak. Die fundamentele verskil was die erkenning van die feit dat *in situ*-rotsmassa-aanslae (RMR) volgens die mynbou-omgewing aangesuiwer moes word om die finale aanslae (MRMR) vir mynontwerp te kan gebruik. Die aansuiweringsparameters is verwering, mynbougeïnduseerde spannings, naatoriëntasie en die gevolge van skietwerk.

Dit is ook moontlik om die aanslae (RMR) by die bepaling van empiriese rotsmassasterkte (RMS) te gebruik, en dan by die toepassing van aansuiwerings om 'n ontwerprotsmassasterkte (DRMS) te kry. Hierdie klassifikasiestelsel is veelsydig en die rotsmassa-aanslag (RMR), die mynrotsmassa-aanslag (MRMR), en die ontwerprotsmassasterkte (DRMS) verskaf goeie riglyne vir die doeleindes van mynontwerp. Daar kan egter in sommige gevalle 'n uitvoeriger ondersoek nodig wees waarin daar meer aandag aan spesifieke parameters van die stelsel geskenk word.

Smal en swak geologiese aspekte wat deurlopend is in en verby die afbouplek of pilaar, moet geïdentifiseer en afsonderlik aangeslaan word.

Die referaat beskryf die prosedure wat nodig is om die aanslae te kry en gee praktiese voorbeelde van die toepassing van die stelsel op mynbou-ontwerp.

INTRODUCTION

The classification system known as the mining rock-mass rating (MRMR) system was introduced in 1974 as a development of the CSIR geomechanics classification system[1,2]. The development is based on the concept of *in situ* and adjusted ratings, the parameters and values being related to complex mining situations. Since that time, there have been modifications and improvements[3-5], and the system has been used successfully in mining projects in Canada, Chile, the Philippines, Sri Lanka, South Africa, the USA, and Zimbabwe.

This paper consolidates the work presented in previous papers and describes the basic principles, data-collection procedure, calculation of ratings (RMR), adjustments (MRMR), design rock-mass strength (DRMS), and practical application of the systems.

An important development of this classification makes it suitable for use in the assessment of rock surfaces, as well as borehole cores.

Taylor[2] reviewed the classification systems developed by Wickham, Barton, Bieniawski, and Laubscher and concluded that

> Thus, the four systems chosen as being the most advanced classifications are based on relevant parameters. Each technique undoubtedly yields meaningful results, but only Laubscher's geomechanics classification and the 'Q' system of Barton offer suitable guidelines for the assessment of the main parameters: namely, the joint attributes. For general mining usage and where the application of a classification varies widely, Laubscher's geomechanics classification has the added advantage of allowing further adjustments to the rating for different situations. This, coupled with the fact that the technique has been in use for six years, gives no reason for changing to another system which offers no substantial improvements.

The figure below shows a 98 per cent correlation between the RMR of the MRMR system and the NGI system based on the classification by Taylor[2] of thirty sites ranging from very poor to very good. Thus, if NGI data are available, this information can be used in the practical applications.

PRINCIPLES

A classification system must be straightforward and have a strong practical bias so that it can form part of the normal geological and rock-mechanics investigations to be used for mine design and communication. Highly sophisticated techniques are time-consuming, and most

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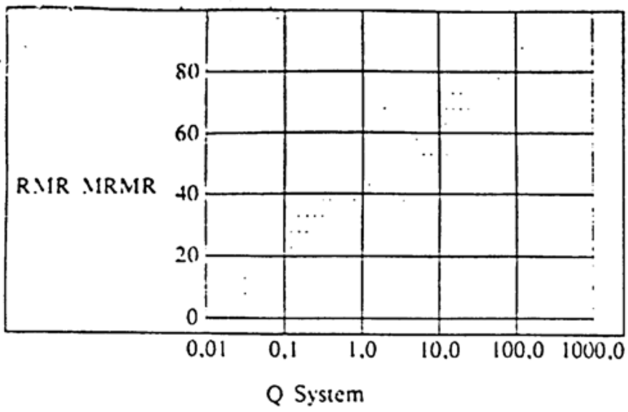


mines cannot afford the large staff required to provide complex data of doubtful benefit to the planning and production departments.

The approach adopted involves the assignment to the rock mass of an *in situ* rating based on measurable geological parameters. Each geological parameter is weighted according to its importance, and is assigned a maximum rating so that the total of all the parameters is 100. This weighting was reviewed at regular intervals in the development of the system and is now accepted as being as accurate as possible. The range of 0 to 100 is used to cover all variations in jointed rock masses from very poor to very good. The classification is divided into five classes with ratings of 20 per class, and with A and B sub-divisions.

A colour scheme is used to denote the classes on plan and section: class 1 blue, class 2 green, class 3 yellow, class 4 brown, and class 5 red. Class designations are for general use, and the ratings should be used for design purposes.

The ratings are, in effect, the relative strengths of the rock masses. The accuracy of the classification depends on the sampling of the area being investigated. The terminology *preliminary*, *intermediate*, and *final* should be applied to assessments to indicate the state of drilling and development. It is essential that classification data are made available at an early stage so that the correct decisions are made on mining method, layout, and support requirements.

In the assessment of how the rock mass will behave in a mining environment, the rock-mass ratings (RMR) are adjusted for weathering, mining-induced stresses, joint orientation, and blasting effects. The adjusted ratings are called the mining rock-mass ratings or MRMR.

It is also possible to use the ratings to determine an empirical rock-mass strength (RMS) in megapascals (MPa). The *in situ* rock-mass strength (RMS) is adjusted as above to give a design rock-mass strength (DRMS). This figure is extremely useful when related to the stress environment, and has been used for mathematical modelling.

The classification system is versatile, and the rock-mass rating (RMR), the mining rock-mass rating (MRMR), and the design rock-mass strength (DRMS) provide good guidelines for mine design purposes. However, in some cases where a more detailed investigation is required, examples of these situations are described in which specific parameters of the system are used.

Since average values can be misleading and the weakest zones may determine the response of the whole rock mass, these zones must be rated on their own. Narrow and weak geological features that are continuous within and beyond the stope or pillar must be identified and rated separately.

GEOLOGICAL PARAMETERS, SAMPLING, AND RATINGS

The geological parameters that must be assessed include the intact rock strength (IRS), joint/fracture spacing, and joint condition/water. Before the classification is done, the core or rock surface is examined and divided into zones of similar characteristics to which the ratings are then applied. These parameters and their respective ratings are shown in Table I.

Intact Rock Strength (IRS)

The IRS is the unconfined uniaxial compressive strength of the rock between fractures and joints. It is important to note that the cores selected for testwork are invariably the strongest pieces of that rock and do not necessarily reflect the average values; in fact, on a large copper mine, only unblemished core was tested. The IRS of a defined zone can be affected by the presence of weak and strong intact rock, which can occur in bedded deposits and deposits of varying mineralization. An average value is assigned to the zone on the basis that the weaker rock will have a greater influence on the average value. The relationship is non-linear, and the values can be read off an empirical chart (Fig. 1).

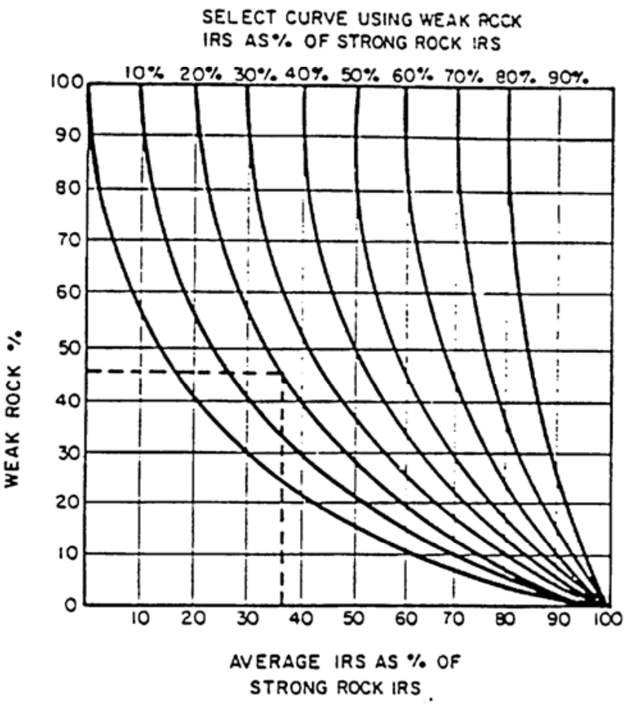


Fig. 1—Determination of average IRS where the rock mass contains weak and strong zones

Example:
Strong rock IRS = 100 MPa
Weak rock IRS = 20 MPa

$$\frac{\textbf{Weak rock IRS}}{\textbf{Strong rock IRS}} \times 100 = 20\%$$

Weak rock IRS = 45%
Average IRS = 37% of 100 MPa = 37 MPa

The rating range is from 0 to 20 to cater for specimen strengths of 0 to greater than 185 MPa. The upper limit of 185 MPa has been selected because IRS values greater than this have little bearing on the strength of jointed rock masses.

Spacing of Fractures and Joints (RQD + JS or FF)

Spacing is the measurement of all the discontinuities and partings, and does not include cemented features. Cemented features affect the IRS and as such must be included in that determination. A joint is an obvious feature that is continuous if its length is greater than the width of the excavation or if it abuts against another joint, i.e. joints define blocks of rock. Fractures and partings do not necessarily have continuity. A maximum of three joint sets is used on the basis that three joint sets will define a rock block; any other joints will merely modify the shape of the block.

Two techniques have been developed for the assessment of this parameter:

- the more detailed technique is to measure the rock quality designation (RQD) and joint spacing (JS) separately, the maximum ratings being 15 and 25 espectively;
- the other technique is to measure all the discontinuities and to record these as the fracture frequency per metre (FF/m) with a maximum rating of 40, i.e. the 15 and 25 from above are added.

Designation of Rock Quality (RQD)

The RQD determination is a core-recovery technique in which only cores with a length of more than 100 mm are recorded:

$$\text{RQD, \%} = \frac{\text{Total lengths of core} > 100\,\text{mm}}{\text{Length of run}} \times 100.$$

Only cores of at least BXM size (42 mm) should be used. It is also essential that the drilling is of a high standard.

The orientation of the fractures with respect to the core is important for, if a BXM borehole is drilled perpendicular to fractures spaced at 90 mm, the RQD is 0 per cent. If the borehole is drilled at an inclination of 40 degrees, the spacing between the same fractures is 137 mm; on this basis, the RQD is 100 per cent. As this is obviously incorrect, it is essential that the cylinder of the cores (sound cores) should exceed 100 mm in length. At the quoted 40 degree intersection, the core cylinder would be only 91 mm and the RQD 0 per cent. The length of core used for the calculation is measured from fracture to fracture along the axis of the core.

In the determination of the RQD of rock surfaces, the sampling line must be likened to a borehole core and the following points observed:

- experience in the determination of the RQD of core is necessary;
- do not be misled by blasting fractures;
- weaker bedding planes do not necessarily break when cored,
- assess the opposite wall where a joint forms the sidewall,
- shear zones greater than 1 m must be classified separately.

Joint Spacing (JS)

A maximum of a three-joint set is assumed, i.e. the number required to define a rock block. Where there are four or more joint sets, the three closest-spaced joints are used. The original chart for the determination of the JS rating has been replaced by that proposed by Taylor[4]. From the chart in Fig. 2 it is possible to read off the rating for one-, two-, and three-joint sets.

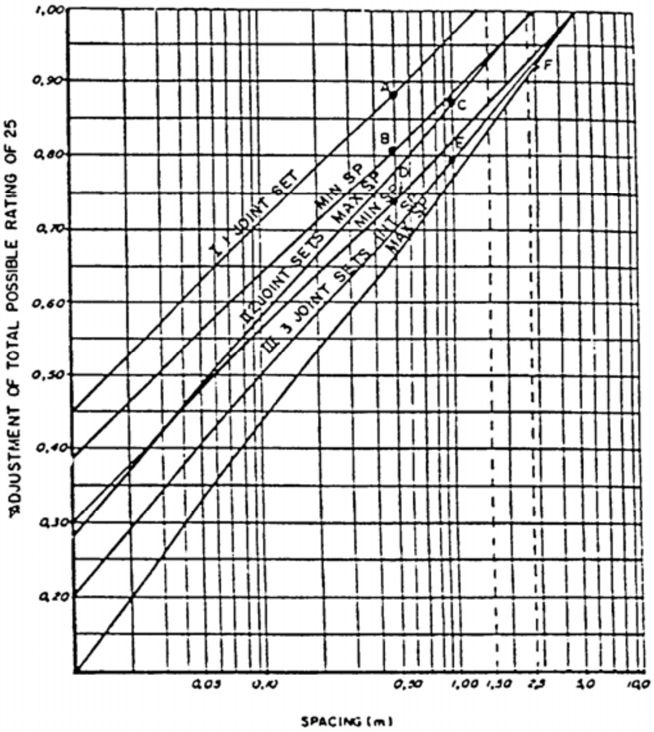


Fig. 2—Assessment of joint-space rating (after Taylor[4])

NB x = spacing × 100

A 1-Joint set $R = 25 \times ((26.4 \times \log_{10} x) + 45)/100$

B 2-Joint set $R = 25 \times ((25.9 \times \log_{10} x_{min}) + 38)/100 \times ((30.0 \times \log_{10} x_{max}) + 28)/100$

C 3-Joint set $R = 25 \times ((25.9 \times \log_{10} x_{min}) + 30)/100 \times ((29.6 \times \log_{10} x_{int}) + 20)/100 \times ((33.3 \times \log_{10} x_{max}) + 10)/100$

Example:

One set, spacing at 0,5 m = A, rating = 0.88 × 25 = 22

Two sets, spacing at 0,5 m and 1,0 m = B + C, rating = 0,81 × 0,86 × 25 = 17

Three sets, spacing at 0.5 m, 1,0 m, and 3.0 m = D, E + F, rating = 0,74 × 0,80 × 0,93 × 25 = 14

Fracture Frequency per Metre (FF/m)

This apparently simplified system requires the measurement of all the discontinuities that are intersected by the sampling line. It is important to determine whether a one-, two-, or three-joint system is being sampled. For the same FF/m, a rock mass with a one-joint set is stronger than one with a two-joint set, which is again stronger than one with a three-joint set. The rating allocation in Table I makes provision for the different joint sets.

In the case of core, it is also necessary to know whether only one or two joints of a three-joint system are intersected.

Underground measurement of fracture frequency is done on the sidewalls and hanging of drifts, tunnels, or stopes, depending on the orientation of the features. The

TABLE I

GEOLOGICAL PARAMETERS AND RATINGS

1. Meaning of the ratings

Class	1 A	1 B	2 A	2 B	3 A	3 B	4 A	4 B	5 A	5 B
Rating	100 – 81		80 – 61		60 – 41		40 – 21		20 – 0	
Description	Very Good		Good		Fair		Poor		Very Poor	
Colour	Blue		Green		Yellow		Brown		Red	

Distinguish between the A and B sub-classes by colouring the A sub-class full and cross-hatch the B.

2. Parameters and ratings

IRS–MPa	rating %	RQD	rating %	Joint spacing m	Fracture frequency, FF/m: Average per metre	Rating: 1 set	Rating: 2 set	Rating: 3 set
>185	20	97–100	15	0<—>25	0,1	40	40	40
165–185	18	84–96	14	See Fig. 2	0,15	40	40	40
145–164	16	71–83	12		0,20	40	40	38
125–144	14	56–70	10		0,25	40	38	36
105–124	12	44–55	8		0,30	38	36	34
85–104	10	31–43	6		0,50	36	34	31
65–84	8	17–30	4		0,80	34	31	28
45–64	6	4–16	2		1,00	31	28	26
35–44	5	0–3	0		1,50	29	26	24
25–34	4				2,00	26	24	21
12–24	3				3,00	24	21	18
5–11	2				5,00	21	18	15
1–4	1				7,00	18	15	12
					10,00	15	12	10
					15,00	12	10	7
					20,00	10	7	5
					30,00	7	5	2
					40,00	5	2	0
					ALLOW FOR CORE RECOVERY			

TABLE I (continued opposite)

TABLE II

FACTORS TO GIVE AVERAGE FRACTURE FREQUENCY

Sampling procedure	Factor
. One set of three sets on a line, or one set only	1,0
. Two sets of three sets on a line or two sets only	1,5
. All of the sets on a line or borehole core	2,0
. Two sets on one line and one on another	2,4
. Three sets on three lines at right-angles	3,0

TABLE III

BOREHOLE LOG SHEET

Borehole No: **Date:**

Zoning of borehole

Interval length (A)

Total sound core (B)

RQD, % $= \frac{B}{A} \times 100$

Joint spacing	Low angle 0–29	Number	
		Mean spacing (A)	
		True distance = A × sin (0,26)	
	Moderate angle 30–59	Number	
		Mean spacing (B)	
		True distance = B × sin (0,71)	
	High angle 60–90	Number	
		Mean spacing (C)	
		True distance = C × sin (0,97)	

Average frequency $= \frac{\text{Sum of individual FF/m (inverse of spacing)}}{2}$

	Final Rating
IRS	
RQD	
Joint spacing	
Joint condition	
Total	

Remarks

Sin values 0–29 = 0,26 30–59 = 0,71 60–90 = 0,97

Signature:

TABLE I (continued from opposite page)

3. Assessment of joint condition

Parameter	Description	Accumulative % adjustment of possible rating of 40 — Dry	Adjustment, % — Moist	Adjustment, % — Mod. pressure 25–125 1/m	Adjustment, % — High pressure >125 1/m
A Large-scale joint expression	Multi wavy directional	100	100	95	90
	Uni	95	90	85	80
	Curved	85	80	75	70
	Slight undulation	80	75	70	65
	Straight	75	70	65	60
B Small-scale joint expression 200 mm × 200 mm	Rough stepped/irregular	95	90	85	80
	Smooth stepped	90	85	80	75
	Slickensided stepped	85	80	75	70
	Rough undulating	80	75	70	65
	Smooth undulating	75	70	65	60
	Slickensided undulating	70	65	60	55
	Rough planar	65	60	55	50
	Smooth planar	60	55	50	45
	Polished	55	50	45	40
C Joint wall alteration weaker than wall rock and only if it is weaker than the filling		75	70	65	60
D Joint filling	Non-softening and sheared material — Coarse	90	85	80	75
	Non-softening and sheared material — Medium	85	80	75	70
	Non-softening and sheared material — Fine	80	75	70	65
	Soft sheared material, e.g. talc — Coarse	70	65	60	55
	Soft sheared material, e.g. talc — Medium	60	55	50	45
	Soft sheared material, e.g. talc — Fine	50	45	40	35
	Gouge thickness < amplitude of irregularities	45	40	35	30
	Gouge thickness > amplitude of irregularities	30	20	15	10

Example: A straight joint with a smooth surface and medium sheared talc under dry conditions gives A = 70%, B = 65%, D = 60%; total adjustment = 70 × 65 × 60 = 27%, and the rating is 40 × 27% = 11.

The rock mass rating (RMR) is the sum of the individual ratings.

following situations apply:

- all the features are present in the sidewalls, establish whether they intersect a horizontal line;
- if they all do not intersect the horizontal line, measure on a vertical line as well;
- if a set is parallel to the sidewall, measure these on a line in the hanging at right-angles to the sidewall.

This conflicting situation of different sampling procedures can be resolved if the sum of the measurements is divided by a factor to arrive at the average frequency. These factors are shown in Table II, which can be appreciated if compared with the sampling of the sides of a cube on different lines on intersection.

The need for accurate sampling cannot be too highly stressed. Often detailed scan-line surveys are done on sidewalls that do not intersect all the features, and then this biased information is analysed in detail.

Where boreholes do not intersect all the features at 45 degrees, a sampling bias will occur unless provision is made for the angle of intersection, as in the log sheet of Table III.

The average fracture frequency per metre (FF/m) is used in Table I to determine the rating. The inverse of this number gives the average fracture spacing. The data from A and B can be used only if the joint spacing for all the sets is approximately the same.

Fig. 3 shows the relationship between FF/m and ratings after the different sampling techniques for core and underground exposures have been adjusted to an average spacing.

Because the FF/m includes both continuous(joints) and discontinuous(fractures) features, the continuity must be estimated to give the joint spacing and rock block size (Fig. 4). Thus; the FF/m will give the rock-mass rating, but this has to be adjusted by the factors given in Table IV.

Core Recovery

As the FF/m does not recognize core recovery, the FF/m must be increased if there is a core loss, which will occur in the weaker sections of the core. The adjustment is done by dividing the FF/m by the core recovery and multiplying the quotient by 100.

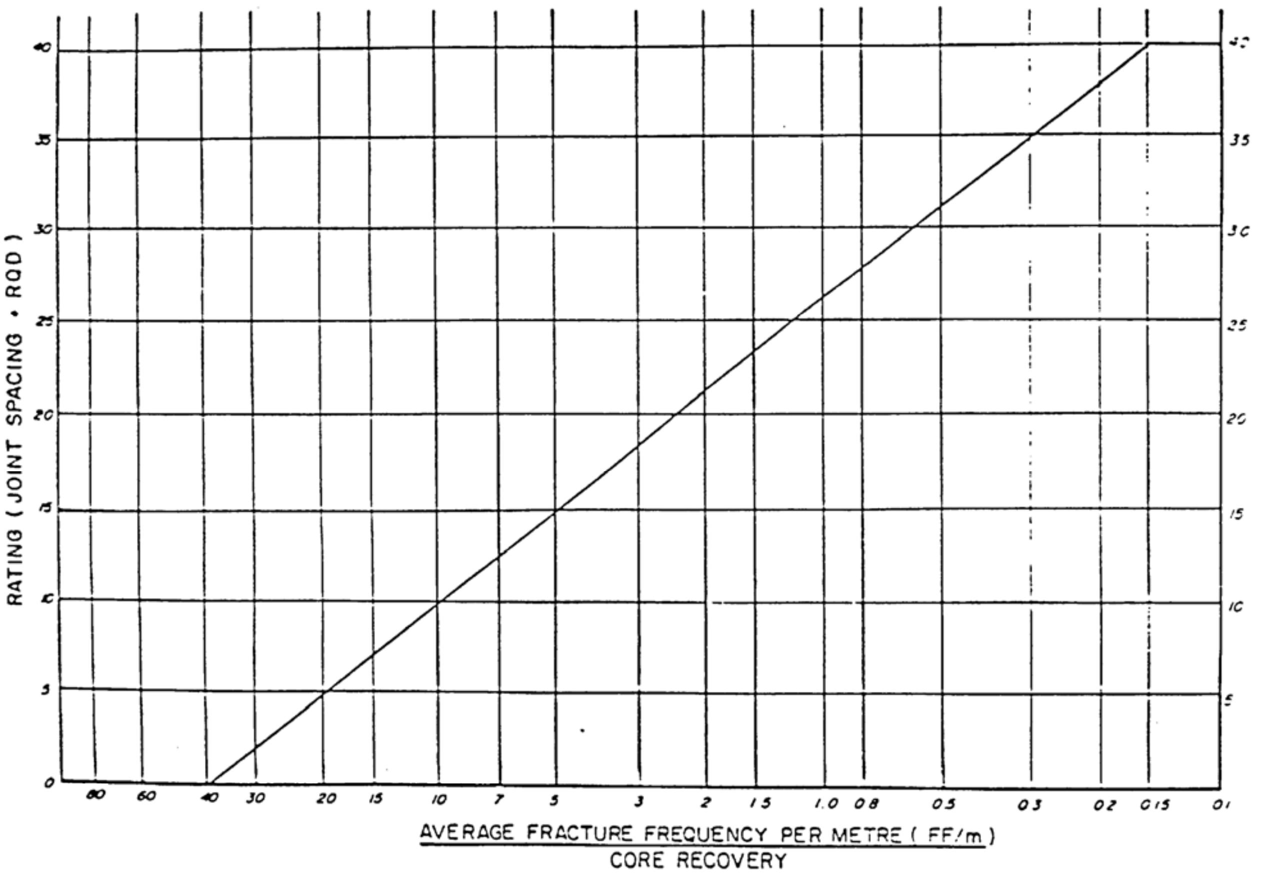


Fig. 3—Ratings for fracture frequency per metre

TABLE IV
FACTORS BY WHICH JOINT FREQUENCIES ARE MULTIPLIED

Continuous features %	Factor
100	1,0
90	0,9
80	0,8
70	0,7
60	0,6
50	0,5

TABLE V
COMPARISON OF TECHNIQUES

Joint spacing m	Rating RQD	Rating JS	Rating Combined	Rating FF/m
0,025	0	1	1	1
0,05	0	1,5	1,5	5
0,10	8	3	11	10
0,20	12	5	17	15
0,50	14	10	24	20
1,00	15	13	28	26
2,00	15	19	34	31
3,00	15	21	36	33
4,00	15	23	38	36
5,00	15	25	40	38

Comparison of the Two Techniques

The advantage of the FF/m technique is that it is more sensitive than the RQD for a wide range of joint spacings, because the latter measures only core less than 100 mm and rapidly changes to 100 per cent. Examples of this are shown in Table V, which assumes that there is a percentage of core greater than 100 mm at joint intersections.

The fracture-frequency technique was first used in Chile in 1985 and then in Canada in 1986. In Zimbabwe, the FF/m technique was used in conjunction with the RQD and JS technique and was found to be just as accurate.

Joint Condition and Water

Joint condition is an assessment of the frictional properties of the joints (not fractures) and is based on expression, surface properties, alteration zones, filling, and water. Originally the effect of water was catered for in a separate section: however, it was decided that the assessment of joint condition allowing for water inflow would have greater sensitivity[3]. A total rating of 40 is now assigned to this section. The procedure for the determination of joint condition is shown in Table I, which divides the joint-assessment section into sub-sections A, B, C, D.

Sub-section A caters for the large-scale expression of the feature, such as across a drift or in a pit face. B assesses the small-scale expression and is based on the profiles shown in Fig. 5. Section C is applied only when there is a distinct difference between the hardness of the host rock and that of the joint wall. Section D covers the variations in joint filling.

As the conditions of the different joint sets are not

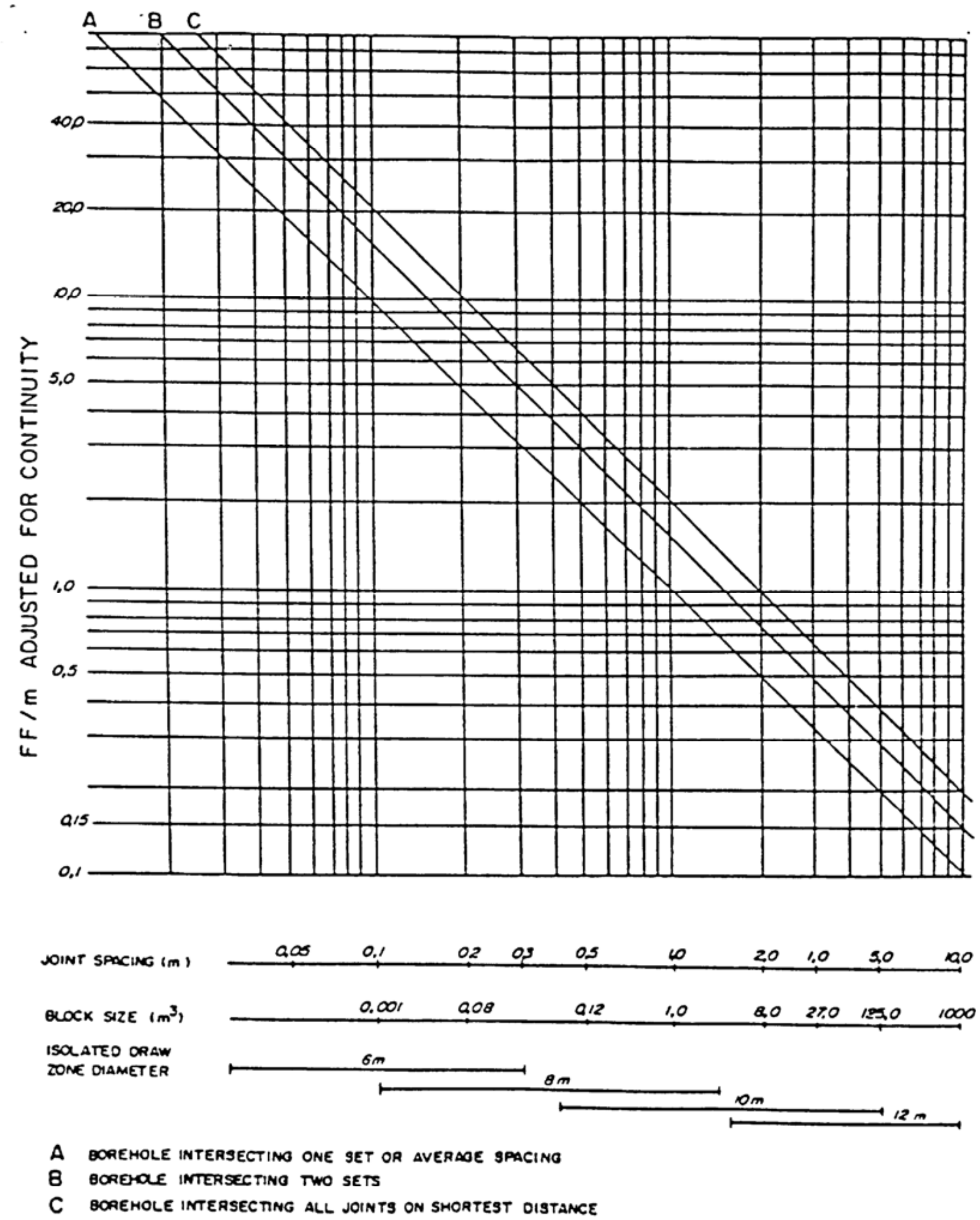


Fig. 4—Joint-spacing diagram

necessarily the same, a weighted average has to be calculated. However, if there is a significant difference in the condition ratings, this should be highlighted in the text or on the plans. A low rating for one joint set could influence the orientation of tunnels and/or the mining sequence.

When there is a preponderance of crosscuts over drives or *vice versa*, a sampling bias can occur, resulting in preference being given to those features that intersect the dominant drifts at a large angle.

ADJUSTMENTS

The RMR is multiplied by an adjustment percentage to give the MRMR. The adjustment percentages are empirical, having been based on numerous observations in the field. The adjustment procedure requires that the engineer assess the proposed mining activity in terms of its effect on the rock mass. For example, poor blasting influences the stability of a drift or pit slope but has no influence on the cavability of the rock mass.

It has been found that there is a better appreciation of the operation when planning personnel have to think in terms of adjustments. The adjustment concepts developed for the MRMR system were used by Engineers International, Inc. to prepare a classification of caving-mine rock mass and a support estimation system[7].

Weathering

Certain types of rock weather readily, and this must be taken into consideration in decisions on the size of opening and the support design. Weathering is time-dependent, and influences the timing of support installa-

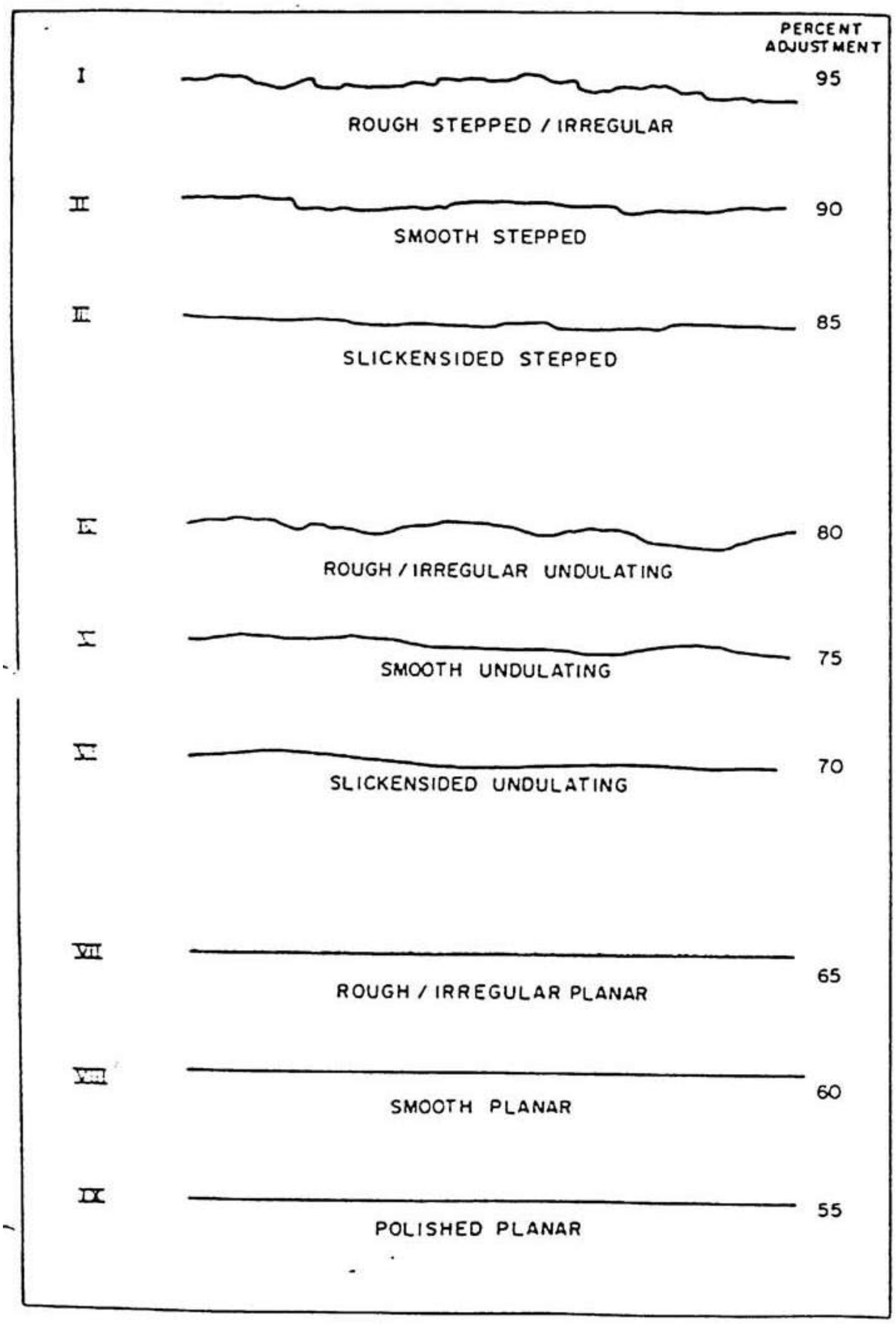


Fig. 5—Joint roughness profiles

tion and the rate of mining.

The three parameters that are affected by weathering are the IRS, RQD or FF/m, and joint condition. The RQD percentage can be decreased by an increase in fractures. The IRS can decrease significantly as chemical changes take place; in fact, there is the situation with kimberlites, where solid hard rock becomes sand in a short time. The joint condition is affected by alteration of the wallrock and the joint filling. Weathering data based on the examination of borehole cores can be conservative owing to the large surface area of core relative to the volume—underground exposures are more reliable.

Table VI shows the adjustment percentages related to degree of weathering after a period of exposure of half, one, two, three, and four-plus years.

TABLE VI
ADJUSTMENTS FOR WEATHERING

Degree of weathering	Potential weathering and adjustments, %				
	½ y	1 y	2 y	3 y	4 + y
Fresh	100	100	100	100	100
Slight	88	90	92	94	96
Moderate	82	84	86	88	90
High	70	72	74	76	78
Complete	54	56	58	60	62
Residual soil	30	32	34	36	38

Joint Orientation

The size, shape, and orientation of an excavation affects the behaviour of the rock mass. The attitude of the joints, and whether or not the bases of blocks are exposed, have a significant bearing on the stability of the excavation, and the ratings must be adjusted accordingly. The magnitude of the adjustment depends on the attitude of the joints with respect to the vertical axis of the block. As gravity is the most significant force to be considered, the instability of the block depends on the number of joints that dip away from the vertical axis. The required adjustments are shown in Table VII.

TABLE VII

PERCENTAGE ADJUSTMENTS FOR JOINT ORIENTATION

No. of joints defining the block	No. of faces inclined away from the vertical				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6	5	4	3	2,1

The orientation of joints has a bearing on the stability of open stopes and the cavability of undercut rock masses.

The adjustments for the orientation of shear zones with respect to development are as follows: 0–15° = 76%, 15–45° = 84%, 45–75° = 92%.

Advance of the ends in the direction of dip of structural features is preferable to development against the dip. An adjustment of 90 per cent should be made to previous adjustments when the advance is against the dip of a set of closely spaced joints. This is because it is easier to support rock blocks that have the prominent joints dipping with the advance.

The adjustment for shear-zone orientation does not apply to 'jointed rock'. The maximum rating is therefore joint orientation multiplied by direction of advance, which is 70% × 90% = 63%.

The effect of joint orientation and condition on stability is clearly displayed in bridge arches made from high-friction rock blocks.

Joint-orientation Adjustment for Pillars and Sidewalls

A modified orientation adjustment applies to the design of pillars or stope sidewalls. Adjustments are made where joints define an unstable wedge with its base on the sidewall. The instability is determined by the plunge of the intersection of the lower joints, as well as by the condition of the joints that define the sides of the wedge (Table VIII).

Mining-induced Stresses

Mining-induced stresses result from the redistribution of field(regional) stresses that is caused by the geometry and orientation of the excavations. The magnitude and ratio of the field stresses should be known. The redistribution of the stresses can be obtained from modelling or from published stress-redistribution diagrams[7,8]. The redistributed stresses that are of interest are maximum, minimum, and differences.

TABLE VIII

PERCENTAGE ADJUSTMENTS FOR THE PLUNGE OF THE INTERSECTION OF JOINTS ON THE BASE OF BLOCKS

Average rating	Plunge degree	Adjustment %	Plunge degree	Adjustment %	Plunge degree	Adjustment %
0–5	10–30	85	30–40	75	>40	70
5–10	10–20	90	20–40	80	>40	70
10–15	20–30	90	30–50	80	>50	75
15–20	30–40	90	40–60	85	>60	80
20–30	30–50	90	>50	85		
30–40	40–60	90	>50	90		

Maximum Stress

The maximum principal stress can cause spalling of the wall parallel to its orientation, the crushing of pillars, and the deformation and plastic flow of soft zones. The deformation of soft intercalates leads to the failure of hard zones at relatively low stress levels. A compressive stress at a large angle to joints increases the stability of the rock mass and inhibits caving. In this case, the adjustment can be up to 120 per cent, i.e. improving the strength of the rock mass.

Minimum Stress

The minimum principal stress plays a significant role in the stabilities of the sides and back of large excavations, the sides of stopes, and the major and minor apexes that protect extraction horizons. The removal of a high horizontal stress on a large stope sidewall will result in relaxation of the ground towards the opening.

Stress Differences

A large difference between maximum and minimum stresses has a significant effect on jointed rock masses, resulting in shearing along the joints. The effect increases as the joint density increases (since more joints will be unfavourably orientated) and also as the joint-condition ratings decrease. The adjustment can be as low as 60 per cent.

Factors in the Assessment of Mining-induced Stress

The following factors should be considered in the assessment of mining-induced stresses:

- drift-induced stresses;
- interaction of closely spaced drifts;
- location of drifts or tunnels close to large stopes;
- abutment stresses, particularly with respect to the direction of advance and orientation of the field stresses (an undercut advancing towards maximum stress ensures good caving but creates high abutment stresses, and *vice versa*);
- uplift;
- point loads from caved ground caused by poor fragmentation;
- removal of restraint to sidewalls and apexes;
- increases in size of mining area causing changes in the geometry;
- massive wedge failures;
- influence of major structures not exposed in the excavation but creating the probability of high toe stresses or failures in the back of the stope;

- presence of intrusives that may retain high stress or shed stress into surrounding, more competent rock.

The total adjustment is from 60 to 120 per cent. To arrive at the adjustment percentage, one must assess the effect of the stresses on the basic parameters and use the total.

Blasting Effects

Blasting creates new fractures and loosens the rock mass, causing movement on joints, so that the following adjustments should be applied:

Technique	*Adjustment, %*
Boring	100
Smooth-wall blasting	97
Good conventional blasting	94
Poor blasting	80.

The 100 per cent adjustment for boring is based on no damage to the walls; however, recent experience with roadheader tunnelling shows that stress deterioration occurs a short distance from the face. This phenomenon is being investigated since good blasting may create a better wall condition.

It should be noted that poor blasting has its greatest effect on narrow pillars and closely spaced drifts owing to the limited amount of unaffected rock.

Summary of Adjustments

Adjustments must recognize the life of the excavation and the time-dependent behaviour of the rock mass:

Parameter	*Possible adjustment, %*
Weathering	30–100
Orientation	63–100
Induced stresses	60–120
Blasting	80–100.

Although the percentages are empirical, the adjustment principle has proved sound and, as such, it forces the designer to allow for these important factors.

STRENGTH OF THE ROCK MASS

The rock-mass strength (RMS) is derived from the IRS and the RMR[4]. The strength of the rock mass cannot be higher than the corrected average IRS of that zone. The IRS has been obtained from the testing of small specimens, but testwork done on large specimens shows that their strengths are 80 per cent of those of small specimens[3]. As the rock mass is a 'large' specimen, the IRS must be reduced to 80 per cent of its value. Thus, the strength of the rock mass would be IRS × 80% if it had no joints! The effect of the joints and its frictional properties is to reduce the strength of the rock mass.

The following procedure is adopted in the calculation of RMS:

- the IRS rating(B) is subtracted from the total rating(A) and, therefore, the balance, i.e. RQD, joint spacing, and condition are a function of the remaining possible rating of 80;
- the IRS(C) is reduced to 80 per cent of its value.

$$\text{RMS} = \frac{(A-B)}{80} \times C \times \frac{80}{100},$$

e.g. if the total rating was 60 with an IRS of 100 MPa and a rating of 10, then

$$\text{RMS} = 100\,\text{MPa} \times \frac{(60-10)}{80} \times \frac{80}{100} = 50\,\text{MPa}.$$

DESIGN STRENGTH OF THE ROCK MASS

The design rock-mass strength (DRMS) is the strength of the unconfined rock mass in a specific mining environment. A mining operation exposes the rock surface, and the concern is with the stability of the zone that surrounds the excavation. The extent of this zone depends on the size of the excavation and, except with mass failure, instability propagates from the rock surface. The size of the rock block will generally define the first zone of instability. Adjustments, which relate to that mining environment, are applied to the RMS to give the DRMS. As the DRMS is in megapascals, it can be related to the mining-induced stresses. Therefore, the adjustments used are those for weathering, orientation, and blasting. For example, if

weathering = 85%, orientation = 75%, blasting = 90%, total = 57%, and RMS = 50, the adjustment = 57% and the DRMS = 50 × 57% = 29 MPa.

Therefore, the rock mass has an unconfined compressive strength of 29 MPa, which can be related to the total stresses.

PRESENTATION OF DATA

The rating data for the rock mass should be plotted on plans and sections as class or sub-class zones. If the A and B sub-divisions are used, the A zones can be coloured full and the B cross-hatched. These plans and sections now provide the basic data for mine design. The layouts are plotted with the adjusted ratings (MRMR), which will highlight potential problem areas or, if the layout has been agreed, the support requirements will be based on the MRMR or DRMS. In the case of the DRMS, the values can be contoured.

Practical Applications

The rock mass can now be described in ratings or in megapascals; in other words, these numbers define the strength of the material in which the mining operation is going to take place. Excavation stability or instability has been related to these numbers. On the mines in which the system has been in operation, its introduction was welcomed by all departments from those dealing with geology to those involved in production.

Within the scope of this paper, the practical applications are described in broad terms to indicate the benefits achieved from the use of this system.

Communication

Communication between various departments has improved since the introduction of the classification system because numbers are used instead of vague descriptive terms. It is well known that the terminology used to describe a particular rock mass by personnel experienced in the mining of good ground is not the same as that used by personnel experienced in the mining of poor ground.

Support Principles

The RMR is taken into consideration in designing support even though the adjusted ratings (MRMR) are used. The reason is that a class 3A adjusted to 5A has reinforcement potential, whereas an *in situ* class 5A has no reinforcing potential.

Support is required to maintain the integrity of the rock mass and to increase the DRMS so that the rock mass can support itself in the given stress environment. The installation must be timed so that the rock mass is not allowed to fail and should therefore be early rather than late. A support system should be designed and agreed before the development stage so that there is interaction between the components of the initial and the final stages. To control deformation and to preserve the integrity of the rock mass, the initial support should be installed concurrently with the advance. The final support caters for the mining-induced stresses.

An integrated support system consists of components that are interactive, and the success of the system depends on the correct installation and the use of the right material. Experience has shown that simple systems correctly installed are more satisfactory than complicated techniques in which the chances of error are higher. The supervisory staff must understand and contribute to the design, and design staff must recognize the capabilities of the construction crews and any logistical problems. The construction crews should have an understanding of the support principles and the consequences of poor installation.

Layout of Support Guide for Tunnels Using MRMR

Table IX shows how the support techniques, in alphabetical symbols, increase in support pressure as the MRMR decreases. Both the RMR and the MRMR are shown as sub-classes.

TABLE IX
SUPPORT* PRESSURE FOR DECREASING MRMR

	RMR									
MRMR	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
	← Rock reinforcement—plastic deformation →									
1A										
1B										
2A										
2B	a	a								
3A	b	b	a	a						
3B	b	b	b	b	b	c				
4A	r	r	c	c	c	d	d			
4B				d	e	f	f	c – l		
5A						f/p	h – f/p	h – f/l	h – f/l	
5B							h – f/p	f p	t	t

* The codes for the various support techniques are given in Table X.

Adjusted ratings must be used in the determination of support requirements. In specialized cases, such as draw-point tunnels, the attrition effects of the drawn caved rock and secondary blasting must be recognized, in which case the tunnel support shown in Table IX would be supplemented by a massive lining.

The support techniques shown in Table X are examples of a progressive increase in support pressures and are not a complete spectrum of techniques. Where weathering is likely to be a problem, the rock should be sealed on exposure.

TABLE X
SUPPORT TECHNIQUES

Rock reinforcement		
	a	Local bolting at joint intersections
	b	Bolts at 1 m spacing
	c	b and straps and mesh if rock is finely jointed
	d	b and mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
	e	d and straps in contact with or shotcreted in
	f	e and cable bolts as reinforcing and lateral restraint
	g	f and pinning
	h	Spilling
	i	Grouting
Rigid lining		
	j	Timber (Low deformation)
	k	Rigid steel sets (Low deformation)
	l	Massive concrete (Low deformation)
	m	k and concrete (Low deformation)
	n	Structurally reinforced concrete
Yielding lining, repair technique, high deformation		
	o	Yielding steel arches
	p	Yielding steel arches set in concrete or shotcrete
Fill		
	q	Fill
Spalling control		
	r	Bolts and rope-laced mesh
Rock replacement		
	s	Rock replaced by stronger material
	t	Development avoided if possible

Layout of Support Guide Using the DRMS

The support guide for tunnels using the DRMS and the support techniques of Table IX are shown in Figs. 6 and 7.

Stability and Cavability

The relationship between the ratings adjusted for stability or instability (MRMR) and the size of excavation is shown in Fig. 8. The examples of different situations were taken from operations at the following mines:

- Freda, Gaths, King, Renco, and Shabanie Mines in Zimbabwe
- Andina, Mantos Blancos, and Salvador Mines in Chile
- Bell and Fox Mines in Canada
- Henderson Mine in the USA.

The diagram refers to the stability of the rock arch, which is depicted in three empirical zones:

- a stable zone requiring support only for key blocks or brows, i.e. skin effects
- a transition zone requiring substantial penetrative support and/or pillars, or provision to be made for dilution owing to failure of the intradosal zone.

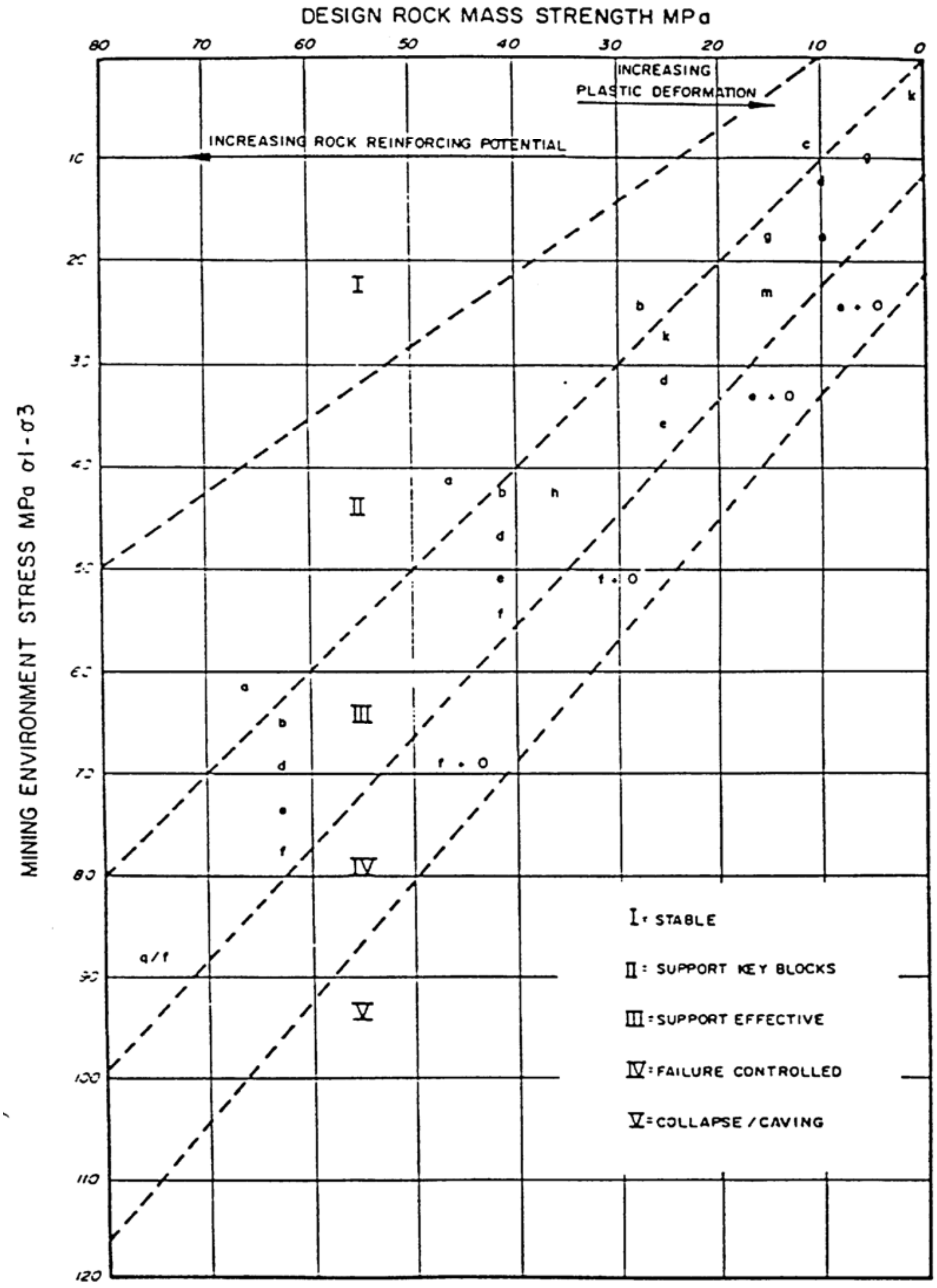


Fig. 6—Support requirements for maximum stress

- a caving or subsidence zone in which caving is propagated provided space is available or subsidence occurs.

The size of the excavation is defined by the 'hydraulic radius' or stability index, which is the plan area divided by the perimeter. Only the plan area is used for excavations where the dip of the stope or cave back is less than 45 degrees. Where the dip is greater than 45 degrees, the area and orientation of the back with respect to the major stress direction must be assessed.

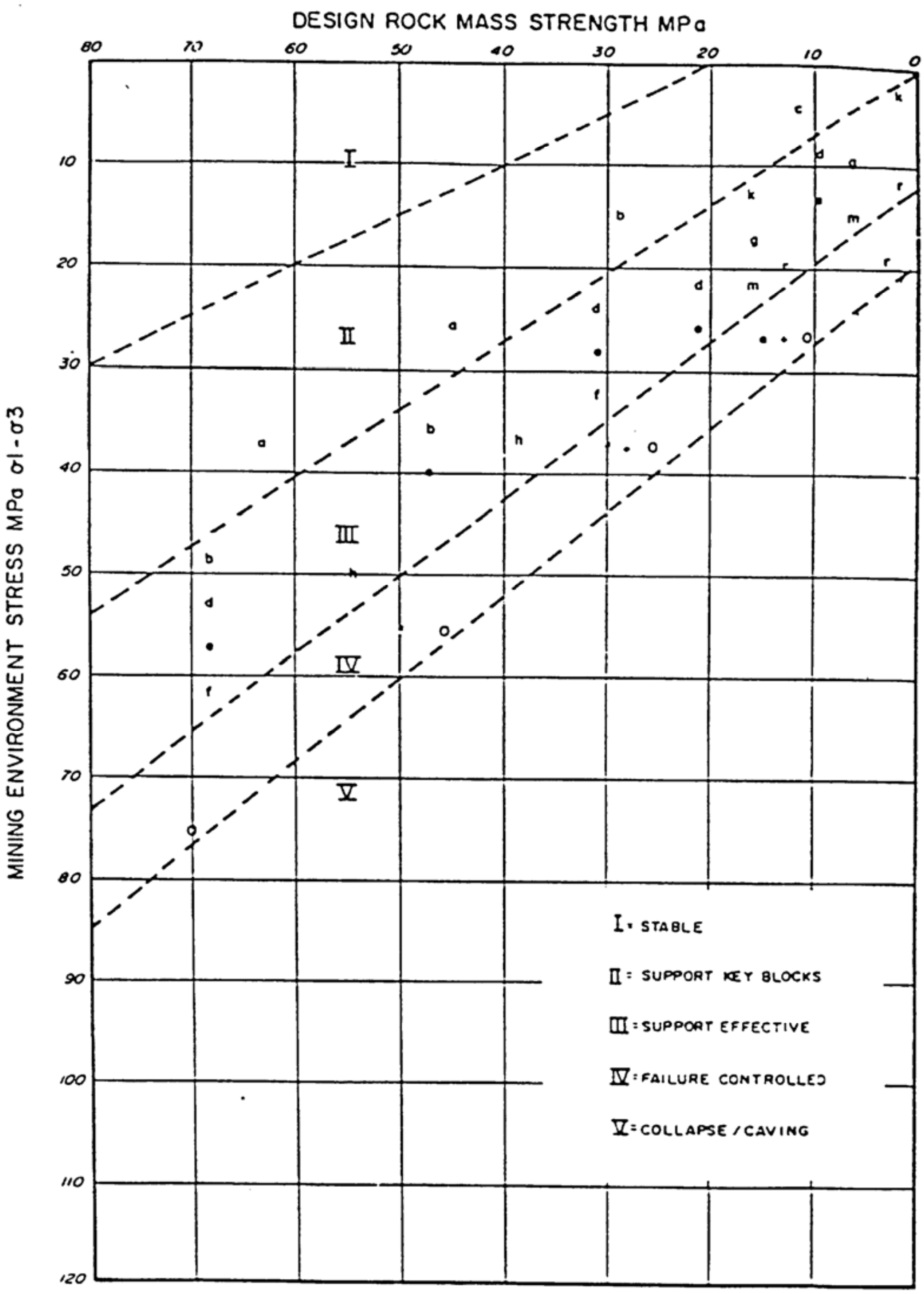
For the same area, the stability index (SI) will vary depending on the relationship between the maximum and the minimum spans. For example, 50 m × 50 m has the same area as 500 m × 5 m, but the SI of the first is 12,5 whereas the SI of the second is only 2,5. The large 50 m × 50 m stope is less stable than a 500 m × 5 m tunnel, and this is well illustrated by the difference in the SI.

Indestructible pillars(regional) reduce the spans so that the SI is applied to individual stopes. Small pillars, as in a post-pillar operation, apply a restraint to the hangingwall, which results in a positive adjustment and, as such, a higher rating, so that the overall stope dimensions can be increased within the dictates of regional stability.

In a room-and-pillar mine, the pillars are designed to ensure regional stability.

The stability or cavability of a rock mass is determined by the extent and orientation of the weaker zones.

There is a distinction between massive and bedded



—Support requirements fo. various stress differences

deposits in that the bedding could be a dominant feature.

Stability of Open Stopes

Large, open stopes are generally mined in competent ground, the size of the stope being related to the criteria for regional stability (Fig. 8). The stability of the stope hangingwall has to be assessed in terms of whether the personnel are to work in the stope or not.

If personnel are to work in the stope, the back must be stable immediately after mining. In order to achieve this stability, potential rock falls need to be identified and dealt with. If the environment and mining rate permit it, or if skilled personnel are available, the support can be designed for the local situation. However, if the mining rate is high, or the identification of potential falls is difficult, a blanket-support design is required.

In the case of open stopes where the activity is from sub-levels outside the stope, the local instability affects the amount of dilution before a stable arch has formed. In the worst situation, the intradosal zone can have a height that is 25 per cent of the span.

By use of a combination of joint-condition ratings and joint-orientation data, a condition/orientation percentage can be derived. These percentages are shown in Table XI.

These percentages can be used to define areas requiring support as follows:

60% – 70% Highly unstable, collapse with blast, requires presupport

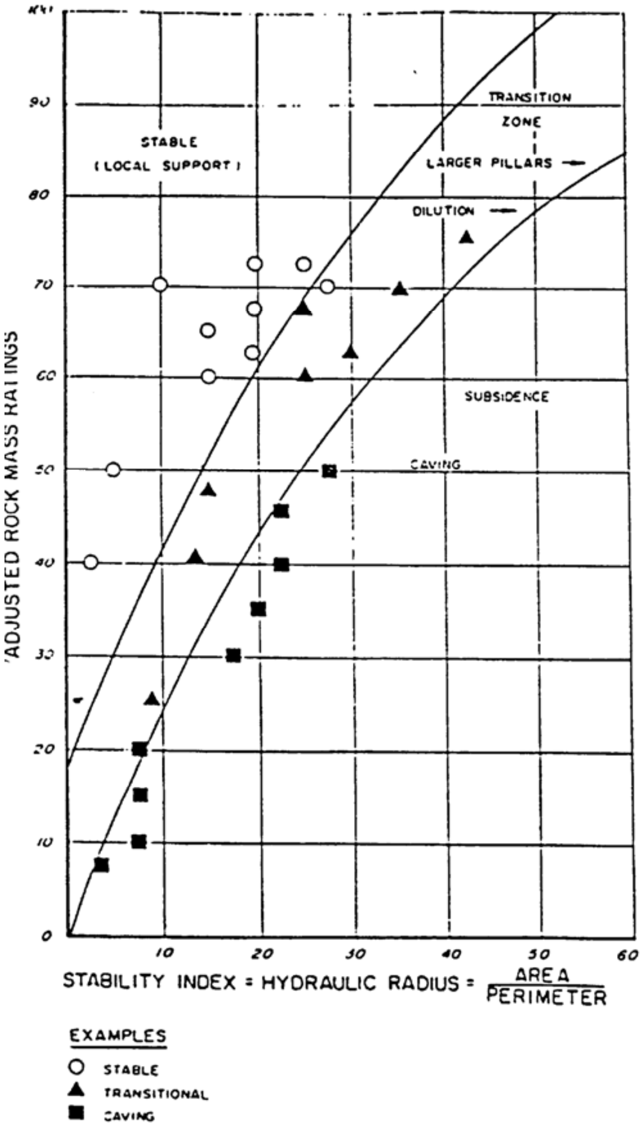


Fig. 8—Stability/instability diagram

70% – 80%	Unstable, time-dependent falls, may require presupport
80% – 90%	Relatively stable, requires support or scaling, even light blasting
90% – 100%	Stable.

In the case of bedded deposits, thinly bedded and massively bedded zones must be rated as distinct units. Bed separation occurs in the thinly bedded zones, while the massive zones contribute to the stability.

Cavability

The joint patterns bear directly on the cavability and fragmentation of the rock mass, and can be used in assessments of whether a cave-mining method can be employed. It is imperative that the hangingwall zone for at least the height of the orebody should be classified. Diagrams like that shown in Fig. 7 are used to define the undercut area for different rock masses.

Fragmentation

Caving results in primary fragmentation, which is the particle size developed in the failure zone of an advancing cave. Primary fragmentation is determined by the stresses in the cave back and by the strength(condition) and orientation of the joints with respect to those stresses. The size of the potential rock blocks is based on the adjusted FF/m in Fig. 4.

Secondary fragmentation is the breaking up of the primary rock block in the draw column. For comminution to occur, the stresses generated must exceed the strength of the rock block, which is unlikely if the block is moving and cushioned by finer material or softer rocks. This is evident in heterogeneous orebodies that contain classes 3, 4, and 5. In these cases, class 4 and 5 zones fragment readily, but class 3 zones arrive at the drawpoint as large blocks even though they contain joints with high joint-condition ratings.

Extent of Cave and Failure Zones

The result of a block cave is the formation of a zone of caved material that has differential rates of movement within boundaries defined by the cave angle. Beyond the cave boundary, a failure zone is developed with fractures(cracks) and limited movement. As shown in Table XII, the strength of the rock mass, the amount of drawdown, and the major structures dictate the angle of the cave and the extent of the failure zone.

Mining Method as Related to MRMR

Table XIII shows how the MRMR varies with mining method.

Pillar Design

Pillars are designed to ensure regional stability or local support in stopes and along drifts, or to yield under a measure of control. In all cases, the strength of the material and the variations in strength must be known both for the pillar and for the roof and floor. The shape of the pillar with respect to structure, blasting, and stresses is significant, and is catered for by the adjustment procedure. For example, for a width-to-height ratio of less than 4,5:1, the following formula uses SI and DRMS[8]:

$$\text{Pillar strength } Ps = k\,\frac{W^{0,5}}{H^{0,7}},$$

where

$$k = \text{DRMS in MPa},\ W = 4 \times \frac{\text{Pillar area}}{\text{Pillar perimeter}}\ (\text{SI}),$$

$$H = \text{height}.$$

Initial Design of Pit Slopes

Table XIV can be used in the design of the initial pit slopes. If the rock mass is homogeneous, the angles shown are comparatively accurate. However, in a heterogeneous rock mass, the classification data of the significant feature must be used. For example, a shear zone dipping into the pit with a rating of 15 would dominate even if the rest of the rock mass had a rating of 50.

OVERVIEW OF THE SYSTEM

Table XV gives an overview of the MRMR system.

TABLE XI
PERCENTAGE ADJUSTMENTS FOR DEGREE OF DIP

Condition rating	Dip from vertical*				Dip towards vertical*			
	0–40°	40–60°	60–80°	80–90°	90–80°	80–60°	60–40°	40–0°
0–10	60	65	70	75	75	80	90	90
11–15	65	70	75	80	80	85	90	100
16–20	70	75	80	85	90	95	100	
21–25	75	80	85	90	95	100		
26–30	80	85	90	95	100			
31–40	85	90	95	100				

* Angles from horizontal.

TABLE XII
THE ANGLE OF CAVE AND THE FAILURE ZONE

	MRMR 1		MRMR 2		MRMR 3		MRMR 4		MRMR 5	
1. Cave Angle										
Depth, m	Unres	Res	Unres	Res	Unres	Res	Unres	Res	Unres	Res
100	70–90	85–95	60–70	75–85	50–60	65–75	40–50	55–65	30–40	45–55
500	70–80	80–90	60–70	70–80	50–60	60–70	40–50	50–60	30–40	40–50
2. Extent of Failure Zone										
Depth, m	Surf.	U/G	Surf	U/G	Surf	U/G	Surf	U/G	Surf	U/G
100	10 m	10 m	20 m	20 m	30 m	30 m	50 m	50 m	75 m	100 m
500	10 m	20 m	20 m	30 m	30 m	50 m	50 m	100 m	75 m	200 m

Unres = No lateral restraint
Res = Lateral restraint
Surf = At surface
U/G = Underground

CONCLUSIONS

The RMR/MRMR classification system has been in use since 1974, during which period it has been refined and applied as a planning tool to numerous mining operations.

It is a comprehensive and versatile system that has widespread acceptance by mining personnel.

The need for accurate sampling cannot be too highly stressed.

There is room for further improvements by the application of practical experience to the empirical tables and charts.

The DRMS system has not had the same exposure but has proved to be a useful back-up tool in difficult planning situations, and has been used successfully in mathematical modelling.

The adjustment concept is very important in that it forces the engineer to recognize the problems associated with the environment with which he is dealing.

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TABLE XIII
MINING METHOD RELATED TO MRMR

Class rating	5 0–20	4 21–40	3 41–60	2 61–80	1 80–100
Block Caving					
Undercut SI, m	1–8	8–18	18–32	32–50	+50
Cavability	Very good	Good	Fair	Poor	Very poor
Fragmentation, m	0,01–0,3	0,1–2,0	0,4–5	1,5–9	3–20
2nd lay-on blast/drill, g/t	0–50	50–150	150–400	400–700	+700
	0–20	20–60	60–150	150–250	+250
Hangups as % of tonnage	0	15	30	45	>60
Dia. of draw zone, m	6–7	8–9	10–11,5	12–13,5	15
Drawpoint span, m					
Grizzly	5–7	7–10	9–12		
Slusher	5–7	7–10	9–12		
LHD, m	9	9–13	11–15	13–18	
Brow support	Steel and concrete Reinf. concrete		Concrete	Blast protection	
Drift support	Lining, rock reinf., repair techniques		Lining, reinf.	Rock reinf.	
Width of point, m	1,5–2,4	2,4–3,5	2,4–4	4	
Direction of advance	Towards low stress		Towards high stress		
Comments	Fine fragmentation, poor ground, heavy support, repairs	Medium fragmentation, good ground, fair support	Medium coarse fragmentation, good drill hangups	Coarse fragmentation, large LHDs, drill hangups	
Sub-level Caving					
Loss of holes	Excessive	Fair	Negligible	Nil	Nil
Brow wear	Excessive	Fair	Low	Nil	Nil
Support	Heavy	Medium	Low	Localized	Nil
Dilution	Very high	High	Medium	Low	Very low
Cave SI, m	1–8	8–18	18–32	32–50	+50
Comments	Not practicable	Applicable	Suitable	Suitable	Suitable, large HW cave area
Sub-level Open Stoping					
Minimum span, m	1–5	5–20	20–30	30–80	100
Stable area, i.e. SI, m	N/A	1–8	8–16	16–35	+35

N/A = Not available

TABLE XIV
APPROXIMATE ANGLES OF PIT SLOPES

Adjusted class	1	2	3	4	5
Slope angle	75	65	55	45	35

Appendix E A NEW APPROACH TO SLOPE STABILITY PROBABILITY CLASSIFICATION (SSPC)

A new approach to rock slope stability – a probability classification (SSPC)

R. Hack · D. Price · N. Rengers

Abstract The newly developed system presented in this paper is based on a three-step approach and on the probabilistic assessment of independently different failure mechanisms in a slope. First, the scheme classifies rock mass parameters in one or more exposures and allowance is made for weathering and excavation disturbance. This gives values for the parameters of importance to the mechanical behaviour of a slope in an imaginary, unweathered and undisturbed 'reference' rock mass. The third step is the assessment of the stability of the existing slope or any new slope in the reference rock mass, taking into account both method of excavation and future weathering. From the large quantity of data obtained in the field, the Slope Stability Probability Classification (SSPC) system has been proposed, based on the probabilities of different failure mechanisms occurring. Developed during 4 years of research in Falset, Tarragona province, Spain, it has been used with good results in Austria, South Africa, New Zealand and the Dutch Antilles.

Résumé Le nouveau système de classification présenté dans ce papier est basé sur une approche en trois étapes et sur une évaluation probabiliste de différents mécanismes de rupture de pentes. D'abord, les paramètres du massif rocheux sont mesurés sur un ou plusieurs affleurements, tout en considérant les effets de l'altération et des perturbations du massif résultant de l'excavation. Cette démarche permet ensuite d'obtenir les valeurs des paramètres jouant un rôle important dans le comportement mécanique d'une pente pour un massif rocheux non altéré et non perturbé par le processus d'excavation. La troisième étape est l'évaluation de la stabilité des pentes dans le massif rocheux de référence, prenant en compte à la fois la méthode d'excavation et l'altération future. A partir du nombre important de données obtenues sur le terrain, le système de classification probabiliste de stabilité des pentes (SSPC) a été proposé, basé sur les probabilités d'occurrence de différents types de mécanismes de rupture. Développé pendant quatre ans à Falset, dans la province de Tarragone (Espagne), il a été utilisé avec de bons résultats en Autriche, Afrique du Sud, Nouvelle Zélande et aux Antilles néerlandaises.

Keywords Slope stability · Probability classification · SSPC

Mots clés Stabilité des pentes · Mécanismes de rupture de pente · Analyse probabiliste · SSPC

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Introduction

In the last decades, knowledge of the behaviour of discontinuous rock masses has developed tremendously. For constructions such as slopes, foundations and shallow tunnels, it has been recognized that discontinuities have a major influence on the mechanical properties of a rock mass. This perception has had consequences for the assessment of the engineering behaviour of a rock mass. Calculations for engineering structures in or on a rock mass must include discontinuity properties. Variations in properties can be considerable along the same discontinuity plane, however. As there may be hundreds of discontinuities in a rock mass, each with its own variable properties, these, taken together with inhomogeneities in the rock material, require that in order to describe or

calculate the mechanical behaviour of the rock mass accurately, a large amount of data is required. Laboratory and field tests may be used to obtain discontinuity properties. However, testing in large quantities is both time-consuming and troublesome.
Discontinuous 'distinct block' numerical calculations can model the discontinuities and calculate the behaviour of a rock mass in detail, provided that property data are available. Apart from the need to have powerful computers to do the large number of calculations required by the vast quantity of discontinuities, the test data needed for a detailed numerical discontinuous calculation are never available. An often-applied practice to avoid these problems is to simplify the discontinuity model and estimate or guess the properties or to use values from the literature. To what extent the result is still representative for the real situation is a question that often remains unanswered.

Existing rock mass classification systems for slopes

An altogether different approach to assess the engineering behaviour of a rock mass is rock mass classification. In a classification system, empirical relations between rock mass properties and the behaviour of the rock mass in relation to a particular engineering application are combined to give a method of designing engineering structures in or on a rock mass. Rock mass classification has been applied successfully for some years in tunnelling and underground mining (Barton 1976, 1988; Bieniawski 1989; Laubscher 1990). Some rock mass classification systems developed originally for underground excavations have been used for slopes (Barton et al. 1974; Bieniawski 1989) or have been modified for slopes (Haines and Terbrugge 1991; Romana 1985, 1991; Selby 1980, 1982). A system specially designed for slope stability has been developed by Shuk (1994).
The calculation methods and parameters in existing slope stability classification systems have been analysed and the systems used to establish the stability of existing slopes (Hack 1998). Generally, all systems include parameters for slope geometry, intact rock strength, discontinuity spacing or block size and parameters related to the shear strength along discontinuities. Some systems include the presence of water or water pressures, deformation of the rock and rock mass, susceptibility to weathering and method of excavation. Use of the existing classification systems has shown that some parameters are difficult or impossible to measure (for example, water pressures and deformation of rock masses). Most systems present the final stability as a single point value with a description (Barton et al. 1974; Barton 1976; Bieniawski 1989; Haines and Terbrugge 1991; Selby 1980, 1982). This can give results that are difficult to appreciate. Parameters influence the stability rating for a slope whose instability may be caused by a physical mechanism that is independent of those parameters. For example, intact rock strength is used to calculate the stability rating, while a slope is unstable because of sliding on a discontinuity with a thick clay infill. Hence, intact rock strength is of no importance for the stability or instability of that slope.

A new approach: slope stability probability classification (SSPC)

Expressions for uncertainty in establishing rock mass properties and for variation of properties (Nilsen 2000) and the applicability of the calculation method are also absent in existing rock mass classification systems, although they are of fundamental importance in establishing the safety of a slope design. Another important problem identified in existing systems is that generally no distinct differentiation is made between the rock mass in the exposures used for the classification and the rock mass in which a slope is to be made. Local influences such as weathering and method of excavation may be the cause of major differences. The ravelling type of failure of slopes is again generally not considered in classification systems, although rock mass classification is the only feasible option for the predicting this type of failure (Maerz 2000).
For these reasons and the generally unsatisfactory results obtained with existing rock mass classification systems, a new classification system for slope stability assessment has been developed (Hack 1998). The concept of this newly developed system is based on:

1. The introduction of the principle of a three-step classification system to describe the 'exposure', 'reference' and 'slope' rock mass.
2. The assessment of stability by determining the probability of the occurrence of different failure mechanisms instead of a single-point rating value.
3. Unambiguous and simple procedures for collection of data in the field.

Three-step classification system

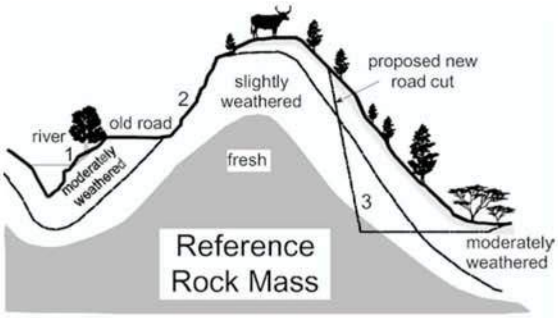
The SSPC system considers three rock masses:

1. The rock mass in the exposure – the 'exposure rock mass' (ERM).
2. The rock mass in an imaginary, unweathered and undisturbed condition prior to excavation – the 'reference rock mass' (RRM).
3. The rock mass in which the existing or new slope is to be situated – the 'slope rock mass' (SRM).

Rock mass parameters of importance are described and characterized in an exposure, resulting in the 'exposure rock mass'. Local influences on the parameters measured in the exposure such as weathering and the disturbance due to the excavation method used to create the exposure are then taken into account. This converts the parameters for the ERM to those of the theoretical fresh rock mass (Fig. 1) that exists below the zone of influence of weathering and other disturbances – the 'reference rock mass' (RRM).

This conversion is made with the aid of correction parameters: the exposure-specific parameters (Fig. 2). By this technique, parameters of material in the same geotechnical unit that show different degrees of weathering and different degrees of excavation disturbance are brought back to parameters reflecting their original basic geotechnical properties.

The actual stability assessment is made in the 'slope rock mass' (SRM). This is derived from the 'reference rock mass' (RRM) by adjustment of the parameters of the RRM with the slope-specific parameters. Slope-specific parameters are correction parameters for the influence of future weathering within the engineering lifetime of the slope and the influence of the method of excavation to be used. The ERM and SRM are the same if an existing slope is examined and future weathering is not considered.



1: natural exposure made by scouring of river, moderately weathered; 2: old road, made by excavator, slightly weathered; 3: new to develop road cut, made by blasting, moderately weathered to fresh.

Fig. 1
Sketch of exposures in rock masses with various degrees of weathering and different types of excavation, and indicating the concept of the 'reference rock mass'

Research area

The research for the development of the classification system was done in the area around Falset in northeast Spain, in the province of Tarragona. The area is particularly suitable for this type of research as there is a large variation in the geology, lithology and tectonic environment, giving different geological environments for the development of the classification system. Rocks in the Falset area vary from Tertiary conglomerates to Carboniferous slates and include rocks containing gypsum, shales, granodiorite, limestone and sandstones. The topography is mountainous and the vegetation limited, such that large areas of rock are exposed. In addition, numerous old roads (built some 40 to 60 years ago) exist and several roads have been built in recent years creating large numbers of road cuts, excavated using different techniques. Many old and new slopes had been designed or excavated poorly, resulting in a number of slope instabilities over the years. The height of the slopes in the road cuts is typically between 5 and 25 m with a maximum of about 45 m. This has allowed a comparison of both slope stand-up times and excavation methods as well as an assessment of the influence of weathering and the method of excavation.

The climate in the Falset area is Mediterranean, characterized by dry and hot summers (temperature ranges from ≈15 to 35 °C) and moderate winters (10 to 15 °C). Part of the area is mountainous, ranging to about 1,200 m above sea level. Rivers and streams in the area are mostly dry from March to October, but it can rain for long periods during the winter and even into April, although this is not

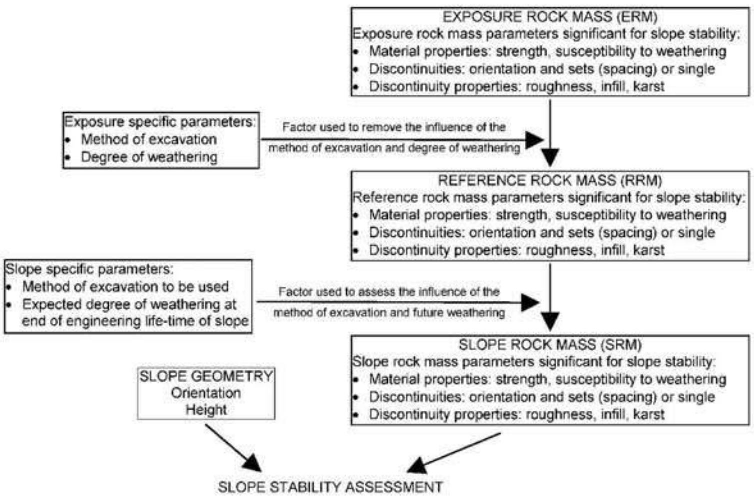


Fig. 2
Flow diagram of the three-step concept of the SSPC system

typical. Sometimes the rain is torrential. Occasionally, temperatures below zero occur but snowfall in the area is rare.

Description of field conditions

Visual assessment of slope stability

The research was directed towards designing a slope stability classification system incorporating all possible mechanisms and modes of slope failure. To develop a new slope stability classification system, the stability of the slopes was visually classified in the field as stable or unstable, with a further subdivision into unstable with small problems and unstable with large problems. In principle, 'large' implies that the unstable rock mass is in the order of tonnes in weight, while 'small' implies that the unstable rock mass would weigh in the order of kilograms.

Visually estimating the degree of stability of a slope is to a certain degree subjective. This is a problem for all classification systems. For the SSPC system, estimates have been made over a period of 4 years using at least 60 observers from staff and students of ITC and Delft University of Technology, working on 184 slopes. It is therefore reasonable to assume no observer bias.

Geotechnical units in a rock mass

Theoretically, a proper assessment to determine the behaviour of a rock mass should include all properties in a rock mass and all spatial variations of the properties. This would be unrealistic and is not possible without the destruction of the rock mass, hence standard procedure is to divide a rock mass into homogeneous geotechnical units. In practice, such homogeneity is seldom found and material and discontinuity properties vary within a selected range of values within the units. The smaller the allowed variability of properties in a geotechnical unit, the more accurate the assessment can be. Limiting the variability of the properties of the geotechnical units involves collecting more data, however, and is thus more costly. Higher accuracy based on more data must therefore be balanced against the economic, social and environmental value of the engineering structure to be built and the possible risks for the engineering structure, environment or human life. For a road cut along a major highway, the variations allowed within a geotechnical unit will be smaller than those for a geotechnical unit in a road cut along a local road. No standard rules are available for the division of the rock mass into geotechnical units; this depends on experience and 'engineering judgment'.

Failure mechanisms

Slope failure mechanisms (such as shear displacement) and the resulting different failure modes (plane sliding, wedge failure, partial toppling and buckling) are discontinuity-related and depend on the orientations of the slope and discontinuity. However, other mechanisms not related to the orientations of the slope and the discontinuities can also cause failure of a slope, e.g. the breaking of intact rock under the influence of the stresses in the slope and the removal of slope surface material due to surface (rain)-water and seepage of water out of the rock mass (ravelling).

Traditional rock slope stability analyses are based on recognition of the failure mode in the field followed by a (back)calculation. Although the failure modes causing slope instability are theoretically well defined, it is often difficult to recognize the operating failure mode in the field. In many unstable slopes, multiple modes are at work at the same moment or successively. Not all of these may be visible or easily recognizable. Moreover, not only do the proper failure modes have to be identified, but for slopes with multiple modes at work, the contribution of each mode to the overall (in)stability should also be quantified. In cases where different modes of failure operate successively, the moment the slope is examined may determine the failure mode recognized. For these reasons, in this research both stable and unstable slopes were analysed without regard to the cause of instability, to avoid the problem of identifying the exact failure modes in the field.

Determination of rock mass parameters

The rock mass properties necessary for the SSPC system (intact rock strength and discontinuity spacing and condition) are determined by relatively simple means in the field.

Intact rock strength

Intact rock strength is established in the field by 'simple means' following the table in Fig. 15. The method has been tested extensively and the results compared with strengths obtained by laboratory unconfined compressive strength tests. The strengths determined by 'simple means' by about 50 different people showed that the results of the 'simple means' field tests are at least comparable to the quality of results obtained by laboratory UCS tests (Hack 1998). Although the 'simple means' tests may be thought to be subjective, only a short training on rock pieces with known intact rock strengths is enough to reduce subjectivity to an acceptable level. Large numbers of 'simple means' tests can be done in a short time span and are not dependent on obtaining a sample large enough for laboratory testing. The large number of tests also gives a better indication of the variation of the intact rock strength throughout the rock mass than can be obtained from a limited number of UCS tests values (Hack 1998).

Orientation, spacing, and condition of discontinuities

The orientation of discontinuities in combination with the shear strength along discontinuities determines the possibility of movement along discontinuities and thus has a major influence on the mechanical behaviour of a rock mass. It should first be established whether discontinuities belong to a 'set' or should be treated as a 'single' feature. Determining the parameters for a set of discontinuities requires a form of averaging of the parameters of individual discontinuities.

The average orientation of a discontinuity set can be found mathematically or by stereo-projection methods (Terzaghi 1965; Taylor 1980; Hoek and Bray 1981; Davis 1986). The characteristic properties of each discontinuity set are the average of the properties of each measured discontinuity belonging to that set. A disadvantage of these methods is that it may be difficult to distinguish between the different discontinuity sets. Furthermore, an important discontinuity set may be missed or underrated in importance because the discontinuity spacing is large. This and other errors that may affect the results of stereographic projection methods to determine discontinuity sets and orientations are discussed in extenso by Terzaghi (1965).

Alternatively, a studied assessment can be undertaken, in which the discontinuities that are representative for a set are visually selected. The properties of the selected discontinuities are then measured in detail in predetermined locations. In the opinion of the authors based on both experience during former work and this research, this method gives a result equal to or better than the results of large numbers of measurements of discontinuities for a statistical analysis. A large number of measurements are usually done on a part of the exposure that is (easily) accessible, whether representative of the rock mass or not. The same observations have been made by other researchers (Gabrielsen 1990). Moreover, the variation of discontinuity properties in one discontinuity set is often so large that a high degree of accuracy for an individual measurement is not very important (ISRM 1978, 1981) and the variation of properties is covered by the probability approach of the SSPC system.

Determining properties representing shear strength of a discontinuity

The shear strength of a discontinuity is determined by the sliding criterion that converts a visual and tactile (roughness established by touch) characterization of a discontinuity into an apparent friction angle along the discontinuity plane (Hack and Price 1995). Figure 15 lists the different descriptive terms. The method of characterization of discontinuities in the SSPC system is partly based on existing literature (Rengers 1971; ISRM 1978; Laubscher 1990). The large-scale roughness is determined following the examples in Fig. 3. The small-scale roughness factors are a combination of visible roughness on an area of about 20×20 cm^2 and tactile roughness. Visible small-scale roughness (e.g. 'stepped', 'undulating' and 'planar') is established following the examples in Fig. 4. Tactile (material) roughness is established by touch (e.g. rough, smooth and polished). The relation between the different roughness parameters is illustrated in Fig. 5.

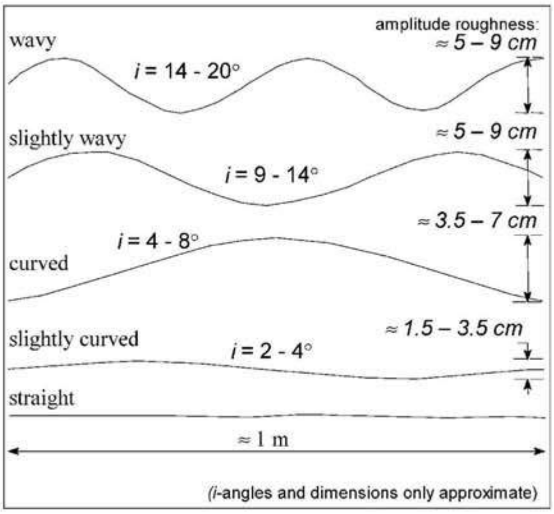


Fig. 3
Large-scale roughness profiles

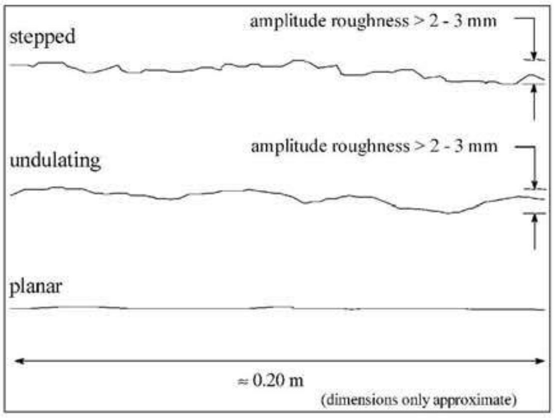


Fig. 4
Small-scale roughness profiles

Infill material in discontinuities and the presence of karst along discontinuities are characterized following the table in Fig. 15. This figure also shows how the characteristics of the discontinuity are translated into values for four factors: large-scale *(Rl)* and small-scale *(Rs)* roughness, infill material *(Im)* and karst *(Ka)*). The condition factor for a discontinuity (*TC*) is calculated by a simple multiplication of these four factors:

$$TC = Rl * Rs * Im * Ka \qquad (1)$$

Non-fitting discontinuities

The contribution of the roughness to the shear strength reduces if discontinuities are non-fitting (Rengers 1971).

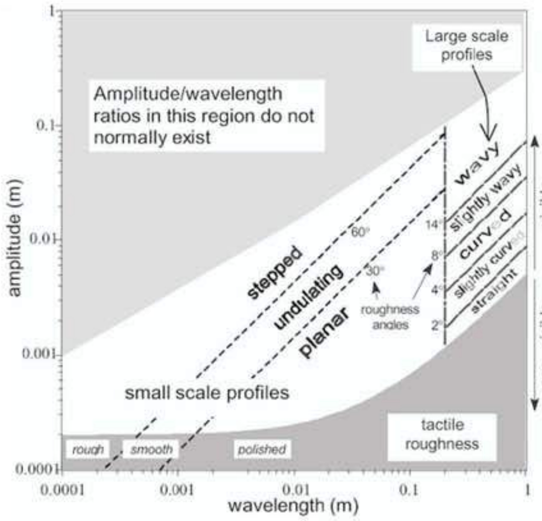


Fig. 5
Interpretation of roughness as a function of wavelength and amplitude. For small amplitudes and wavelengths, the roughness changes to a more sinusoidal form. Lustre is not included in the boundary non-visible to visible roughness. The boundaries in the graph are dashed, as these are not exact

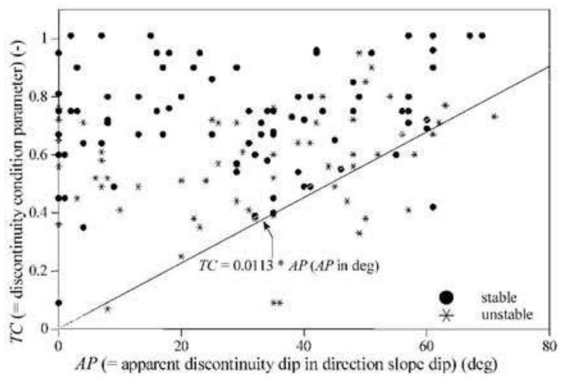


Fig. 6
Discontinuity condition TC vs. AP for day-lighting discontinuities in stable and unstable slopes

For large-scale roughness, only an estimate can be made as to how much the contribution of the large-scale roughness to shear strength is reduced due to non-fitting. For visible small-scale roughness, a similar procedure may be followed, but tilt or shear box tests can also be undertaken and the results converted into a roughness characterization (Hack and Price 1995). If a discontinuity is completely non-fitting, the shear strength depends only on the material roughness, e.g. rough, smooth or polished. Such a discontinuity would be characterized as 'planar' for small-scale visible roughness and 'straight' for large-scale roughness.

Non-persistent discontinuities

A non-persistent discontinuity (e.g. a discontinuity ending in intact rock) is treated as a discontinuity with a small-scale roughness of 'rough stepped'. Breaking through asperities has to take place before displacement along a rough stepped discontinuity can occur. This approach is similar to that used in the Q-system (Barton 1976).

Stability analysis

The stability is determined by two analyses: the first is related to the orientation of the discontinuities and the slope ('orientation-dependent stability') and the second to the strength of the rock mass in which the slope occurs, independent of the orientation of both the discontinuities and the slope ('orientation-independent stability').

Orientation-dependent stability

Failures in a rock slope often depend on the orientation of the slope and the discontinuities in the rock mass. The main parameter governing this type of failure is the shear strength of the discontinuity. Two criteria were developed in the SSPC system to predict the orientation-dependent stability of a slope: the sliding and toppling criteria.

Sliding criterion

A relation was found between the condition value of a discontinuity TC (Eq. 1) and the apparent angle of the dip of the discontinuity plane in the direction of the slope dip (AP):

$$
\begin{aligned}
&AP = \arctan\left(\cos\delta * \tan dip_{discontinuity}\right)\\
&\quad \text{if } AP > 0^\circ \rightarrow AP = \text{apparent discontinuity dip}\\
&\qquad \text{in the direction of the slope dip}\\
&\quad \text{if } AP < 0^\circ \rightarrow |AP| = \text{apparent discontinuity dip}\\
&\qquad \text{in the direction opposite the slope dip}\\
&\delta = dipdirection_{slope} - dipdirection_{discontinuity}
\end{aligned}
\tag{2}
$$

Below the dashed line in Fig. 6 only two combinations of values for TC and AP exist for day-lighting discontinuities in stable slopes (the two points below the line are likely due to a measuring error). The 'sliding criterion' (Eq. 3) is therefore considered as the boundary condition for sliding in slopes and sliding occurs if:

$$TC < 0.0113 * AP \tag{3}$$

The sliding criterion is confirmed by field and laboratory test values for discontinuity friction and by friction values for discontinuities discussed in the literature (Hack and Price 1995).

Toppling criterion

Analogous to the sliding criterion, the 'toppling criterion' considers the interlayer slip necessary for toppling as defined by Goodman (1989). The SSPC toppling criterion is:

$$TC < 0.0087 * \left(-90^\circ - AP + dip_{discontinuity}\right) \quad (4)$$

Additional conditions

Additional conditions concern the minimum difference between the slope and discontinuity planes for the sliding criterion: $dip_{slope}>AP+5^\circ$. This is necessary because a daylighting discontinuity with an apparent dip (in the direction of the slope dip) similar to that of the slope dip will form the slope face and will not cause sliding failure. A second condition is that the discontinuity plane should not be near vertical as a vertical plane cannot be a sliding plane or a cause of toppling. Therefore, for sliding to occur AP must be $<85^\circ$ and for toppling to occur it must be $>-85^\circ$. The value of 5° is based on field observations which indicated that measuring accuracy is normally around 5°. Figures 9 and 10 give the probabilities for sliding and toppling respectively as a function of the apparent discontinuity dip in relation to the condition of the discontinuity. Probability calculations are given below.

Methodology to optimize sliding and toppling criteria

The sliding and toppling criteria can be demonstrated on a graph but as this introduces some subjectivity, the criteria have also been established mathematically. To determine the boundary line for the sliding criterion, 300 sets of data points (AP, TC) were generated randomly out of the data set for discontinuities in stable slopes in the research area, with the standard error distribution in AP and TC for each original data point (see probability analyses below). A number of data points (X) with the lowest ratios of TC/AP were determined from each set of data points. Those with the lowest TC/AP were used because the boundary line should be the lower boundary of the data set. The slope and intercept of a linear regression of these X data points were computed for each of the 300 sets of data points, resulting in 300 regression lines for which the mean and standard error were calculated. The number of data points (X) used for the regression varied from 2 to 30. Figure 7 illustrates the procedure for X=2 and Fig. 8 the mean and standard error of the intercept and the slope versus X. If six points are used for regression, the values for the mean intercept and mean slope are robust (changing only slightly if more points are used) and the standard errors approximately constant. As would be expected, the value for the mean slope coincides with the visually determined boundary. The same procedure was followed for the toppling criterion.

Orientation-independent stability

A large number of the slopes were found not to be unstable using the sliding and toppling criteria, although they were assessed visually in the field as unstable. For these slopes, a mathematical model could be formulated to predict the orientation-independent stability. Most of the failures in these slopes were approximately linear, although not following one and the same existing discontinuity plane. Often, fracturing of intact rock over small distances (relative to the size of the slope) results in linear failure planes developing partly through intact rock and partly following existing discontinuity planes. This effect was more prominent in rock masses in which the block size was smaller. Intact rock strength, block size and shear strength along discontinuities thus have an influence on the development of failure planes not related to a single existing discontinuity.

The orientation-independent stability of such a slope was modelled by a linear shear plane model following the Mohr-Coulomb failure criterion (Hack 1998). The friction

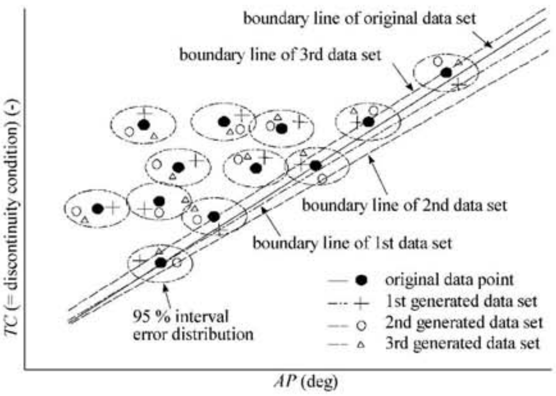


Fig. 7
Sketch showing the procedure to calculate the boundary line for the 'sliding criterion' for X=2 (e.g. boundary line based on two data points)

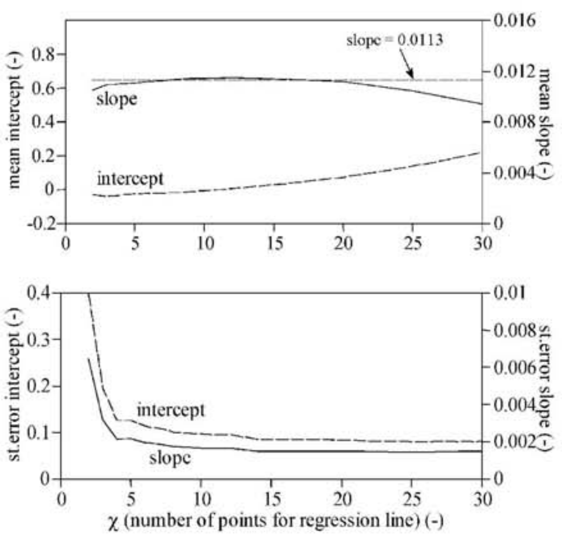


Fig. 8
Mean and standard error of intercept and slope of boundary lines vs. χ, for 'sliding criterion'

and cohesion parameters in the Mohr-Coulomb failure criterion are the apparent friction and cohesion for the rock mass. The rock mass friction and cohesion are dependent on intact rock strength, block size (e.g. discontinuity spacing) and shear strength (e.g. the condition of discontinuities) along all discontinuities in the rock mass.

Block size and condition of discontinuities

Several options exist to incorporate the intact rock strength, the block size and the condition of all discontinuities in the shear plane model. Extensive analyses (Hack 1998) gave the best results for the block size (*SPA*) if based on the factors of Taylor (1980); see graph in Fig. 16:

$$\begin{aligned}
&\text{for three discontinuity sets :}\\
&SPA = factor_{maximum} * factor_{intermediate} * factor_{minimum}\\
&\text{for two discontinuity sets :}\\
&SPA = factor_{maximum} * factor_{minimum}\\
&\text{for one discontinuity : } SPA = factor\\
&SPA = \text{spacing parameter}\\
&factor_x = \text{determined by graph included in figure 16}
\end{aligned} \tag{5}$$

The best formulation for the condition of discontinuities in a rock mass (*CD*) was found to be based on the mean of the conditions of three discontinuity sets weighted against the spacing of the sets:

$$CD = \frac{\frac{TC_1}{DS_1} + \frac{TC_2}{DS_2} + \frac{TC_3}{DS_3}}{\frac{1}{DS_1} + \frac{1}{DS_2} + \frac{1}{DS_3}} \tag{6}$$

$TC_{1,2,3}$ are the condition, and $DS_{1,2,3}$ are the spacings of discontinuity sets 1, 2, 3

SSPC rock mass friction and cohesion

Optimizing the Mohr-Coulomb failure criterion with the intact rock strength (*IRS*), spacing (*SPA*) and condition of discontinuities (*CD*) gives the following:

$$\begin{aligned}
&\varphi'_{mass} = IRS * 0.2417 + SPA * 52.12 + CD * 5.779\\
&coh'_{mass} = IRS * 94.27 + SPA * 28629 + CD * 3593\\
&if\ intact\ rock\ strength > 132\ MPa \rightarrow IRS = 132\\
&\varphi'_{mass} = angle\ of\ internal\ friction\ of\\
&\qquad the\ rock\ mass\ (in\ degrees)\\
&coh'_{mass} = rock\ mass\ cohesion\ (in\ Pa)
\end{aligned} \tag{7}$$

The intact rock strength in Eq. (7) is maximized. Above a value of about 132 MPa, it was found that the stability of the slopes did not further increase with increasing intact rock strength. This was valid for the slopes in the research area with heights ranging up to 45 m. A higher value for the intact rock strength maximum may be necessary for significantly higher slopes with higher stresses. For both spacing (*SPA*) and condition (*CD*), the combination of discontinuity sets that results in the minimum rock mass friction value was always taken. Even if the rock mass contains three or more discontinuity sets, spacing (*SPA*) and condition (*CD*) factors calculated from only one or two discontinuity sets may give a lower result.

Linear shear plane model and maximum slope height

The model of a linear shear plane following the Mohr-Coulomb failure criterion implies that the stability of a slope is independent of the height of the slope if the slope has a dip angle less than the friction angle of the rock mass. If the dip angle is higher than the friction angle, however, the maximum slope height is determined by the stresses in the slope. For a rock mass unit weight of 25 kN/m^3:

$$\begin{aligned}
&If\ the\ dip_{slope} \le \varphi'_{mass}:\\
&the\ maximum\ slope\ height\ (H_{\max})\ is\ infinite\\
&else\\
&H_{\max} = 1.6 * 10^{-4} * coh'_{mass} * \\
&\frac{sin\left(dip_{slope}\right) * cos\left(\varphi'_{mass}\right)}{1 - cos\left(dip_{slope} - \varphi'_{mass}\right)}
\end{aligned} \tag{8}$$

Stability

In terms of probabilities, the orientation independent stability is given in Fig. 11. The axes are horizontally normalized on the ratios of rock mass friction to slope dip and vertically on the maximum possible height (H_{max}) as a ratio of the true height (H_{slope}).

Methodology to optimise orientation independent failure criterion

It is assumed that φ_{mass} and coh_{mass} are dependent on the rock mass parameters measured in the field, e.g. intact rock strength (*IRS*), spacing of discontinuities (*SPA*) and condition of discontinuities (*CD*). In addition to linear relationships between φ_{mass} and coh_{mass}, and *IRS*, *SPA* and *CD*, the following have also been investigated:

$$\begin{aligned}
&\varphi_{mass} \text{ or } coh_{mass} = IRS * e^{\frac{-w1}{SPA*CD}}\\
&\varphi_{mass} \text{ or } coh_{mass} = IRS * SPA^{w1} * CD^{w2}
\end{aligned} \tag{9}$$

The research has found that both φ_{mass} and coh_{mass} can be reasonably represented by a linear combination of *IRS*, *SPA*, and *CD*. The influence of the intact rock strength on slope stability is bounded by a maximum (cut-off) value. Linear relationships for φ_{mass} and coh_{mass} with a cut-off value for the intact rock strength (*IRS*) result in the following:

$$
\begin{aligned}
&coh_{mass} = w0 * IRS + w1 * SPA + w2 * CD\\
&\varphi_{mass} = w3 * IRS + w4 * SPA + w5 * CD\\
&\text{if } IRS \leq \text{cut} - \text{off value} \rightarrow IRS = \text{intact rock strength}\\
&\text{(as measured in the field)}\\
&\text{if } IRS > \text{cut} - \text{off value} \rightarrow IRS = \text{cut} - \text{off value}\\
&\text{weight factors} : w0, w1, .., w5 \geq 0
\end{aligned} \tag{10}
$$

φ_{mass}, the friction of the rock mass, has a value within a range from 0 to 90° (0 to $\pi/2$). In order to optimize the shear plane model, φ_{mass} has to be normalized so that the value is never outside this range. The maximum value for φ_{mass} is obtained for an intact rock strength (*IRS*) equal to the cut-off value, *SPA* equal to its maximum value of 1.00 and *CD* to its maximum value of 1.0165. Hence, the maximum for φ_{mass} is expressed by:

$$
\begin{aligned}
&\varphi_{mass}(maximum)\\
&= w3 * cut{-}off\ value + w4 * 1.00 + w5 * 1.0165
\end{aligned} \tag{11}
$$

φ_{mass} in Eq. (10) must thus be divided by φ_{mass} *(maximum)* and multiplied by $\pi/2$.
Large differences in the order of magnitude of parameter values may have an influence on the optimum values found in the non-linear optimization. The intact rock strength (*IRS*) in Eq. (10) has therefore been divided by 100 to reduce the difference with SPA and CD. Combining with Eqs. (8) and (10) with the normalization of φ_{mass} and the division of *IRS* by 100 leads to a set of equations describing the shear plane model:

$$
\begin{aligned}
&dip_{slope} \geq \varphi_{mass} \rightarrow\\
&H_{\max} = 4 * \frac{coh_{mass}}{UW} * \frac{\sin(dip_{slope}) * \cos(\varphi_{mass})}{1 - \cos(dip_{slope} - \varphi_{mass})}\\
&dip_{slope} < \varphi_{mass} \rightarrow H_{\max} = unlimited\\
&coh_{mass} = a0 * \frac{IRS}{100} + a1 * SPA + a2 * CD\\
&\varphi_{mass} = \left(\frac{a3 * \frac{IRS}{100} + a4 * SPA + a5 * CD}{a3 * a6 + a4 + a5 * 1.0165}\right) * \frac{\pi}{2}\\
&if\ \frac{IRS}{100} \leq a6 \rightarrow IRS = intact\ rock\ strength\ (in\ MPa)\\
&if\ \frac{IRS}{100} > a6 \rightarrow IRS = a6 * 100\\
&a0\ through\ a6 = weight\ factors\ dip_{slope} = dip\ of\ slope\\
&H_{\max} = maximum\ possible\ slope\ height\\
&UW = Unit\ Weight\ of\ the\ rock\ mass
\end{aligned} \tag{12}
$$

SPA is calculated following Eq. (5) and *CD* following Eq. (6).
Measured intact rock unit weights ranged between 25.5 and 27.0 kN/m^3. A small proportion of open discontinuities in the rock mass indicates that the unit weight of the rock mass is approximately the same as that of the material. The value of the unit weight of the rock mass was taken to be the same for all rock masses in the research area – it was considered that the karstic rock units do not have a rock mass unit weight considerably less than the intact rock unit weight.
In Eq. (12), the values of the factors *a0* through *a6* are unknown. Equation (12) is therefore optimized (following the set of optimization rules in Eq. 13) over the slopes that are not unstable due to orientation-dependent stability.

$$
\begin{aligned}
&\text{For each slope } j : \text{visually estimated stability} =\\
&\text{stable} \begin{cases} \frac{\varphi_{mass}}{dip_{slope}} \geq 1(stable) \rightarrow er = 1\\ \frac{\varphi_{mass}}{dip_{slope}} < 1 \begin{cases} \frac{H_{\max}}{H_{slope}} \geq 1(stable) \rightarrow er = 1\\ \frac{H_{\max}}{H_{slope}} < 1(unstable) \rightarrow er = \frac{H_{slope}}{H_{\max}} \end{cases} \end{cases}\\
&\text{visually estimated stability} =\\
&\text{unstable} \times \begin{cases} \frac{\varphi_{mass}}{dip_{slope}} \geq 1(stable) \rightarrow er = \frac{\varphi_{mass}}{dip_{slope}}\\ \frac{\varphi_{mass}}{dip_{slope}} < 1 \begin{cases} \frac{H_{max}}{H_{slope}} \leq 1(unstable) \rightarrow er = 1\\ \frac{H_{max}}{H_{slope}} > 1(stable) \rightarrow er = \frac{H_{max}}{H_{slope}} \end{cases} \end{cases}\\
&ER = \sum_j er_j
\end{aligned} \tag{13}
$$

In Eq. (13), H_{slope} and dip_{slope} are the real height and dip of the existing slope (*j*). φ_{mass} is the rock mass friction of the rock mass in which this slope (*j*) is made (defined in Eq. 12) . H_{max} is the theoretical maximum height of a slope which, with the same dip as that of the existing slope (dip_{slope}), can be sustained by the rock mass in which the existing slope is made. H_{max} is defined in Eq. (12). ER in Eq. (13) is the value over which optimization is undertaken, i.e. it is the value which will be minimized during the optimization procedure. *ER* equals the summation of er_j over all slopes used in the optimization.
The procedure of the optimization is that for each slope (*j*) the φ_{mass} and H_{max} are calculated following Eq. (12) with (initially randomly) chosen values for the unknowns *a0* to *a6*. If for slope (*j*) the φ_{mass} is larger than the dip of the existing slope (dip_{slope}), then slope (*j*) should be stable following the shear plane model. If this is in accordance with the visually estimated stability of the existing slope (*j*) the unknowns *a0* to *a6* are correctly chosen. Hence, er_j is set to the value 1. If the existing slope (*j*) is not stable then the unknowns *a0* to *a6* are not correct and er_j is set to a value larger than 1, which reflects how much the calculated φ_{mass} differs from values that would result in stability at equilibrium (i.e., $er_j = \varphi_{mass}/dip_{slope}$).
The procedure is more complex if $\varphi_{mass} < dip_{slope}$. The theoretical possible height ($=H_{max}$) should then be compared to the real height of the existing slope (*j*) ($=H_{slope}$). If it is more than the height of the existing slope (*j*) then slope (*j*) should be stable following the shear plane model with the chosen set of values for the unknowns *a0* to *a6*. If this is in accordance with the visually estimated stability of slope (*j*) then the calculation is correct and er_j is set to the value 1. If the existing slope (*j*) is visually assessed as unstable then the unknowns *a0* to *a6* are not correct and

er_j is set to a value larger than 1 which reflects how much the calculated H_{max} differs from values that would result in stability at equilibrium following the shear plane model calculated for slope (*j*) (i.e. $er_j=H_{max}/H_{slope}$). If slope (*j*) is calculated to be unstable ($H_{max}<H_{slope}$) and slope(*j*) is also visually assessed as unstable, the unknowns are correctly estimated and $er_j=1$. If the slope is visually assessed as stable, however, the unknowns are incorrect and the value of er_j must be set to a value reflecting the degree of mis-calculation ($er_j=H_{slope}/H_{max}$).
The above is undertaken for all slopes with one set of values for the unknowns *a0* to *a6*. The er_j of all the slopes are then added to give an *ER*. A second set of unknowns *a0* to *a6* is determined following the Levenberg-Marquardt (1963) optimization routine and the *ER* calculated following the same procedure. The optimization routine compares the *ER* values for the different sets of unknowns and based on this determines a new set of values for the unknowns *a0* to *a6*, such that the *ER* calculated based on these new values is likely to be lower than those previously used. The optimization routine will continue until no further reduction of *ER* is obtained. The values for the unknowns *a0* to *a6* resulting in the lowest value for *ER* are assumed to be the most appropriate values for which the shear plane model formulated in Eq. (12) best fits the data. *ER* would equal the total number of slopes used in the optimization if the shear plane model is the completely correct model for orientation-independent stability, if the data set is ideal (no errors in any parameter of any slope), and if the factors *a0* through *a6* are at optimum values. The stability calculated with the shear plane model would then be the same as the visually estimated slope stability in the field for all slopes. Obviously, this is unlikely because the shear plane model is not a completely correct model and the data set is not likely to be ideal. There are thus always a certain percentage of the slopes for which the slope stability following the shear plane model is not equal to the visually estimated stability in the field. For this reason, the value of *ER* is always larger than the total number of slopes used in the optimization. The goal of the optimization is to minimize *ER*. The values for *a0* through *a6* in Eq. (12) belonging to the minimum value for *ER* are then taken to be the values that best fit the data set.
During the optimization process, the ratios of H_{slope}/H_{max} (for slopes visually estimated to be stable) and H_{max}/H_{slope} (for slopes visually estimated to be unstable) are limited to a maximum of 2. The ratio of $\varphi_{mass}/dip_{slope}$ (for visually estimated unstable slopes) is similarly limited to 2. These limitations are necessary to avoid too strong an influence of possible outliers. In particular, H_{max} becomes (extremely) large and influences the optimization very significantly for an outlier with φ_{mass} smaller than, but almost equal to, the slope dip.
The maximum possible height of a rock slope (H_{max}) is infinite if the slope dip angle is less than the rock mass friction (φ_{mass}). As a consequence of this and of the use of a cut-off value for the intact rock strength, the function in Eq. (12) is not continuous in the first derivative. Because of the likely errors in the data (visually estimated stability, dip, height, intact rock strength, etc.), the function also contains multiple minima. Optimization of a function that is not continuous in the first derivative and that contains multiple minima is difficult and it is often doubtful whether the absolute minimum can be found. The function was therefore examined graphically to find ranges for the factors in which the function is likely to minimize (decreasing *ER*). Then the Levenberg-Marquardt optimization routine (Marquardt 1963) was implemented with starting values for the factors within the ranges graphically determined. A lowest minimum was found for which multiple optimizations with different starting values resulted in approximately the same values for the six factors. A graphic examination of the function with these values showed that these values were probably the best possible and represented the absolute minimum of the function. These values are used as starting values in the optimizations for the probability analyses. Figure 12 shows the results of the optimization with the data from the research area. Note that, as would be expected, the slopes characterized visually as 'unstable with small problems' plot near the line of equilibrium (dashed line). However, this information on the scale of the instability is not used in the optimization and therefore confirms the optimization results independently.

Influence of water

Water pressures in discontinuities are traditionally believed to be of major importance in rock slope stability. However, a more thorough examination indicates that, in general, this must be regarded as doubtful for many rock masses. Most rock masses at or near the surface contain many discontinuities and these will generally allow water to freely flow out of the rock mass in a slope cut, while the cover of topsoil frequently present above a slope will reduce the rate of water inflow as it is generally less permeable than the rock mass. This inhibits any build-up of water pressures.
Another reason for smaller or non-existent water pressures is that near the slope face, stresses will be smaller than those occurring deeper in the rock mass. Smaller stresses will cause the discontinuities to open and hence reduce water pressures in discontinuities with water flowing in the direction of the slope cut. The only situation in which water pressures will be a major influence is if a new slope intersects a groundwater table. However, it would be anticipated that good engineering practice would ensure appropriate drainage measures were taken to lower the groundwater table behind such a slope cut.
In the more established rock mass classification systems, the influence of water varies widely ranging between 3 and 15%, but lower values are more common in recently developed rock mass classification systems for slope stability (Hack 1998). For example, the maximum influence of the presence of water in the RMR system is 15% (Bieniawski 1989), in the SMR system 13% (Romana 1985), 6% in the system developed by Selby (1980) and only 3% in the

system developed by Haines and Terbrugge (1991). The SSPC system does not explicitly incorporate a factor for the presence of water pressures. However, the presence of water is incorporated in the factors for the infill material in discontinuities for materials that lose strength if water is present.

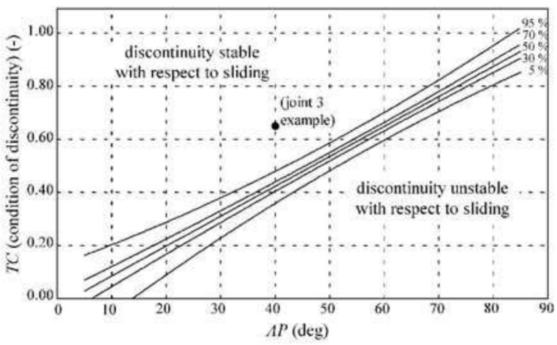


Fig. 9
Sliding criterion

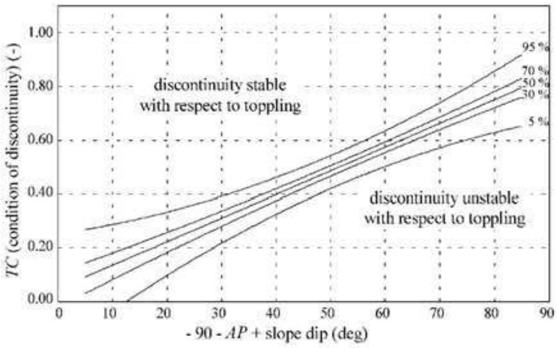


Fig. 10
Toppling criterion

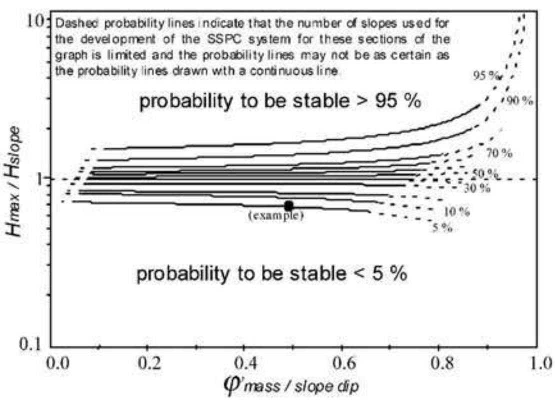


Fig. 11
Probability of orientation-independent stability

Local influences: weathering and method of excavation

The three-step approach allows for correction of local influences such as weathering and the damage due to the method of excavation. The 'exposure' rock mass is first divided into geotechnical units. For each geotechnical unit the rock mass parameters are determined and converted into parameters for the 'reference' rock mass by correction for local weathering in the exposure characterized (Hack and Price 1997) and for damage due to the method of excavation used to create the exposure. These correction factors are listed in Fig. 15. The weathering characterization follows BS 5930 (1981) although for most rock masses these can easily be converted to comply with the 1999 revised standard. The parameters that characterize the 'slope' rock mass are obtained by correction of those for the 'reference' rock mass to allow for damage due to the method of excavation to be used for the new slope and to take into account present and future weathering (see Fig. 17).

Future weathering is predicted by examining the same geotechnical unit in exposures which have been in existence for a known period of time. It should be noted that weathering may well depend on very local influences, such as orientation of the exposure, position in the landscape (wind), use of fertilizers by farmers which may influence the mineral stability (via the groundwater), etc. Although this may sometimes be difficult, experience suggests that enough information can be found to estimate the most likely degree of weathering of the geotechnical unit at the end of the engineering lifetime.

Probability analyses

A probabilistic approach using the Monte Carlo method (Hammersley and Hanscombe 1964) was applied in the

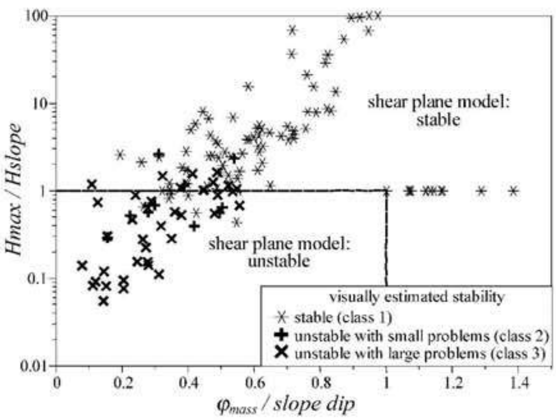


Fig. 12
Results of optimization for orientation-independent stability

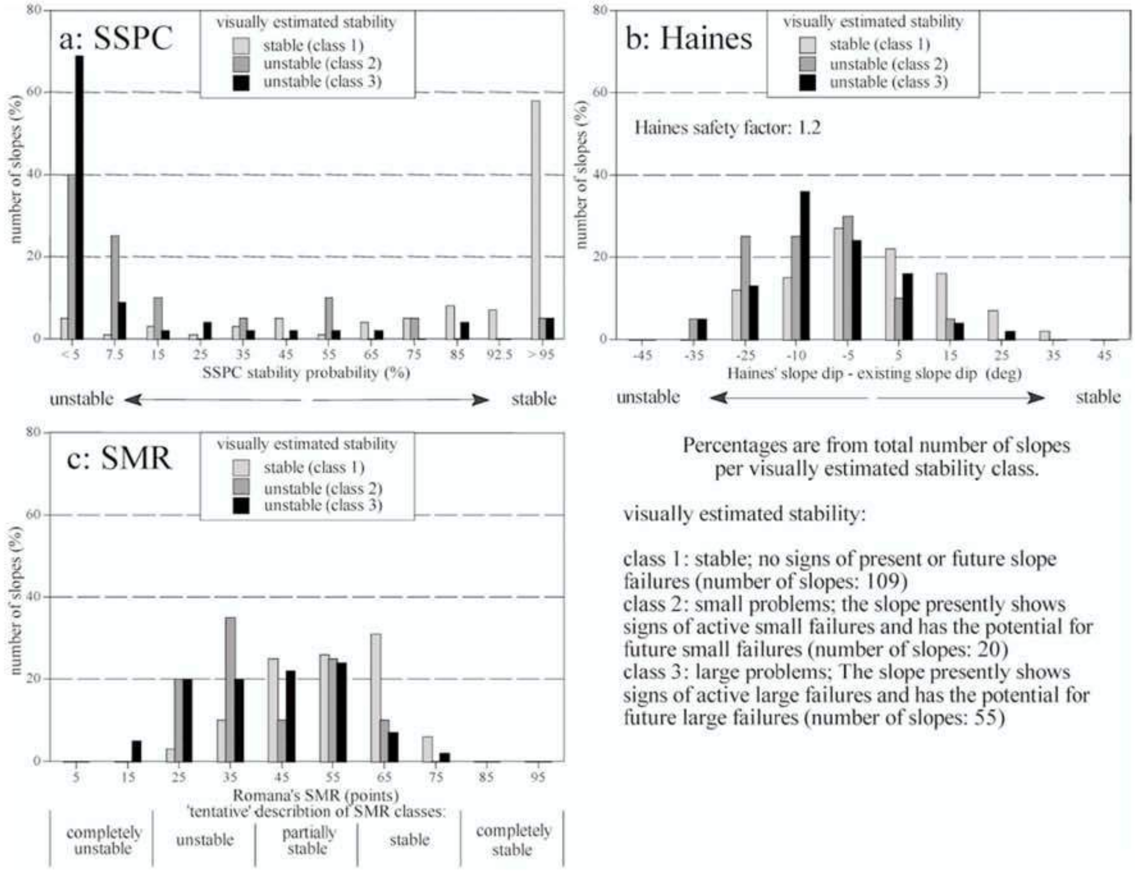


Fig. 13
Comparison of slope stability measurements by different classification systems

analysis to quantify the reliability of the functions found for slope stability in the SSPC system. The results of the Monte Carlo simulations were used to indicate the probability lines in Figs. 9, 10 and 11. This methodology also allows an evaluation of the sensitivity of the result for input errors. A measured rock mass parameter has a distribution related to a combination of: (1) the variation of a parameter in a rock mass, (2) the limitations of the variation of a rock mass parameter imposed by the subdivision of the geotechnical units and (3) the error made in measuring a rock mass parameter in a geotechnical unit. The latter can be determined by repeating a measurement many times at exactly the same location such that a standard error is obtained. Clearly, only one single location should be used or the distribution of a parameter in the geotechnical unit would contribute to the standard error.

During the research, repeated measurements of the same parameter in the same geotechnical unit were made by different students and staff members. The variation resulting from these measurements is assumed to be typical

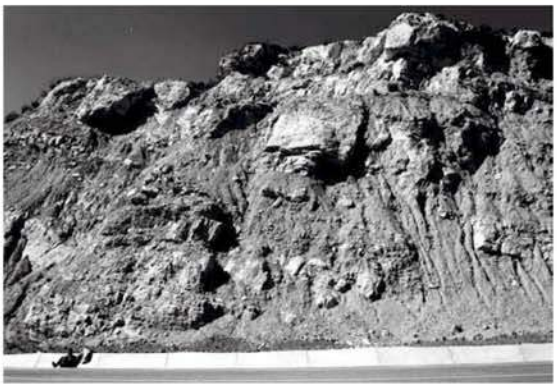


Fig. 14
Photo showing unstable slope

Fig. 15
Example of exposure characterization

ITC/TUD ENGINEERING GEOLOGY	exposure characterization		SSPC - SYSTEM
LOGGED BY: zz	DATE: 10/04/96	TIME: 16:00 hr	exposure no: example (old slope)

WEATHER CONDITIONS		LOCATION		
Sun:	cloudy/fair/**bright**		map no:	472-i
Rain:	**dry**/drizzle/slight/heavy	Map coordinates:	northing:	4,558,750
			easting:	321,625

METHOD OF EXCAVATION (ME)	
(tick)	
natural/hand-made	✔ 1.00
pneumatic hammer excavation	0.76
pre-splitting/smooth wall blasting	0.99
conventional blasting with result:	
good	0.77
open discontinuities	0.75
dislodged blocks	0.72
fractured intact rock	0.67
crushed intact rock	0.62

DIMENSIONS/ACCESSIBILITY							
Size total exposure:	(m)	l:	200	h:	5	d:	50
mapped on this form	(m)	l:	200	h:	5	d:	50
Accessibility:			poor/fair/**good**				

FORMATION NAME: tg23 thin bedded units

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
brownish off-white	fine	thin bedded, small tabular	slightly	limestone and dolomite

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa	Lumps broken by heavy hammer blows	
100 - 200 MPa ✔	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✔ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			B1	J2	J3	4	5
Dip direction		(degrees)	082	310	244		
Dip		(degrees	30	87	62		
Spacing (DS)		(m)	0.03	0.04	0.03		
Persistence	along strike	(m)	> 200	> 14	0.2		
	along dip	(m)	> 50	> 20	0.2		
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl) (on an area between 0.2 x 0.2 and 1 x 1 m2)	wavy: slightly wavy: curved: slightly curved straight	1.00 0.95 0.85 0.80 0.75	1.00	0.75	0.75		
Roughness small scale (Rs) (on an area of 0.2 x 0.2 m2)	rough stepped smooth stepped polished stepped rough undulating smooth undulating polished undulating rough planar smooth planar polished planar	0.95 0.90 0.85 0.80 0.75 0.70 0.65 0.60 0.55	0.75	0.60	0.95		
Infill material (Im)	cemented/cemented infill no infill - surface staining non softening & sheared material, e.g. free of clay, talc, etc.: coarse / medium / fine soft sheared material, e.g. clay, talc, etc.: coarse / medium / fine gouge < irregularities gouge > irregularities flowing material	1.07 1.00 0.95 / 0.90 / 0.85 0.75 / 0.65 / 0.55 0.42 0.17 0.05	0.55	0.55	1.00		
Karst (Ka)	none karst	1.00 0.92	0.92	0.92	0.92		

EXISTING SLOPE?	
dip-direction/dip	180/ 70
height:	5m
Stability (tick)	
stable	✔ 1
small problems	2
large problems	3

notes:
1) For infill 'gouge > irregularities' and 'flowing material' small scale roughness = 0.55.
2) If roughness is anisotropic (e.g. ripple marks, striation, etc.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)			remarks:
degree of weathering:	date excavation:	remarks:	
slightly	> 40 years old	old road cuts, hand-made or small shovel ?	

error distributions for the measurement of a characteristic value for a particular rock mass parameter within a geotechnical unit. Most of the distributions of rock mass parameters were about normal, although some were discrete or showed a non-normal behaviour near the limit values of the ranges allowed. As the differences were small, however, in the probability analyses the non-normal and discrete distributions were replaced by a continuous normal distribution. The standard deviations of these normal distributions, either direct or expressed as a percentage of the mean (characteristic) value, were taken as the standard error of the characteristic value of a rock mass parameter. The standard errors are not the same for all geotechnical units, as those with a wider range of permitted values will probably also have a wider distribution of characteristic values and thus a larger standard error. In the research area, however, they were approximately identical in different rock mass types. This implies that different observers divide different rock masses in geotechnical units for slope stability assessment such that the variation allowed in a unit is similar. It is therefore considered realistic to assume that the error distributions are representative for measuring a characteristic parameter value in a geotechnical unit.

Results and example

Results

Figure 13 compares the results of slope stability assessments for 184 slopes following the SSPC system with results using the Haines (Haines and Terbrugge 1991) and SMR (Romana 1985) systems. The calculation of the stability of a slope with the SSPC system gives a more distinctive differentiation between stable and unstable conditions than with either the Haines or SMR systems. In addition, the correlation between the visually estimated slope stabilities and the predictions of stability of the SSPC system is better than the correlation with the other classification systems.

Example

The SSPC system, as applied to the thinly bedded units in a slope of newly blasted (1988) limestone and dolomite, originally with a face of about 75° to 80° (see Fig. 14), is shown in Figs. 15, 16, and 17. The present (2000) angle of the face is between 60° and 70°. The slope consists of interlayered thin bedded (above the seated person in Fig. 14) and medium to thickly bedded units. The same thinly-bedded units are found exposed in road cuts less than 50 m away which are known to be more than 40 years old. These old road cuts, with slopes of 60° to 70° and heights of about 5 m, are still (1995) stable, very little or no degradation of the rock mass is observed and the material appears only slightly weathered. The method of excavation used for these old slopes was either hand shovels or small mechanical shovels.
The slope directions in the old and new road cuts are approximately equal and the general position of the old road cuts is comparable to that of the new road cut. Both the old and new road cuts were excavated into a hill that flattens above them. Any surface flow of heavy rainfall is therefore likely to be the same for both the old and new cuts. In addition, with respect to geology (faults, etc.), no major differences have been noted between the old and the new road cuts.
The new road cut is clearly unstable, large parts show rill erosion and erosion of the thinly bedded units is causing undercutting of the more thickly bedded horizons, making these unstable. The general impression of the slope is extremely poor. On close examination, those parts of the slope that appear to be 'soil' are in fact the thinly bedded units, moderately to highly weathered, which are only partly covered by topsoil transported from higher parts of the slope. In some places the thinly bedded units would be classified as moderately or highly weathered for at least 0.5 to 1 m into the rock mass. The structure and coherence of the rock mass and in particular of the thinly bedded units have been disturbed by the method of excavation. Discontinuities have opened, blocks are displaced and at many locations the intact rock is fractured or occasionally crushed as a result of blasting. This has disturbed the structure of the rock mass so severely that water can flow through the near-surface rock slope and cause weathering of the thinly bedded units. The slope is not at risk due to sliding or toppling along discontinuities.
The SSPC system gives a probability of stability of >95% for the old road cuts with slope faces of 70° and a height of 5 m. The same rock mass characteristics were used for the new slope as both slopes are in the same 'reference' rock mass as far as the thinly bedded units are considered. For a new road cut with a height of 13.8 m, a 'moderate' degree of rock mass weathering and 'dislodged blocks' due to blasting, the stability assessment was about 8% for a 70° slope. This is in agreement with reality as in its present condition (2000) the rock mass is clearly not able to support a slope of 70°. According to the SSPC system, stability will be achieved if the slope angle is decreased to about 45°.
This example shows that the SSPC classification of slope stability is also applicable in situations where the stability is governed by damage due to the method of excavation and the influence of weathering. If the slope had been designed using the SSPC system, the increased weathering would not have been anticipated as the old road cuts do not show this. However, the new road cut would never have been designed with a steep face of 80° if it was anticipated that perhaps poorly executed blasting would be used.

Discussion

The SSPC system (although based on a large variation of lithologies and rock mass types) has been developed in a ▶

Fig. 16
Example of reference rock mass calculation

ITC/TUD ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY: *zz*	DATE: *10/04/96*	exposure no: *example*
REFERENCE UNIT NAME: *Tg23 thin bedded units*		

INTACT ROCK STRENGTH (RIRS)

RIRS = IRS (in MPa) / WE (correction for weathering) = *150/ 0.95* = *158*

DISCONTINUITY SPACING (RSPA)

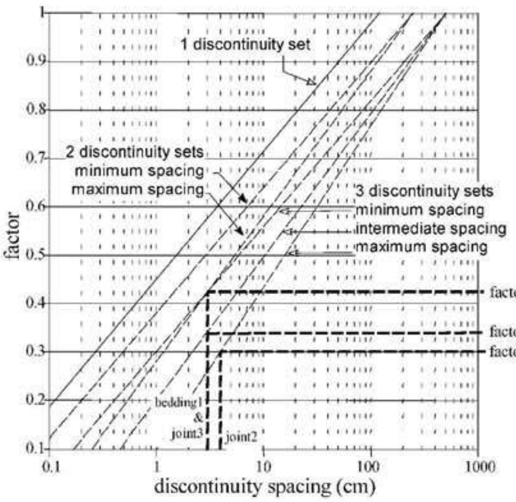
DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	*082*	*310*	*244*		
Dip	(degrees)	*30*	*87*	*62*		
Spacing (DS)	(m)	*0.03*	*0.04*	*0.03*		

SPA (see figure below) = factor1 * factor2 * factor3 =
*0.43 * 0.30 * 0.34 = 0.043*

corrected for weathering and method of excavation:

RSPA = SPA / (WE * ME)
(with a maximum of 1.00)
RSPA = *0.043/ (0.95 * 0.99)* = *0.046*

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:



CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES		B1	J2	J3	4	5
Roughness large scale	(Rl)	*1.00*	*0.75*	*0.75*		
Roughness small scale	(Rs)	*0.75*	*0.60*	*0.95*		
Infill material	(Im)	*0.55*	*0.55*	*1.00*		
Karst	(Ka)	*0.92*	*0.92*	*0.92*		
Total	(Rl*Rs*Im*Ka = TC)	*0.38*	*0.23*	*0.66*		
RTC		*0.38*	*0.23*	*0.66*		

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 - 1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC_1}{DS_1}+\frac{TC_2}{DS_2}+\frac{TC_3}{DS_3}}{\frac{1}{DS_1}+\frac{1}{DS_2}+\frac{1}{DS_3}} = \frac{\frac{0.38}{0.03}+\frac{0.23}{0.04}+\frac{0.66}{0.03}}{\frac{1}{0.03}+\frac{1}{0.04}+\frac{1}{0.03}} = 0.44$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = *0.44/ 0.95* = *0.46*

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

φ_{RRM} = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779 (if RIRS > 132 MPa then RIRS = 132)

φ_{RRM} = *132* * 0.2417 + *0.046* * 52.12 + *0.46* * 5.779 = *37°*

coh_{RRM} = RIRS * 94.27 + RSPA * 28629 + RCD * 3593 (if RIRS > 132 MPa then RIRS = 132)

coh_{RRM} = *132* * 94.27 + *0.046* * 28629 + *0.46* * 3593 = *15413* Pa

notes: 1) For IRS (intact rock strength) take average of lower and higher boundary of class.
2) Roughness values should be reduced or shear strength has to be tested if discontinuity roughness is non-fitting.
3) WE = 1.00 for 'soil type' units, e.g. cemented soils, etc..
4) If more than three discontinuity sets are present in the rock mass then the reference rock mass friction and cohesion should be calculated based on the combination of those three discontinuities that result in the lowest values for rock mass friction and cohesion.

ITC/TUD ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *10/04/96*	slope no: *example (new slope)*

	LOCATION	map no:	*472-i*
	Map coordinates:	northing:	*4,558,850*
		easting:	*321,725*

DETAILS OF SLOPE

METHOD OF EXCAVATION (**SME**)		WEATHERING (**SWE**)			
(tick)		(tick)			
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees):	*180*
pneumatic hammer excavation	0.76	slightly	0.95		
pre-splitting/smooth wall blasting	0.99	moderately	✓ 0.90	**Slope dip** (degrees):	*70*
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35	Height (**Hslope**) (m)	*13.8*
open discontinuities	0.75				
dislodged blocks	✓0.72			:	
fractured intact rock	0.67	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
crushed intact rock	0.62				

SLOPE UNIT NAME: *H (Carboniferous) slate, v. thin cleavage*

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (**SIRS**)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = *158* * *0.90* =	*142*

DISCONTINUITY SPACING (**SSPA**)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope) SSPA = *0.046* * *0.90* * *0.72* =	*0.030*

CONDITION OF DISCONTINUITIES (**SCD**)

SCD = RCD (from reference rock mass) * SWE (weathering slope) SCD = *0.464* * *0.90* =	*0.418*

SLOPE UNIT FRICTION AND COHESION (**SFRI** & **SCOH**)

φ_{SRM} = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779 (if SIRS > 132 MPa then SIRS = 132) φ_{SRM} = 1*32* * 0.2417 + *0.030* * 52.12 + *0.418* * 5.779 =	*34*°
coh_{SRM} = SIRS * 94.27 + SSPA * 28629 + SCD * 3593 (if SIRS > 132 MPa then SIRS = 132) coh_{SRM} = 1*32* * 94.27 + *0.030* * 28629 + *0.418* * 3593 =	*14146*Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (**Hmax**)

Maximum possible height: Hmax = 1.6 * 10^{-4} * coh_{SRM} * sin(slope dip) * cos ($_{SRM}$) / (1-cos(slope dip - $_{SRM}$)) Hmax = 1.6 * 10^{-4} * *14146* * sin(*70*°) * cos(*34*°) / (1-cos(*70*° - *34*°)) =		*9.3*m
ratios:	$_{SRM}$ / slope dip = *34*° / *70*° =	*0.49*
	Hmax / Hslope = *9.3*m / *13.8*m =	*0.67*
Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability:		*8*%

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		*B1*	*J2*	*J3*	4	5
Dip direction	(degrees)	*082*	*310*	*244*		
Dip	(degrees)	*30*	*87*	*62*		
With, Against, Vertical or Equal		*a*	*v*	*w*		
AP	(degrees)	*-5*	*-85*	*40*		
RTC (from reference form)		*0.38*	*0.23*	*0.66*		
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		*0.37*	*0.22*	*0.65*		
Probability stable:		*100*%	*100*%	*97*%	%	%

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: $\delta = \sigma - \tau$: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

(graphs: sliding figure 9; toppling figure10; orientation independent stability figure 11)

Fig. 17
Example of slope stability calculation

particular region, in a particular climate and with particular types of lithologies and rock masses, etc. As for all empirical systems, using the SSPC system on rock masses in an environment that is very different implies a risk.

The quality of a rock mass consisting of a highly inhomogeneous, intensely folded or faulted rock presents a special problem. The rock mass should be divided in geotechnical units in which the rock mass properties are broadly homogeneous and can be calculated for each specific geotechnical unit. If it is impossible to distinguish geotechnical units with a suitably small range of permitted values for properties, due to the limited size of the inhomogeneous areas, the worst case rock mass parameters can be used, although this would probably lead to an over-conservative assessment.

Rock types that are deformed very easily (gypsum, salts, etc.) were included in the development of the SSPC system. However, the stability of slopes in rock masses containing gypsum is influenced more by erosion and weathering (in particular solution of gypsum) than by mechanical deformation of the rock. The SSPC system cannot be used if the strength of the rock mass is governed by deformation of the intact rock.

It should be noted that the SSPC classification was developed for uniform plane slopes, while real slopes (and in particular, those poorly excavated) contain re-entrants, niches, overhangs, etc. which may allow slope movement in directions that would not be possible if the slope was one continuous plane. Rock falls resulting from such slope irregularities are not uncommon.

Generally, the errors made in assessing the rock mass field data by students are larger than would be expected from experienced rock mechanics engineers. Consequently, the slope stability probabilities calculated by the SSPC system may be conservative. In the opinion of the authors, this is not a problem as the SSPC system is likely to be used by experienced and inexperienced users. Experienced users will note that the results based on the SSPC system may be conservative and will interpret the results accordingly while it is highly unlikely that an inexperienced user would be able to recognize that the results are too optimistic and be able to correct for this. Conservatism in the results is therefore considered to be advantageous.

The SSPC system does not assess stability for slopes that are subject to external stresses, such as tectonic stresses or stresses induced by a (large) hill or mountain behind the investigated slope. A criterion for buckling such as proposed for the sliding and toppling criteria could not be developed. This is in agreement with field observations in the research area where buckling as a cause of slope failure is seldom found with almost none of the slopes being sufficiently high and steep for this to occur.

Conclusions

The Slope Stability Probability Classification (SSPC) system provides a better assessment of slope stability than other slope stability classification systems because its three-step approach allows for the incorporation of past and future weathering, the damage due to excavation methods and the assignment of probabilities for different failure mechanisms. The repeatability and reliability of the characterization of rock mass properties are generally good, since the more difficult to measure or ambiguous parameters such as RQD, water and elaborate testing (UCS, shear-box tests, etc.) are not required. The SSPC system was developed using data from 184 stable and unstable slopes. The amount of data and the fact that the data were collected by a large number of different persons at different times eliminate a designer bias in the system. Susceptibility to weathering is a major factor for the stability of a slope in a rock mass prone to weathering within the engineering lifetime of the slope. The SSPC system quantifies the future strength of a discontinuity and rock mass if the future degree of rock mass weathering can be predicted. This methodology is independent of the climate. The system has recently been used with good results in Austria, South Africa, New Zealand (Lindsay et al. 2000) and in the Dutch Antilles (Rijkers and Hack 2000) and it is considered that it will also be applicable outside these areas.

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Erratum

A new approach to rock slope stability – a probability classification (SSPC)

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Unfortunately, there were some errors in Eq. 13 in the HTML versions. This equation is given correctly below.

$$
\begin{array}{ll}
\text{For each slope } j: & \\
\begin{array}{l}\text{visually estimated}\\ \text{stability} = \text{stable}\end{array} & \left\{\begin{array}{l} \frac{\varphi_{mass}}{dip_{slope}} \geq 1(stable) \rightarrow er = 1 \\ \frac{\varphi_{mass}}{dip_{slope}} < 1 \left\{\begin{array}{l} \frac{H_{\max}}{H_{slope}} \geq 1(stable) \rightarrow er = 1 \\ \frac{H_{\max}}{H_{slope}} < 1(unstable) \rightarrow er = \frac{H_{slope}}{H_{\max}} \end{array}\right. \end{array}\right. \\
\begin{array}{l}\text{visually estimated}\\ \text{stability} = \text{unstable}\end{array} & \left\{\begin{array}{l} \frac{\varphi_{mass}}{dip_{slope}} \geq 1(stable) \rightarrow er = \frac{\varphi_{mass}}{dip_{slope}} \\ \frac{\varphi_{mass}}{dip_{slope}} < 1 \left\{\begin{array}{l} \frac{H_{max}}{H_{slope}} \leq 1(unstable) \rightarrow er = 1 \\ \frac{H_{max}}{H_{slope}} > 1(stable) \rightarrow er = \frac{H_{max}}{H_{slope}} \end{array}\right. \end{array}\right. \\
ER = \sum_j er_j &
\end{array}
\qquad (13)
$$

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GLOSSARY

Definitions for rock, rock mass and their properties are not used uniformly in the literature. Therefore, definitions of terminology that is frequently used are listed below to avoid confusion. Geological terminology is based on the Dictionary of Geological Terms of the American Geological Institute (AGI, 1976) and the Geological Nomenclature of the Royal Geological and Mining Society of the Netherlands (KNGMG, 1980).

i-angle

See 'bi-linear shear criterion'.

φ_{basic}, $\varphi_{discwall}$

In these notes φbasic is not used. In stead is used φdiscwall. φdiscwall denotes the friction of a non-displaced (fitting) discontinuity which friction does not cause opening of the discontinuity (dilatancy). Confusion has arisen in the literature about φbasic. Some authors use φbasic also for the φm of rock material, for φresidual (which is the φ obtained after large displacements), or use the term for artificial surfaces (saw cuts). See further 'bi-linear shear criterion'.

φ_m

See 'bi-linear shear criterion'.

$\varphi_{sliding\ angle}$

Angle obtained by using "sliding criterion"

Abutting

(discontinuities) See persistence.

Anisotropy

The dependency on direction of properties of rock or rock mass.

Bi-linear shear criterion

See ch. C.2.1.

Backbreak

The damage (fractures, opening of discontinuities, etc.) inflicted by the method of excavation on a soil or rock mass beyond the circumference of an excavation (see also overbreak).

Break

See back- and overbreak

Characteristic discontinuity orientation

The characteristic discontinuity orientation is the mean of the orientations of the discontinuities in a discontinuity set.

Characteristic discontinuity spacing

The spacing of discontinuities within one set of discontinuities is defined as the perpendicular distance between two discontinuity planes. The characteristic discontinuity spacing is the mean of the spacings between discontinuities in a discontinuity set.

Characterization

Characterization is the description of a unit. A characterization is not automatically a classification.

Cleavage (slaty cleavage)

A tendency to cleave or split along definite, parallel, closely spaced planes, which may be highly inclined to the bedding planes. It is a secondary structure, commonly confined to bedded rocks, is developed by pressure, and ordinarily is accompanied by at least some re-crystallization of the rocks. (In this study used for Carboniferous rocks which contain a 'slaty cleavage').

Classification

Classification is the characterization (description) of a unit by standard parameters which are empirically related to an engineering application. A weighting of the parameters according to standard rules will lead to a recommendation for an engineering application.

Cohesion (apparent)

For the strength description of rock, rock mass and soil see 'Mohr-Coulomb failure criterion'; for discontinuities see 'bi-linear shear criterion'.

Compressive strength

See ch. B

Creep

Creep in rock mechanics is a confusing term. Various forms of plastic and time dependent deformation processes which are governed by totally different physical or chemical processes are described as creep.

Cutterhead

The round rotating front of a tunnel boring machine on which the cutters are mounted to excavate the mass.

Cutting wheel

See cutterhead

Day-lighting

'Day-lighting' denotes that a discontinuity has a dip less than, but in the same general direction as, the slope dip, and is outcropping in the slope: the difference between slope dip-direction and discontinuity dip-direction should be less than 90° and the slope dip should be steeper than the discontinuity dip.

Deformation

Deformation of intact rock or of a rock mass is the change in volume or shape.

Dilatancy

The tendency of a fitting discontinuity to open perpendicular to the discontinuity plane if sheared along the discontinuity because the asperities on the discontinuity surface are overridden.

Discontinuity

A discontinuity is a plane which marks a change in the physical or chemical characteristics of rock material.

Single discontinuity A single discontinuity denotes a single isolated discontinuity (single fault, isolated crack or joint, etc.) that is not part of a discontinuity set or, if part of a discontinuity set, then the spacing between the different discontinuities is so large that for practical engineering purposes the discontinuity may be considered as a single feature..

Discontinuity set A discontinuity set or discontinuity family denotes a series of discontinuities of which the geological and mechanical characteristics as well as their orientation are broadly the same (examples are: sets of bedding planes, schistosity planes, cleavage planes, joint sets, etc.).

Integral discontinuities Integral discontinuities are discontinuities for which there is no change in strength compared to the surrounding rock material. Intact rock may contain integral discontinuities.

Mechanical discontinuities Mechanical discontinuities are planes of physical weakness. Bedding, joints, fractures, faults, etc. are mechanical discontinuities if the tensile strength perpendicular to the discontinuity or the shear strength along the discontinuity are lower than in the surrounding rock material.

Mechanical discontinuities will in general be the boundaries for 'banks' of intact rock. The term bank is, however, not used as the definition of a bank is based on sedimentological characteristics.

In these notes 'discontinuities' is used for mechanical discontinuities except where otherwise stated.

Discontinuous rock mass

A rock mass containing discontinuities (see also rock mass).

Engineering lifetime

Engineering lifetime denotes the expected existence of an engineering structure. Slopes are often designed for a lifetime of about 50 years.

Failure mechanisms and modes

Processes leading to slope failure are divided into different mechanisms that are sub-divided into different modes. For example, slope failure mechanisms are shear displacement, deterioration of rock material, intact rock creep, etc.; the resulting failure modes of the shear displacement mechanism are plane sliding, wedge failure, partially toppling and, to some extent, buckling.

Fitting discontinuity

A discontinuity in which the asperities of both discontinuity walls are complementary and the discontinuity walls are not displaced. Displacement along a fitting discontinuity can only take place if the asperities are sheared off, deformed or if the asperities are overridden (causing dilatancy; see before).

Non-fitting discontinuity: the asperities are not complementary or the opposing discontinuity walls have been displaced causing that the asperities are not fitting.

Formation

The primary unit of formal geological mapping or description. Most formations possess certain distinctive or combinations of distinctive (lithological) features. Boundaries are not based on time criteria.

Friction (φ)

For the strength description of rock, rock mass and soil see 'Mohr-Coulomb failure criterion'; for discontinuities see 'bi-linear shear criterion'.

Geotechnical unit

See unit - geotechnical.

Gouge

Claylike material containing rock fragments of the surrounding rock, occurring between the walls of a continuous discontinuity (mostly: faults or major discontinuities) as a result of wear during displacement. In a discontinuity described as a 'gouge filled' discontinuity the rock fragments in the discontinuity do not make contact with both discontinuity walls; thus the initial shear strength is governed by the clay material.

Identification

Identification describes the effect that a relation is defined that includes more parameters then necessary to relate the data. The parameters are not determined by the relation. For example:

$$\begin{gathered} y = (a + b) * x \\ x, y = data \quad a, b = parameters \end{gathered} \qquad [\text{P-1}]$$

Both a and b can never be determined from this relation whatever the number of (x, y) data pairs. (Obviously for determination of (a + b) only one data pair (x, y) is sufficient.)

In optimization of complex relation(s) identification problems might not be recognized leading to ambiguous results.

Inhomogeneity

Inhomogeneity is the spatial variation of properties of intact rock or of a rock mass.

Intact rock

Intact rock blocks are blocks of rock for which: 1) The physical and mechanical properties are roughly uniform. 2) The particles (mineral grains, rock grains, etc.) are bounded by a cementing agent which causes a block of intact rock to have a tensile strength. 3) An intact rock block does not contain mechanical discontinuities.

Isotropy

Isotropy designates that properties of intact rock or of a rock mass are not direction dependent.

Lithology - lithological

The science of the rocks; in this study lithology denotes the type of minerals, their origin or sedimentation environment.

Lithostratigraphic (sub-) unit

See unit - lithostratigraphic

Lubrication

Lubrication by water may reduce the shear strength of discontinuities. The effect may be caused by the water itself that changes the mechanical characteristics of some materials. Another effect that is more general is that the presence of water will cause a reduction in friction because the surface stresses of water will cause a reduction of the normal stresses of the discontinuity walls. The quantity of water is not necessarily so large that an overall water pressure is established.

Luster

The appearance of a stone's surface (or of a mineral in general) in reflected light. Refraction index and perfection of polish possessed by the stone are the main factors affecting luster, while hardness is also of some importance.

Mapping unit

See unit - mapping.

Mohr-Coulomb failure criterion

The 'Mohr-Coulomb failure criterion' consists of a linear envelope touching all Mohr's circles representing critical combinations of principal stresses in the rock or rock mass, or soil:

$$\tau_{failure} = cohesion + \sigma_{failure} * \tan(\varphi) \quad \text{[P-2]}$$
$$cohesion \text{ and } \varphi \text{ are the cohesion and angle of internal friction of the material}$$

Expressed in the 'Mohr-Coulomb failure criterion' the unconfined compressive strength (UCS) equals:

$$UCS = 2 * cohesion * \tan\left(45° + \frac{\varphi}{2}\right) \quad \text{[P-3]}$$

The relation between minor (σ3) and major (σ1) principal stress at failure is:

$$\sigma_1 = UCS + \sigma_3 * \tan^2\left(45° + \frac{\varphi}{2}\right) \quad \text{[P-4]}$$

Non-fitting discontinuity

See fitting discontinuity.

Non-persistent discontinuities

See persistence.

Orientation

See characteristic discontinuity orientation.

Overbreak

The amount of soil or rock that is (accidentally) excavated beyond the planned circumference of an excavation (see also backbreak).

Overcoring

Overcoring is done in a borehole by first drilling a small diameter borehole. In the hole, a device is fixed which will measure strain differences. Then the same hole is drilled with a large diameter bit and the piece of rock where the strain measurement device is mounted is overcored.

Overfit

Overfit describes the effect that a relation is defined that includes more parameters then necessary to relate the data. In optimization scatter on the data will cause that multiple, equally good, solutions are found. Each solution is a solution on different (clustered) subsets of the data set. None of these solutions need to be the solution for the full data set.

Outlier

An outlier is a data point that is clearly detached, or out from the main set of data points.

Persistence, Persistent discontinuities, Abutting discontinuities, Non-persistent discontinuities

See ch. C.1.

Porphyritic, porphyrite

A textural term for those igneous rocks in which larger crystals are set in a finer groundmass.

Rock mass

A rock mass is a mass of rock blocks with or without discontinuities. A rock mass may be homogeneous or inhomogeneous. Based on rock mass parameters the rock mass is divided in homogeneous geotechnical units.

Rock (mass) failure

A rock mass is supposed to have failed if the rock mass deforms more than allowed for a safe engineering application.

Shear strength

The shear strength is the shear stress at failure of a sample under a shear stress. See for shear strength along a discontinuity ‘bi-linear shear criterion’.

Slaty cleavage

See cleavage.

Slickensided

Usually striated surface of rock produced by friction.

Sliding angle

Friction obtained by using the ‘sliding criterion’

Soil type units

‘Soil type’ units describe units which consist of loosely cemented grains or small particles, generally either without clearly defined mechanical discontinuities or having highly irregular and thinly laminated mechanical discontinuities, and having a low intact rock strength. ‘Soil type’ units resemble cemented soils rather than a rock mass.

Spacing

See characteristic discontinuity spacing.

Spalling

Spalling is the breaking of intact firm cohesive or cemented soil or intact rock from an excavation wall due to the stress in the wall of the excavation.

Striated

Surface of rock characterized by fine, narrow, curved or straight parallel grooves.

Stylolite

A term applied to parts of certain limestones which have a column like development; the columns being generally at right angles or highly inclined to the bedding planes, having grooved, sutured or striated sides, and irregular cross sections. Stylolites result from solution under pressure of limestone. The clay particles which were originally in the limestone, remained on the solution surface.

Susceptibility to weathering

See weathering.

Tactile roughness

Roughness that can be felt by using fingers.

Tensile strength

The tensile strength is the tensile stress at failure of a sample under a tensile stress.

Tilt-angle

Friction angle obtained by doing a tilt test

Triaxial compressive strength

See compressive strength.

Unconfined Compressive Strength (UCS)

See compressive strength.

Unit

The following definitions are used in these notes:

Lithostratigraphic unit

A layer or a body of layers characterized by consisting dominantly of a certain lithologic type (sand, clay, sandstone, shale, granodiorite, etc.).

Lithostratigraphic sub-unit

A lithostratigraphic unit which characteristic bedding or cleavage spacing is within the ranges for discontinuity spacing as given by BS 5930 (1981).

Geotechnical unit

A geotechnical unit is a part of the rock mass in which the mechanical characteristics of the intact rock material are uniform in each block of intact rock and the mechanical properties (including orientation) of the discontinuities within each set of discontinuities are uniform. Anisotropy of properties, if present, is uniform.

Engineering geology mapping unit

The divisions made on an engineering geological map.

Weathering

Weathering is the chemical and physical change in time of intact rock and rock mass material under influence of atmosphere and hydrosphere (temperature, rain, circulating ground water, etc.). A distinction is made between 1) the degree (state) of weathering (at a certain moment) and 2) the susceptibility to weathering (in a certain time-span).

IFIGURES

Hack, R., 2006. Discontinuous rock mechanics; lecture note, 5.0 ed. ITC lecture note. ITC, Enschede. p. 334.

TABLES