Norwegian Group for Rock Mechanics (NBG)

Handbook No 2

ENGINEERING GEOLOGY AND ROCK ENGINEERING

2000

Prepared in co-operation with

NFF NORWEGIAN TUNNELING SOCIETY

ENGINEERING GEOLOGY AND ROCK ENGINEERING

MANUAL

NORWEGIAN SOIL AND ROCK ENGINEERING ASSOCIATION

PUBLICATION NO. 12

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NORWEGIAN SOIL AND ROCK ENGINEERING ASSOCIATION

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NORWEGIAN SOIL AND ROCK
ENGINEERING ASSOCIATION

NORWEGIAN ROCK MECHANICS GROUP, NBG
Affiliated to
International Society of Rock Mechanics (ISRM)
International Association of Engineering Geology and the Environment (IAEG)

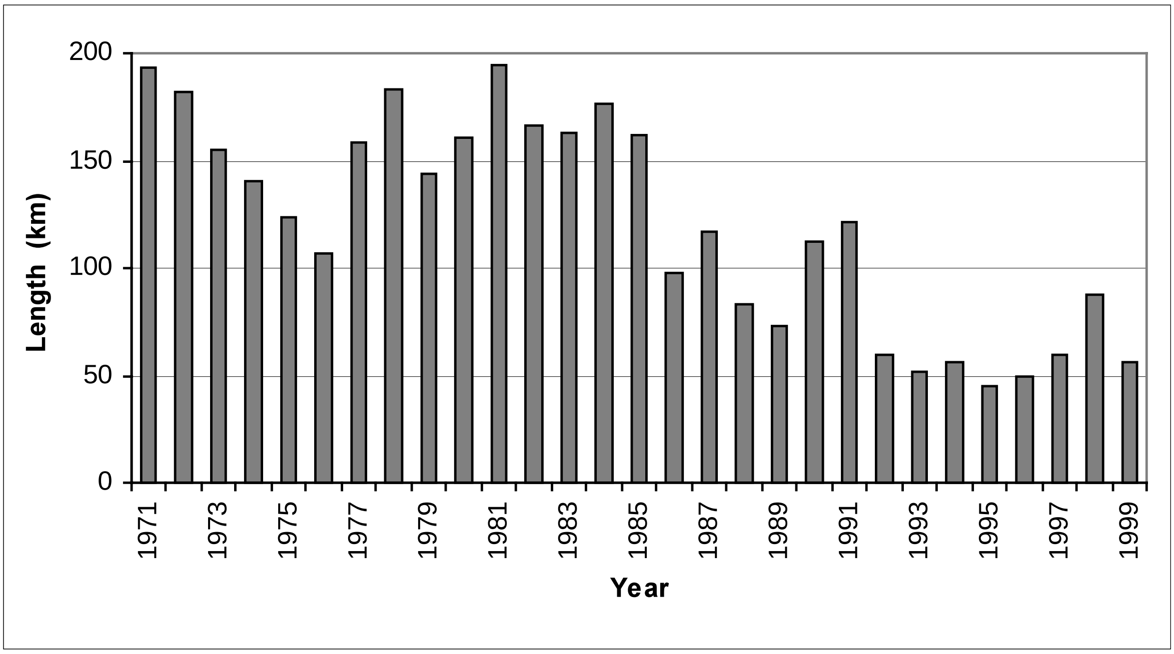
ISBN 82-91341-33-8

The Authors are responsible for the picture on the cover page.

Most of Norway is mountainous. In recent geological times mighty glaciers formed the land leaving a surface where the geological features can often be easily observed.

The country's geology provides a bonus, since its mountains consist mainly of solid rock, which makes the construction of underground and above ground facilities relatively inexpensive and therefore attractive.

There has been a lot of activity in rock construction here during the past 75 years, starting with hydropower development. During the last 50 years the construction of road tunnels has increased gradually, and in the beginning of the 1980s construction includes several undersea tunnels.



Tunnels excavated annually in Norway in terms of length (kilometres per year).

Preface

The Norwegian Rock Mechanics Group (NBG) first issued an engineering geology and rock mechanics handbook in Norwegian in 1974. This book, and the revised 1985 edition of 140 pages, covered short descriptions of rocks, rock masses, rock stresses, groundwater, classification, landslides, symbols, and list of definitions.

The present manual is the result of an increasing need for a more comprehensive book in English, and represents an extensive revision and extension of the previous handbook. The purpose of this manual is to present the main issues of rock engineering in a simple, clear and logical form. The ambition is not only to give short definitions, but also to quantify the terms and their relation to other parameters in engineering geology and rock engineering. Generally known, routine methods and measurements are only briefly described, while new developments and more unknown methods are described in detail.

A large amount of literature is available in a great number of books and papers scattered among many engineering and scientific periodicals and the proceedings of several conferences. From this a selection of references is provided in this manual. It is hoped that the guide can be used by professionals as a reference in their daily work both at home and abroad, and that it will be of help to people working with planning and construction in rock engineering; providing easy access to international definitions, classifications, descriptions and symbols.

The two authors are:

Björn Nilsen, Dr. ing. Professor of geological engineering at Department of Geology and Mineral Resources Engineering, the Norwegian University of Science and Technology (NTNU), Trondheim. He has 25 years of experience in engineering geology/rock engineering, covering teaching and research as well as more applied aspects. Several projects in Norway and abroad have made use of him as an expert advisor.

Arild Palmström, Ph.D. Senior advisor in engineering geology and rock mechanics at Norconsult as, Sandvika. He has more than 30 years experience in planning, investigation, design and supervision of numerous rock construction projects in Norway and abroad. He is the author of more than 50 scientific papers in engineering geology on rock construction works for hydropower plants, sub-sea tunnels, traffic tunnels, and storage caverns in rock.

Both authors are former chairmen of the Norwegian Rock Mechanics group.

Many colleagues have contributed with valuable suggestions and advice in the process of writing this book. The authors would like to express their thanks to all of these people. They would also particularly like to acknowledge the input from the review committee appointed by NBG, which consisted of Per Bollingmo, Eystein Grimstad, Bent Aagaard and Hanna Rachel Broch. The authors are also particularly indebted to Einar Breiseth at the Department of Geology and Mineral Resources Engineering, NTNU, for the final draft of many of the figures in this book, and to Pat Coughlin for editing help and language improvements.

The book has been financed by the Norwegian Rock Mechanics Group, the Norwegian Tunnelling Society and the companies presented at the end of the book. It would, however, not have been completed without support and help from Norconsult as and NTNU, not to mention the efforts of the two authors who have spent a large portion of their leisure time on this project.

Björn Nilsen has been on sabbatical at the Colorado School of Mines (CSM) during the completion of this manual, and would also like to thank the Mining Department there for offering him excellent working conditions during that crucial stage of the work.

Oslo, May 1999

The Authors

Contents

1 Introduction

1.1 Relevant disciplines

This manual has engineering geology and rock engineering as its main focus, but also touches on related fields such as rock mechanics and geotechniques. Since there is often confusion concerning the different terms, a common understanding of their meanings and definitions as applied in this book, is given as follows:

- *Engineering geolog*y is the application of geological knowledge in engineering analysis, planning, design and construction. Emphasis here is on geology.
- *Geological engineering* has much the same meaning as engineering geology, but as indicated by the order of the words, engineering is more in focus here.
- *Rock mechanics* cover field measurements, laboratory testing and analytical methods. Calculation and numerical analyses are emphasised.
- *Rock engineering* includes engineering geology/geological engineering and rock mechanics as well as elements of civil engineering/construction engineering.
- *Soil mechanics* is the experimental and theoretical study of soil behaviour, and a parallel to rock mechanics.
- *Geotechniques* (or geotechnics), as used internationally, means the theoretical and practical aspects of all planning, design and construction related to soil and rock. In some countries, including Norway, the term is used synonymously with soil mechanics.

In Norway the term "engineering geology" as used in the context of the design, engineering and construction sectors covers all engineering work related to the planning, design and supervision of construction. The work typically includes activities such as:

- Geological mapping.
- Interpretation of geology.
- Planning and supervision of site investigations.
- Evaluation of field data for engineering geological design, i.e., cross-sections, orientation and alignment of caverns, tunnels and shafts as input to overall layout.
- Preliminary, final and detailed design of caverns, tunnels, and shafts in order to evaluate:
  - rock excavation methods,
  - rock support and water control methods,
  - cost estimates, and
  - construction time.
- Construction supervision.
- Final tunnel design during the construction period.
- Evaluation of stability and optimum rock support during construction.

Thus, the manual covers a very wide and complex area, and a solid theoretical background as well as a multi-disciplinary attitude and practical experience are necessary to cope efficiently with the fields of engineering geology and rock engineering.

1.2 Rock masses and their use

A rock mass is a material quite different from other structural materials used in civil engineering. It is heterogeneous and quite often discontinuous, but is one of the materials in the earth's crust that is most commonly used in human construction.

Ideally, a *rock mass* is composed of a system of rock blocks and fragments separated by discontinuities forming a material in which all elements behave in mutual dependence as a unit, see Figure 1.1. The material is characterised by shape and dimensions of rock blocks and fragments, by their mutual arrangement within the rock mass, as well as by joint characteristics such as joint wall conditions and possible fillings.

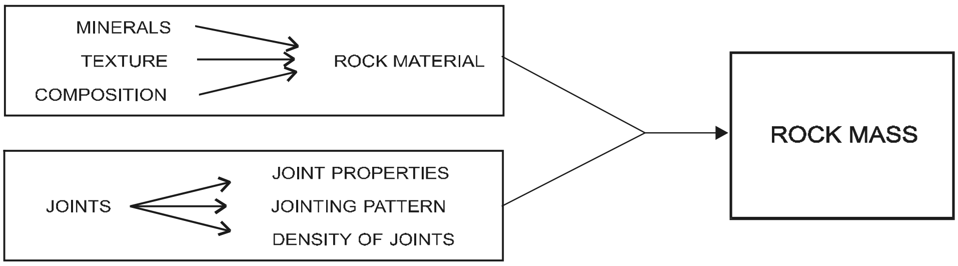


Figure 1.1 The main components of a rock mass.

A rock mass with its complicated structure, defects and inhomogeneities, and wide range of applications can provide challenges and problems in rock engineering and construction which often involve considerations that are of relatively little or no concern in most other branches of engineering. A major challenge is the uncertainties involved in the geological setting and conditions, others are the geotechnical parameters. These are distinctive features of engineering geology compared to other engineering fields, and therefore "engineering judgement" plays an important role in engineering geology and rock engineering.

The quality of the geo-data that form the basis for the calculations and estimates made is important in all work involving rock mechanics, rock engineering and design. The quality depends mainly on two aspects:

1. An understanding and interpretation of the geological setting.
   This is important mainly in the pre-construction phase and comes from field investigations and the experienced interpretation of available results. To a great extent this is often wholly dependent on the skill of the geologist(s) who decide how the investigations should be made and how the acquired geo-data should be combined. Thus, this process can in many instances be said to be more an "art" than a science.

2. The way the (known) rock mass at the site is described or measured.
   This is has to do with the use of geo-data; how these data are transformed into numerical values and which methods or systems are applied in the rock engineering and design work.

Other special aspects of a rock mass and its utilisation in contrast to other construction materials are:

- The size or volume of the material involved, see Figure 1.2.
- The structure and composition of the material.
- The many ways in which it can be used for construction and other purposes, see Table 1.1.
- The difficulties in measuring the quality of the material.

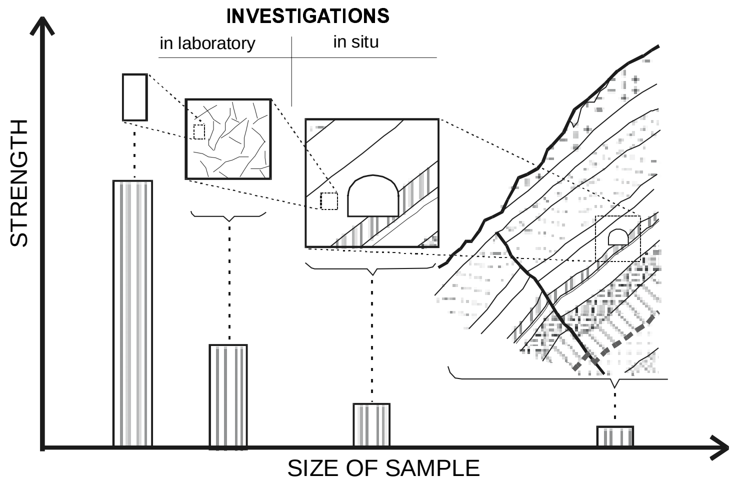


Figure 1.2 The scale factor of rock masses and the variation in strength of the material depending on the size of the "sample" involved (from Janelid, 1965).

Table 1.1 Main types of work connected with rocks and rock masses (from Palmström, 1995)

Type	Actual process or use
Excavation/ breaking of rocks	• drilling (small holes) • boring (TBM boring, shaft reaming) • blasting (tunnel, bench, cutting) • fragmentation • crushing • grinding • cutting
Application of rocks	• rock aggregate for concrete, etc. • rock fill • building/ornamental stone • industrial minerals
Utilisation of rock masses	• in underground excavations (tunnels, caverns, shafts) • in surface cuts/slopes/portals
Construction works in rock masses	• excavation works (above ground/underground) • rock support • water sealing

These factors have made special methods of data acquisition necessary, and special procedures in the application of these data for construction purposes have also been developed. Thus, the material properties of rock masses are generally not measured, but estimated from descriptions and indirect tests. In addition, the stresses acting on the material are mainly inherent in the rock mass, and not applied by the engineering works. Finally, and not the least, the construction work results in changes of the original state of stress.

It is an important principle in rock engineering to measure and use the rock mass and ground parameters of greatest significance for the actual type of design or construction. There is no single parameter or index that can fully designate the properties of jointed rock mass. Different parameters have different significance, and only when combined may they give a meaningful result. Testing of rock masses in situ has brought out very clearly the enormous variations that exist in the mechanical behaviour of a rock mass from place to place. The engineering properties of a rock mass depend far more on the system of geological discontinuities within the rock mass than on the strength of the rock itself.

2 Geology and rocks

Over 2000 names are used for igneous rocks which comprise about 25% of the earth's crust, in contrast to the greater abundance of mudrocks (35%) for which only a handful of terms exist. Yet, the mudrocks have a much wider variation in mechanical behaviour and provide greater challenges in rock constructions. From a rock-engineering point of view, the vast amount of rock types can, as shown in Table 2.3, be grouped into a reduced number of about 20 based on their strength and behaviour.

For some engineering purposes, pure numerical characterisation of rock properties may be used, for example in the evaluation of boreability, aggregates for concrete, asphalt, etc. In assessing the use of full-face tunnel boring machines (TBM), rock properties such as *compressive strength, hardness, anisotropy* are among the most important input. When evaluating stability and rock support the rock properties are mainly of importance where the rock is weak or overstressed. Despite the fact that rock properties in many cases are overruled by the effects of joints, it should be kept in mind that they very much determine the joint characteristics and how the joints have been formed.

The following sections give a brief account of some significant factors concerning the formation, composition and properties of rocks. This is not intended to replace textbooks, but to compile in an abridged form information of special interest to rock engineers and engineering geologists.

2.1 Rock composition, grain size, and texture

The engineering properties of rocks are largely determined by the character of their individual grains, such as the mineral or particle size, the shape and fabric (*texture*), and the bonds between each of them. In addition, anisotropy, weathering and alteration can highly influence the behaviour and strength properties of a rock. These properties are discussed in Sections 2.4 and 2.5.

2.1.1 Minerals

A *mineral* is a homogeneous, naturally occurring phase; by some restricted to an inorganic, crystalline phase. The main rock forming minerals are listed in Table 2.1. The minerals make up the texture and the composition of the rock, as described in the next section.

2.1.2 Texture

Texture refers to the size and shape of the minerals and their arrangement in the rock. The formation of the rock is often reflected in its texture.

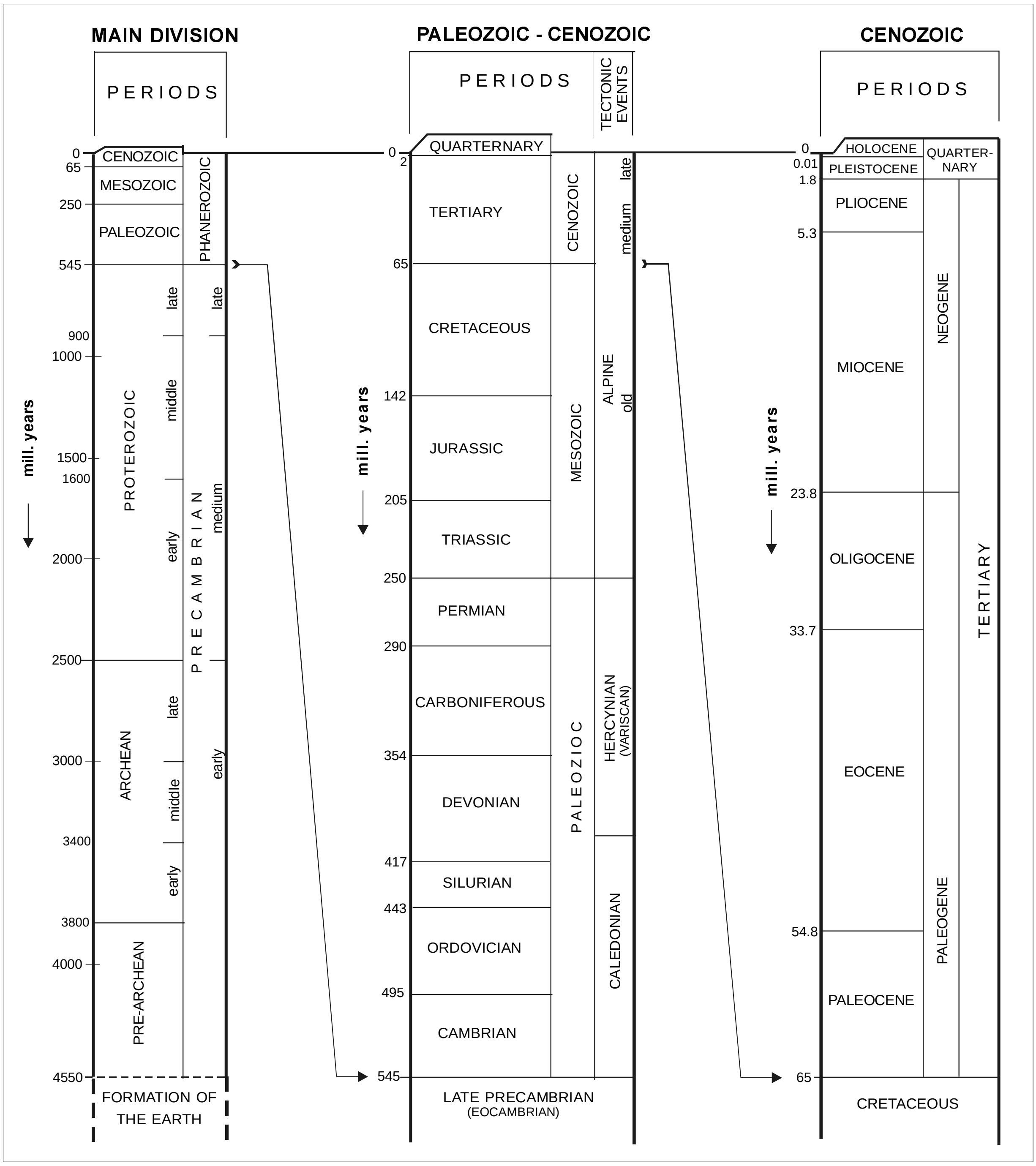


Figure 2.1 *Geological timetable.* Times are from U.S. Geological Survey (*Precambrian*) and Saga Petroleum, Norway (*Phanerozoic*).

Table 2.1 The main rock-forming minerals.

Mineral				Density	Moh's hardness	% of the earth's crust
Quartz				2.65	7	12
Feldspar	Alkaline feldspar	K feldspar	Orthoclase	2.56	6	59
			Microcline	2.56	6	
	Plagioclase	Na feldspar	Albite	2.62	6	
		Ca feldspar	Anorthite	2.76	6	
Micas			Muscovite	2.8-2.9	2.5	5
			Biotite	2.8-3.4	2.5	
Pyroxenes	Rhombic pyroxene		Enstatite Hyperstene	3.2-3.9	6	15
	Monocline pyroxene		Diopside Hedenbergite Ægirine Augite	3.2-3.6	6	
Amphiboles	Rhombic amphiboles		Anthophyllite	2.9-3.3	6	
	Monocline amphiboles		Tremolite Actinolite Hornblende	3.0-3.4	6	
Felspatoids			Nepheline	2.6	6	8
			Leucite	2.47	6	
Carbonates			Calcite	2.7-2.8	3	
			Dolomite	2.8-2.9	3.5-4	
			Magnesite	3.0	4	
Garnet				3.6-4.3	7-7.5	
Epidote				3.3-3.6	6-7	
Olivine				3.3-4.4	6.5	
Serpentine				2.5-2.6	4-6	
Chlorite				2.6-3.0	2	
Talc				2.8	1	
Pyrite				5.0	6.5	
Chalcopyrite				4.0	4.5	

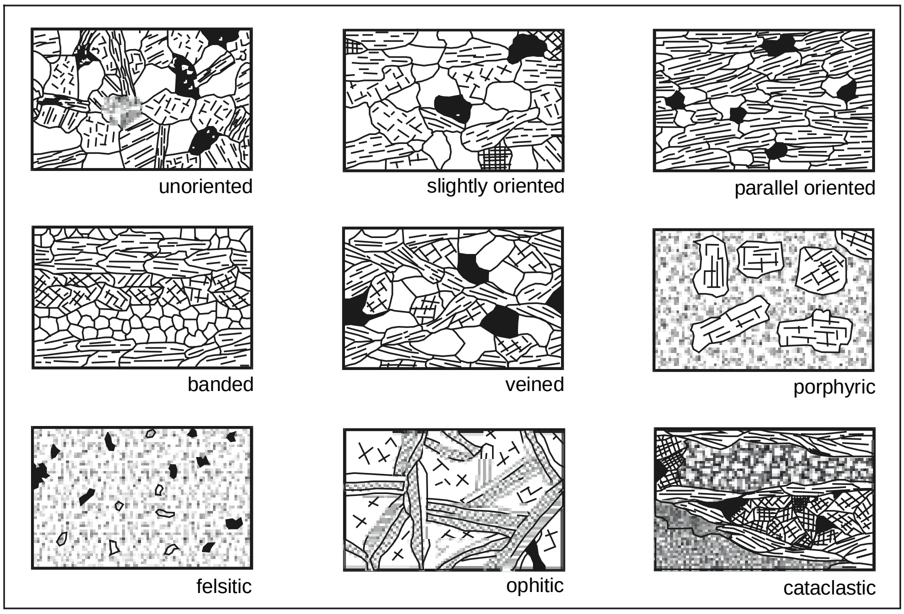


Figure 2.2 The main types of *mineral arrangement* in rocks.

Table 2.2 Classification of *mineral and grain size* (revised from British Standards Institution, 1981). In middle- and coarse-grained rocks the single minerals can easily be seen by eye.

GRAIN SIZE	SEDIMENTARY ROCKS		METAMORPHIC and IGNEOUS ROCKS			MINERAL SIZE
mm	Description	Typical rocks	Typical rocks		Description	mm
> 20	RUDACEOUS			Pegmatite	VERY COARSE	> 20
20		Conglomerate	Gneiss			20
6				Granite, gabbro	COARSE	6
2						2
0.6	ARENACEOUS coarse	Sandstones		Dolerite, diabase	MEDIUM	0.6
	medium		Schist			
0.2						0.2
	fine	Arkose Greywacke				
0.06	ARGILLACEOUS	Mudstone, shale	Phyllite, slate	Rhyolite, basalt	FINE	0.06
0.02						0.02
< 0.002		Flint, chert	Mylonite	Obsidian, volcanic glass	VERY FINE	< 0.002

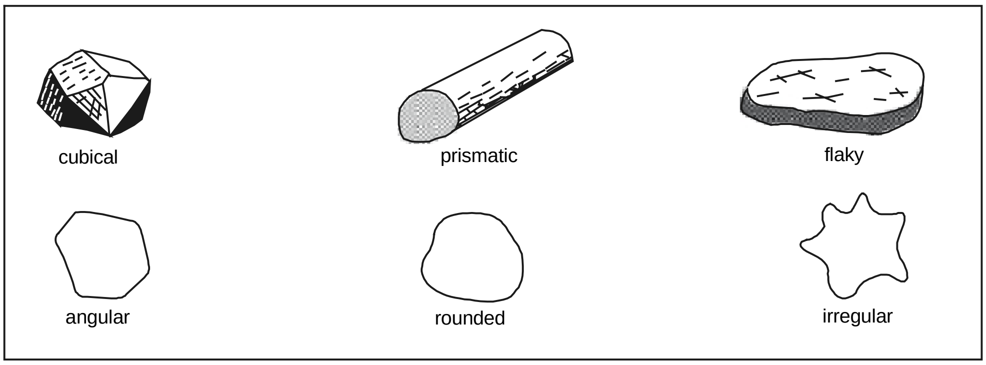


Figure 2.3 Shape and surface character of mineral grains

Some minerals have a stronger influence on the properties of a rock than others. In rock construction, mica and similar flaky minerals are particularly important where they occur as oriented in parallel, continuous layers. Mica schists and phyllites, therefore, have very distinct anisotropic properties of great significance for construction work as discussed in Section 2.5.

2.2 Classification and formation of rocks

Geologists use a system of classification that reflects the origin, formation and history of a rock rather than its potential mechanical characteristics. Thus, the rock names are defined and used according to the abundance, texture and types of the minerals involved, in addition to mode of formation, degree of metamorphism, etc. A chart showing the formation and development of rocks is shown in Figure 2.4, which also may help in identifying the various rock types.

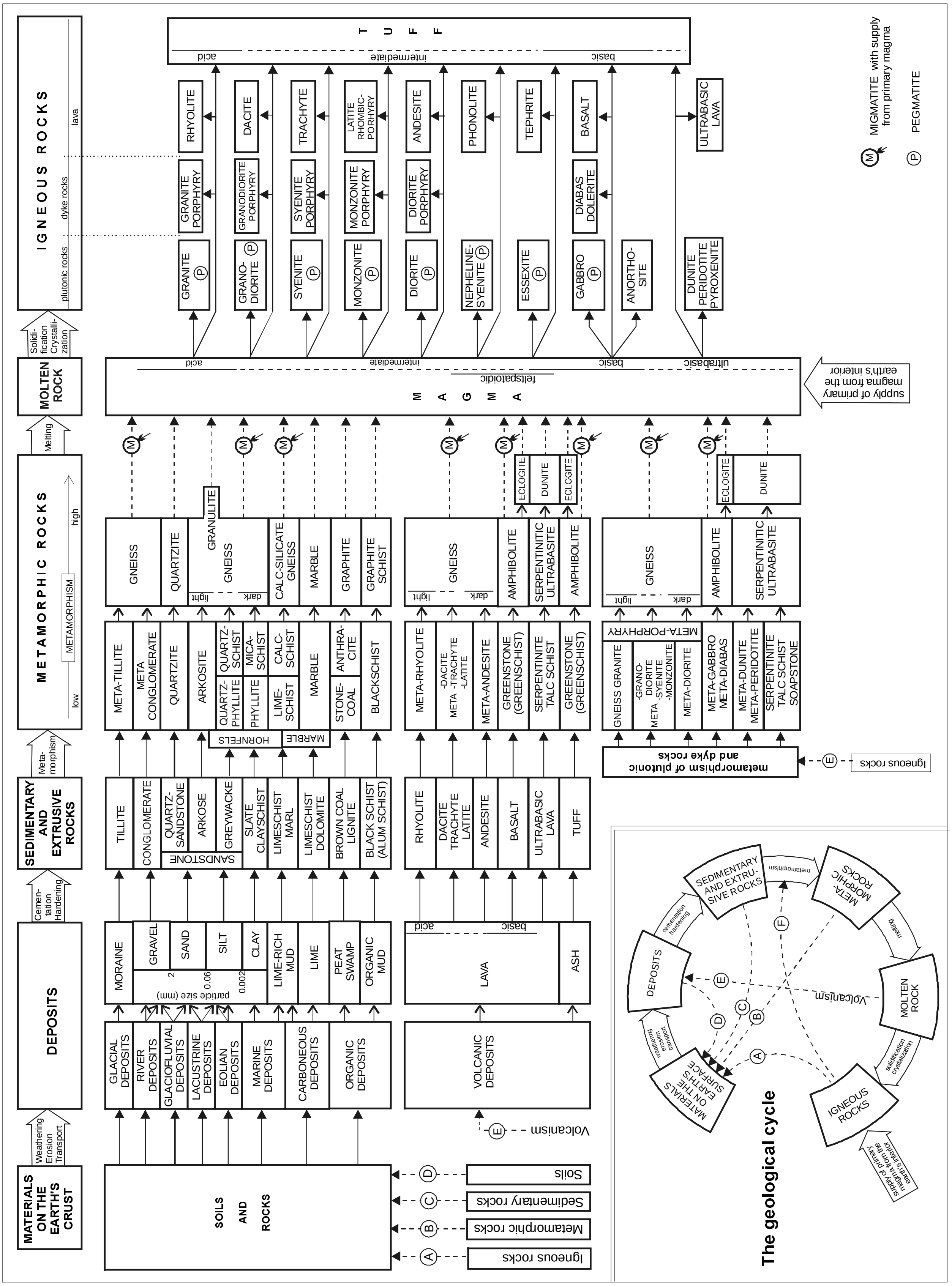


Figure 2.4 Formation and *development of rocks* related to the *geological cycle* (from Palmström, 1986).

2.2.1 Igneous rocks

Igneous rocks may be defined as those rocks, which have solidified from liquid melts, or magmas. These rocks are classified in two ways: 1) by their chemical composition and 2) by their grain size.

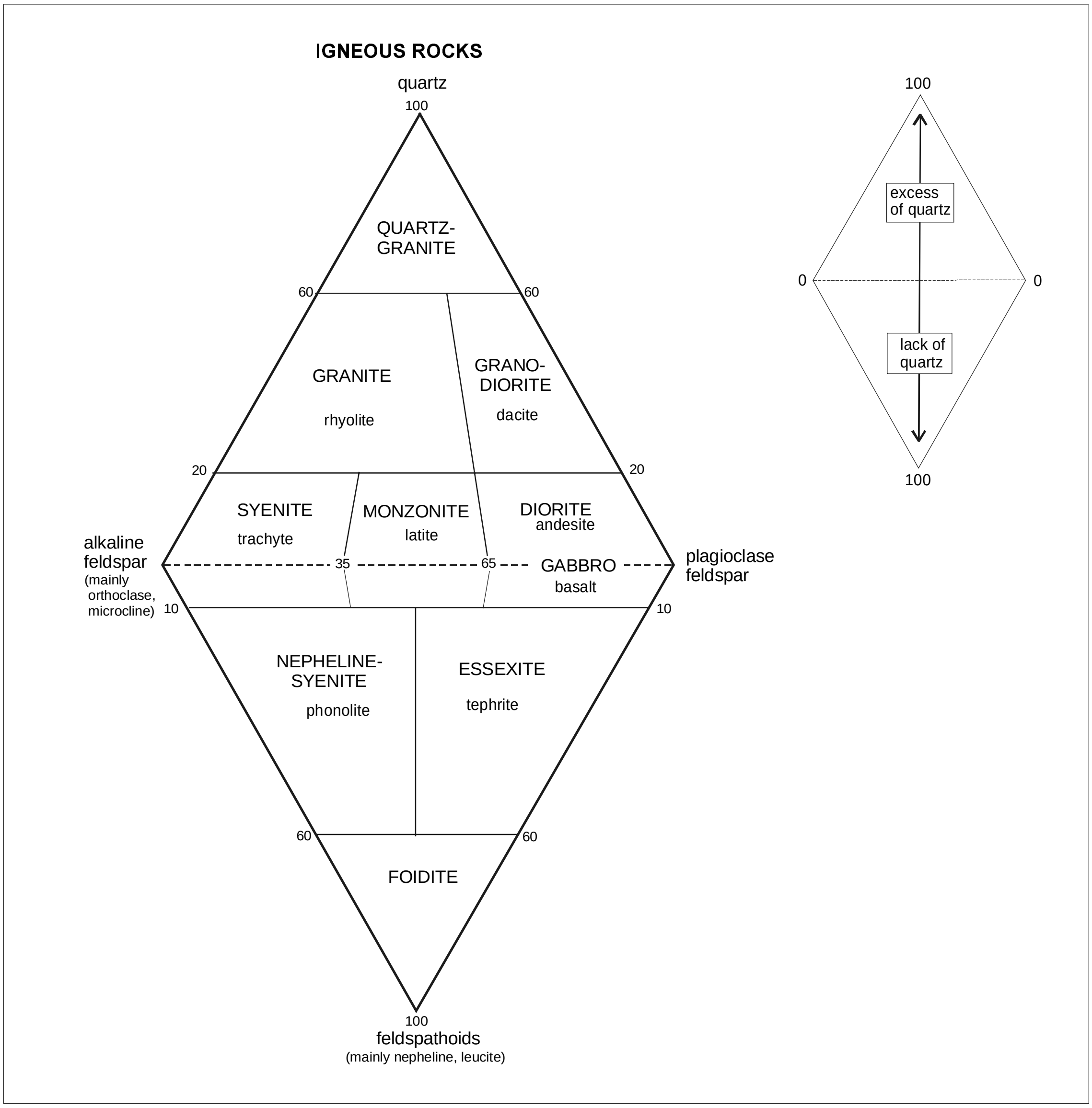


Figure 2.5 Geological classification of igneous rocks (simplified *Streckeisen*, from NBG, 1985). *Plutonic rocks* have been solidified slowly at great depth to coarse-grained rocks (in upper case). *Effusive rocks* have been formed by the rapid movement of magma to the earth's surface with quick solidification into fine-grained rocks (in lower case).

These rocks tend to be massive and generally have high strength values. Their minerals are of a dense, interfingering nature resulting in only slight, if any, directional differences in the mechanical properties of the rock. The chemical composition of igneous rocks varies considerably.

Acid rocks are igneous rock with a *silica* (SiO_2) content > 62%, i.e., rocks with visible quartz (such as granite, some granodiorites).

Intermediate rocks have a SiO_2 content of 54-62% (syenite, monzonite, diorite, nepheline syenite, some granodiorites).
Basic rocks have a SiO_2 content of 44-54% (gabbro, essexite).
Ultrabasic rocks have a SiO_2 content < 44% (dunite, peridotite, pyroxenite).

The magma may have intruded the previously-existing rocks in a number of different ways, giving rise to differently shaped igneous bodies, such as:

- *Dykes* and *sills:* sheet-like structures from centimetres to more than 100 m in thickness. They can be the result of injection of magma from the underground into overlying rock; a dyke cutting more or less vertically through the layers, a sill intruding roughly parallel to the layers. Often the heat from the liquid magma alters the surrounding rock for a short distance on either side of the contact.
- *Volcanic flows* and *ashes:* coming from an active volcano they may occur on land or on the sea floor. Flows of basic lavas are most common as they are less viscous.
- *Volcanic necks*: i.e., near-circular, vertical structures varying from about 100 m to over 1 km, where magma has passed through a once-active volcano. They may be filled with a variety of igneous rocks, often with a conglomerate or brecciated selection of many different rocks.
- *Batholiths:* much larger structures than necks, often elongate in plan and ranging from 1 to 2 km in length. They usually represent the once-molten roots of mountain chains that have moved upwards, first to crystallise and solidify, and finally to become exposed when the covering rock has eroded. Around the sides and in the roofs of batholiths (when these are exposed) a zone of contact metamorphism is usually seen in the *country rock*, with a thickness of metres or even kilometres for large intrusions.

2.2.2 Sedimentary rocks

Sedimentary rocks have the greatest variation in strength and behaviour of the three groups. Their main minerals are usually softer and their assemblage is generally weaker than is the case for igneous rocks. There are two main types of sedimentary rocks:

1. Those where the constituent particles have been ***transported to the place of deposition***, known as *clastic* or *detrital* rocks (conglomerates, sandstones, siltstones, mudstones and shales).
   Detrital rocks are classified according to their average grain size as follows:
   - *Rudaceous* (gravelly) rocks. The spaces between the rounded, coarse-grained particles are generally filled with sand or finer detrital material (as conglomerate), limestone, or a cement that commonly is siliceous or calcareous. Cemented angular and partly rounded rudaceous deposits are called *breccia*. In *volcanic breccia*, the fragments are predominantly of volcanic, rocks, set in a tuff matrix comprising finer-grained broken fragments of the same rock.
   - *Arenaceous* (sandy) rocks. Sands, gravelly sands and similar composite soil types become ***sandstone*** when they are *lithified*. Individual grains are mainly mineral fragments, of which quartz and feldspar grains usually predominate. Particles may be angular, subangular or rounded in shape. There are three common types of cement: silica (as quartz, chalcedony); iron oxides (limonite, haematite); and carbonates (calcite, siderite). Sandstones are named as follows:
     a) According to the nature of the cement: *siliceous sandstone*, *ferruginous sandstone*, *calcareous sandstone*.

b) According to the nature and proportions of minor constituents:
- *Argillaceous sandstone:* in which a finer matrix of compacted fine detrital minerals (clay) is present;
  - *Feldspathic sandstone:* sand grains mainly quartz, but there is an obvious feldspar component;
  - *Micaceous sandstone:* with mica flakes present throughout the rock, or mica flakes concentrated in closely spaced bedding planes in the sandstone variety called *flagstone*.

c) According to the nature and proportions of the constituent mineral or rock fragments:
- *Quartz sandstone* (*orthoquartzite*): sand fraction up to 95% quartz grains, voids are empty or cemented with silica.
- *Arkose*: sand fraction up to 75% quartz and at least 25% feldspar, scanty or absent fine detrital matrix, voids are empty or cemented.
- *Greywacke* has sand fraction up to 75% quartz, remainder rock and feldspar fragments in varying proportions, prominent (up to 15%) fine, detrital matrix; in addition the particles are frequently angular.

- *Argillaceous* (clayey) *rocks*. The term *mudstone* is used for lithified, homogeneous, argillaceous rocks. If the constituents are laminated or the rock is fissile on the bedding planes, the term *shale* is used. Clay mineral type can have an important effect on engineering properties, for example on swelling.
  Siltstone is an appropriate term for rock with more than 50% silt-sized, fine-grained detrital mineral particles. The corresponding term is *claystone*, for rock with more than 50% clay-sized detrital particles; although appropriate, it is not commonly used.

Clastic rocks formed from the deposition of material from subaqueous slides are called *turbidites*, and usually show a mixture of grain sizes; each turbidite bed having coarser particles at its base and finer particles at its top, termed a *graded bed.*

2. Those which have been ***formed from deposition***, either by an aggregation of organic matter or chemically/biochemically. These are:
   - *Limestones*[1] and *dolostones* or *dolomites* from mainly carbonate being remains of fossils. Limestone consists of at least 50% of calcium carbonate (calcite) grains. For lesser concentrations the qualifying term *calcareous* should be used (e.g., calcareous sandstone). Dolomite contains the mineral calcium magnesium carbonate (dolomite).
   - *Chalk* is an example of fine-grained limestone, being a white to light grey in colour and composed almost entirely of microscopic organic remains and shell debris.
   - *Chert*, including its variety *flint*, consisting of amorphous or cryptocrystalline silica (SiO_2) Chert occurs as bands or nodules within limestone sequences. The silica is originally deposited under seawater, and concentrated into bands or nodules during the rock-forming period subsequent to deposition.
   - *Marl* is used for clays or argillaceous rocks that have a significant carbonate content. The term marl is not recommended; the composite terms are preferred (e.g., calcareous mudstone).
   - *Evaporite*, formed from evaporation of enclosed or semi-enclosed basins where the various salts remain behind.

[1] Some limestones are detrital deposits

- *Coal*, which can be high quality *anthracite* or impure *lignite* or brown coal, the latter being brown to black and formed from peat under moderate pressure. Dry lignite contains about 60 to 75% carbon.
- Oil and natural gas.

In these rocks the minerals are not interlocking, but are cemented together with an intergranular matrix material by *diagenesis* (i.e., hardening of loose materials to rocks). Sedimentary rocks usually contain *bedding* and *lamination* or other sedimentation structures and, therefore, may exhibit significant anisotropy in physical properties depending upon the degree of their development. Of this group, argillaceous and arenaceous rocks are usually the most strongly anisotropic. Some of the rocks are not stable in the long term, as for example mudrocks, which are susceptible to *slaking* and *swelling*. This group of rocks therefore creates many problems and challenges in rock construction, see Section 2.7.3.

The degree of *consolidation* of a rock bears no relation to its geological age. For example, Cambrian rocks laid down 500 mill. years ago near St. Petersburg, Russia, are much softer and weaker than rocks of the same grain size laid down in the Alps less than 20 million years ago.

Beds or bedding layers of sedimentary rock are usually separated by a surface discontinuity. As a rule, the rock can readily be separated along these planes. The *bedding spacing* can be classified as follows (ISRM, 1981):

Description	**Spacing (m)**	**Description**	**Spacing (m)**
VERY THICKLY BEDDED	> 2	Thinly bedded	0.06 - 0.2
Thickly bedded	0.6 - 2	Very thinly bedded	0.02 - 0.06
Medium bedded	0.2 - 0.6	Laminated	< 0.02

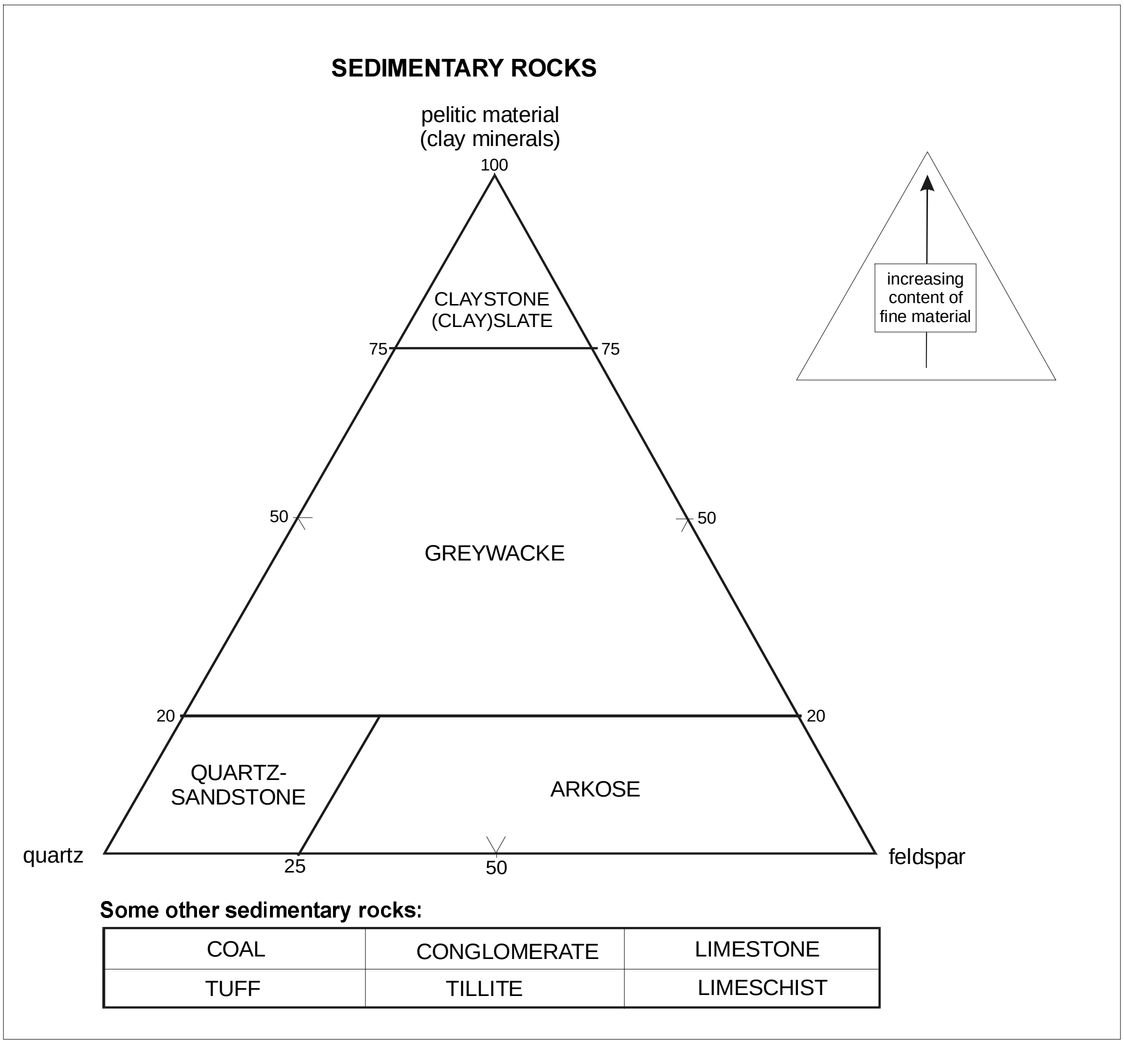
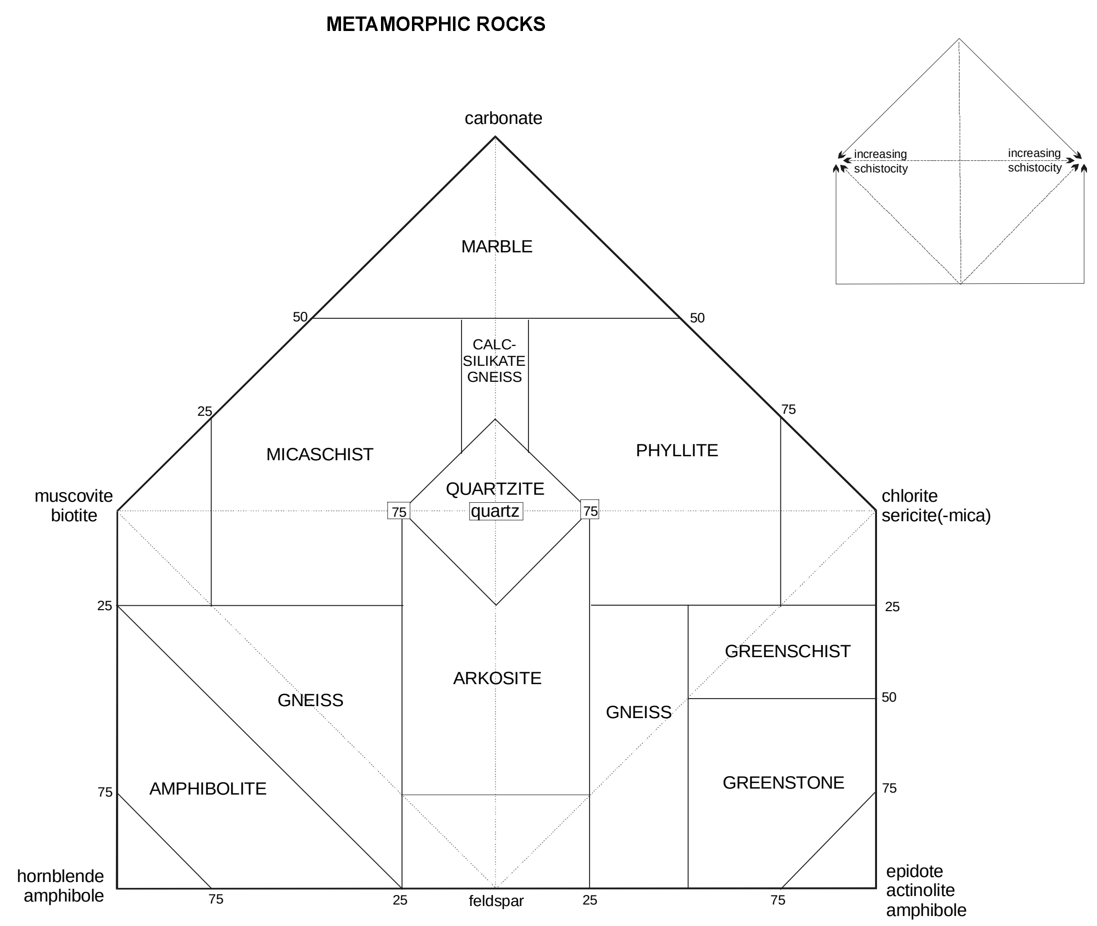


Figure 2.6 Geological classification of sedimentary rocks (simplified *Pettijohn,* from NBG, 1985).

2.2.3 Metamorphic rocks

When rocks of all types are subjected to very high temperature and pressure, they will melt and remobilize to form igneous magmas. However, since the melting point of each of the many constituents within a rock varies widely (and melting point increases with increase of pressure), some constituents will recrystallise and reform before others. Rocks, which have been noticeably altered, but still reflect some traces of their original structure, are termed *metamorphic rocks*. Metamorphism is often characterized by a change or transformation from one group of minerals to another group of minerals.

Contact metamorphism takes place near the contact of a magma penetrating the existing rocks, which are exposed to high temperatures (500-1000°C) but relatively low pressures (< 300 MPa). *Regional metamorphism* takes place at much larger scale, often in connection with mountain range development.



Some other metamorphic rocks:

ECLOGITE	SOAPSTONE	BRECCIA	MIGMATITE
SERPENTINITE	TALCSCHIST	MYLONITE	HORNFELS

Figure 2.7 Simplified classification of metamorphic rocks (from NBG, 1885)

A classical array of regional metamorphism is as follows (in order of increasing metamorphism):

- *Slate*, formed from shale by alteration of its constituent particles by a relatively small increase in pressure and temperature. This causes the rock to become hard and brittle, and split along directions often at angles unrelated to its original bedding planes. This property is called rock *cleavage*. Due to its well-developed slaty cleavage, slate is highly anisotropic.
- *Phyllite,* which is a rock intermediate between slate and schist, and with minerals usually not distinguishable by the naked eye. Further metamorphism brings the phyllite into *(mica) schist* during which there is an increase in mineral size of predominantly flaky or prismatic shape.
- *Schist*, i.e., finer-grained rocks which become completely recrystallised. Sometimes the schistosity is parallel to the bedding plane, at other times it lies oblique to the bedding planes, depending on the direction of the pressure at the time of recrystallization.
- *Gneiss*. This group of rocks is coarsely recrystallized and may be as coarse as igneous rocks. The gneisses are, however, distinguished by their banding or parallel orientation of minerals. *Paragneiss* results from the regional metamorphism of sedimentary rocks, *orthogneiss* from igneous rocks.

- *Migmatite* may be formed by injection of igneous material into schists, giving rise to a rock consisting of thin alternating layers or lenses of granite-type rock and schist. Rocks with gneissic texture are rarely strongly anisotropic.
- *Marble* is by definition a meta-limestone. The geological usage of “marble” is more restricted than the commercial use, which incorporates any kind of limestone for ornamental work under the term, whether metamorphosed or not.
- *Mylonite* has a cataclastic texture resulting from the mechanical breakdown of rock under stress. Bent and fractured mineral grains are in mechanical contact with one another. Thus, mylonite is a microbreccia, composed of angular particles welded together by very fine particles.

Metamorphic rocks show a great variety in structure and composition and properties. The metamorphism has often resulted in hard minerals and high intact rock strength; however, the preferred orientation of platy (sheet) minerals due to shearing movements results in considerable directional differences (*anisotropy*) in mechanical properties. Particularly the micaceous and chloritic schists are outstanding with respect to anisotropy.

2.2.4 Engineering classification of rocks

The geological classification of rocks, based mainly on formation and composition of the material, is so well established that other classification methods have not come into major use. An alternative, based on rock behaviour rather than composition has, however, been proposed by Goodman (1989), and is shown in Table 2.3.

Table 2.3 Behavioural classification of rocks (from Goodman, 1989)

Groups of rocks	Examples
I Crystalline texture	
A. Soluble carbonates and salts	- Limestone, dolomite, marble, rock salt, gypsum
B. Mica or other planar minerals in continuous bands..........	- Mica schist, chlorite schist, graphite schist
C. Banded silicate minerals without continuous mica sheets.	- Gneiss
D. Randomly oriented and distributed silicate minerals of uniform grain size	- Granite, diorite, gabbro, syenite
E. Randomly oriented and distributed silicate minerals in a background of very fine grain and with vugs	- Basalt, rhyolite, other volcanic rocks
F. Highly sheared rocks...	- Serpentinite, mylonite
II Clastic texture	
A. Stably cemented	- Silica-cemented sandstones and limonite sandstones
B. With slightly soluble cement...	- Calcite-cemented sandstone and conglomerate
C. With highly soluble cement..	- Gypsum-cemented sandstones and conglomerates
D. Incompletely or weakly cemented..................................	- Friable sandstone, tuff
Uncemented	- Clay-bound sandstones
III Very fine-grained rocks	
A. Isotropic, hard rocks	- Hornfels, some basalts
B. Anisotropic on a macro scale, but microscopically isotropic hard rocks..	- Cemented shales, flagstones
C. Microscopically anisotropic hard rocks	- Slate, phyllite
D. Soft, soil-like rocks	- Compaction shale, chalk, marl
IV Organic rocks	
A. Soft coal	Lignite and bituminous coal
B. Hard coal	
C. “Oil shale”	
E. Bituminous shale	
F. Tar sand	

2.3 Strength of rocks

The strength of rocks can be measured by various methods and procedures. Some of them are briefly presented below. For more details on execution of the tests, refer to the ISRM suggested methods listed in the Literature List and Section 17.1.1.

Many weak rocks may behave as soils, and conversely some very stiff or hard soils behave as rocks. The range of this overlap of performance is difficult to define.

2.3.1 The uniaxial compressive strength (σ_c)

The distinction between rock and soil is defined in terms of the uniaxial compressive strength and not in terms of structure, texture or weathering. ISRM (1978) considers a material with the strength ≤ 0.25 MPa as soil, see Table 2.4

The uniaxial compressive strength can be determined directly by uniaxial compressive strength tests in the laboratory, or indirectly from point-load strength test (see Section 2.3.3). The tests should be carried out according to the procedures recommended by the ISRM (1972).

Table 2.4 Classification of the *uniaxial compressive strength* of rocks (from ISRM, 1978).

Type	Classification	Uniaxial compressive strength σ_c (MPa)
Soil		< 0.25
ROCK	Extremely low strength Very low strength Low strength Medium strength High strength Very high strength Extremely high strength	0.25 – 1 1 – 5 5 – 25 25 – 50 50 – 100 100 – 250 > 250

ISRM defines 50 mm as reference diameter, and recommends that the uniaxial compressive strength of the rock material in an area be defined as the mean strength of rock samples, excluding faults, joints and other discontinuities where the rock may be excessively weathered. When the rock material is markedly anisotropic in its strength, it is usually of importance to record the uniaxial compressive strength in several directions.

A rock is a fabric of minerals and grains bound or welded together. The rock therefore includes microscopic cracks and fissures. Large samples are more likely to include all the components that influence strength. When the size of the sample is small, the failure is forced to involve a larger part of new crack growth than in a larger sample. Thus the strength is size dependent. Wagner (1987) has shown that the strength of large "samples" follows the equation:

$$\sigma_c = \sigma_{c50}\left(\frac{50}{D}\right)^{0.2}$$

where σ_{c50} = strength for specimens of 50 mm diameter, and
D = the diameter (in mm) of the actual sample.

For large samples (blocks):

$$\sigma_c \approx \sigma_{c50}$$

Many compressive strength tests are carried out on dry specimens. ISRM (1980) recommends that the samples should be tested at a water content pertinent to the problem to be solved. Because rocks are often much weaker in wet than dry condition, it is important to inform about the moisture conditions. This is further discussed in Section 2.3.7.

The uniaxial compressive test is time-consuming and is also restricted to those relatively hard, unbroken rocks that can be machined into regular specimens. Although the strength classification is based on laboratory tests, it can be approximated assessed by simple methods, such as point load test, Schmidt hammer, simple field hammer test, or from a full description of a rock including composition and possible anisotropy and weathering. These matters are briefly mentioned in the following sections.

2.3.2 The point load strength (I_s)

The principle of the *point load strength* test is that a piece of rock is loaded between two hardened steel points. Details on the measuring procedure are described by ISRM (1985).

The advantage of measuring the point load strength index (I_s) is that I_s can be determined in the field on specimens without preparation, using simple portable equipment. It does not require any preparation of the specimen. Any piece of rock, whether the surface is smooth or rough, can in principle be tested. The point load strength is defined as:

$$I_s = \frac{P}{D_e^{\,2}}$$

where P = measured load at failure (kN)
D_e = equivalent sample diameter (mm); for diametrical core tests D_e = core diameter; for axial core tests and block or lump tests $D_e = 4A/\pi$ in which A is minimum cross sectional area between the contact points.

Table 2.5 Classifications of the point load strength (I_s)

Term	Bieniawski (1984)	Deere (1966)
Very high strength	I_s > 8 MPa	I_s > 10 MPa
High strength	I_s = 4 - 8 MPa	I_s = 5 - 10 MPa
Medium strength	I_s = 2 - 4 MPa	I_s = 2.5 - 5 MPa
Low strength	I_s = 1 - 2 MPa	I_s = 1.25 - 2.5 MPa
Very low strength	I_s < 1 MPa	I_s < 1.25 MPa

Test results of $I_{s\,50}$ are related to standard 50 mm samples [see ISRM (1985) for a revised edition of the "Suggested Method for determining point load strength"]. Adjustment for tests where the sample diameter differs from the standard 50 mm is:

$$I_{s50} = F \times I_s$$

where $F = \left(\frac{D_e}{50}\right)^{0.45}$ or $F \approx \sqrt{\frac{D_{50}}{50}}$ when the sample diameter (D_{50}) is near the standard of 50 mm

The point load test is very suitable for determining rock strength anisotropy, particularly when a limited amount of test material is available. Most simply, this is done by using core samples drilled perpendicularly to the bedding or foliation, and applying the point load first diametrically, then axially. Highly anisotropic rocks may have a point load strength anisotropy index higher than 5.

The point load strength varies with the water content of the specimens, see Figure 2.8 and Section 2.3.7. ISRM (1985) mentions that the variations are particularly pronounced for water saturation below 25%. At water saturation above 50% the strength is less influenced by small changes in water content. Therefore, tests in this water content range are recommended unless tests on dry rock are specifically required.

The point load strength test is a form of "indirect tensile" test. I_{s50} is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

2.3.3 The correlation between I_s and σ_c

Point load strength test may often replace the uniaxial compressive strength test as it is, when properly conducted, as reliable and much quicker to measure. The correlation with uniaxial compressive strength is given as

$$\sigma_c = k \times I_s \qquad \text{or} \qquad \sigma_c = k_{50} \times I_{s50} \quad \text{(related to 50 mm diameter samples)}$$

It has been found that the factor k or k_{50} varies with the strength of the rock. It is higher for strong rocks than for weak rocks. Based on this it is suggested that, when no other information is available, the values of k_{50} presented in Table 2.6 be used in correlation from point load strength to uniaxial compressive strength.

Table 2.6 Suggested value of the factor k_{50} .

Compressive strength σ_c (MPa)	**Point load strength** I_{s50} (MPa)	**Suggested value of** K_{50}
25*) - 50	1.8 - 3.5	14
50 - 100	3.5 – 6	16
100 - 200	6 – 10	20
> 200	> 10	25

*) Bieniawski (1973) suggests that the point load strength test not be used on rocks having σ_c < approx. 25 MPa.

2.3.4 Compressive strength estimated from the Schmidt hammer rebound number

The *Schmidt hammer* is used for non-destructive testing of the quality of rocks and concrete. It measures the "rebound hardness" of the tested material. The mechanism of operation is simple; a plunger, released by a spring, impacts against the tested rock surface. The rebound distance of the plunger is read directly from a numerical scale.

Schmidt hammer models are designed in different levels of impact energy, and the two types L and N are commonly adapted for rock property determinations. The type L has an impact energy of 0.735 Nm, 1/3 of that of Type N.

ISRM (1978) suggests that the L hammer can be used for the testing of rocks that have a uniaxial compressive strength in the range of approximately 20-150 MPa. ISRM (1978) also gives a complete test procedure including a chart for correlating Schmidt rebound hardness to uniaxial compressive strength.

The Schmidt hammer rebound test may in some cases give questionable results, and should be used with care.

2.3.5 Rock strength estimated from simple field test

Sometimes, particularly at an early stage in the description of the rock mass, strength may be assessed without testing. Such a first estimate of the uniaxial compressive strength (σ_c) may be made by visual and sensory description of hardness of rock or consistency of a soil. The strength can be judged from simple hardness tests in the field with a geological hammer and by observing the resistance to breaking under impact, as shown in Table 2.7.

The hammer test should be made with a geologist's hammer on pieces about 10 cm thick placed on a hard surface, and tests with the hand should be made on pieces about 4 cm thick. The pieces must not have incipient fractures, and therefore several should always be tested.

For anisotropic rocks tests should be carried out in different directions to the structure. The lowest representative values should be applied in evaluation/analysis. It is also possible to use the foliation anisotropy factor described in Section 2.5 to find the lowest value of compressive strength.

Table 2.7 Simple *field identification* of compressive strength of rocks and clays (from ISRM, 1978).

Grade and Term		Field identification	Approx. σ_c (MPa)
S1	Very soft clay	- Easily penetrated several inches by fist.	< 0.025
S2	Soft clay	- Easily penetrated several inches by thumb.	0.025-0.05
S3	Firm clay	- Can be penetrated several inches by thumb with moderate effort.	0.05 -0.10
S4	Stiff clay	- Readily indented by thumb, but penetrated only with great effort.	0.10 -0.25
S5	Very stiff clay	- Readily indented by thumbnail.	0.25 -0.50
S6	Hard clay	- Indented with difficulty by thumbnail.	> 0.50
The clays in grade S1 - S6 can be silty clays and combinations of silts and clays with sands, generally slow draining.			
R0	Extremely weak rock	- Indented by thumbnail.	0.25 - 1
R1	Very weak rock	- Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife.	1 - 5
R2	Weak rock	- Can be peeled by a pocket knife with difficulty, shallow identifications made by firm blow with point of geological hammer	5 - 25
R3	Medium strong rock	- Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong rock	- Specimen requires more than one blow of geological hammer to fracture it.	50 - 100
R5	Very strong rock	- Specimen requires many blows of geological hammer to fracture it	100 – 250
R6	Extremely strong rock	- Specimen can only be chipped with geological hammer	> 250

2.3.6 Compressive strength estimated from rock description

The correlation between the petrographic names of rocks and their mechanical properties often is poor, caused by difference in composition, grain size, porosity, cementation, anisotropy within each type, etc. Nevertheless, a great deal of associated information about a rock can be inferred from its geological name, such as whether it may be homogeneous, layered or schistose. Used with care, and if very accurate values are not required, an estimate of compressive strength from the rock name based on Table 2.8 may be useful.

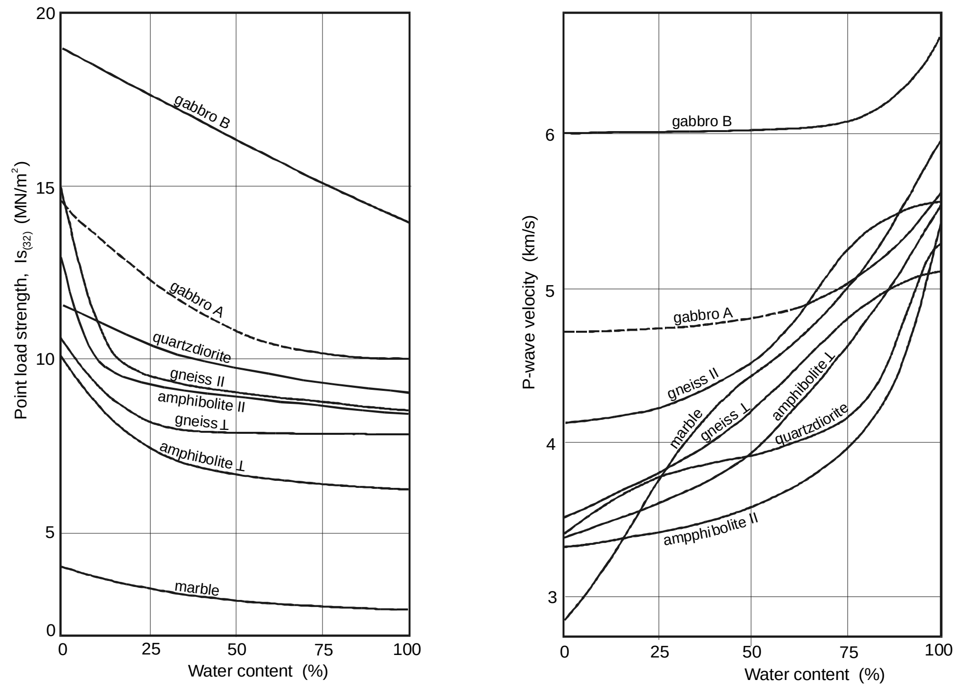


Figure 2.8 Effect of moisture on strength and *sonic velocities* for some rocks (from Broch, 1979).

2.3.7 Effect of saturation upon strength

The influence of water on the strength of rocks is considerable. Moisture may strongly reduce the strength of rocks, up to about 40% and 60% of the dry strength in sandstones and shales respectively. Some examples of strength reduction due to water saturation are shown in Figure 2.8.

It is important that the conditions under which the rocks are tested, are reported. ISRM (1972, 1981) suggests that rock samples are stored in 50% humidity for 5-6 days before testing.

2.4 Alteration and weathering of rocks

Rocks are frequently *weathered* near the earth's surface, and are sometimes *altered* by hydrothermal processes. Both processes generally first affect the walls of discontinuities. The main results of rock weathering and alteration are:

- Mechanical *disintegration* or breakdown by which the rock loses its coherence. The process has little effect upon the change in the composition of the rock material, but results in:
- Opening up of joints.
- Formation of new joints, the opening up of grain boundaries.
- Fracturing of individual mineral grains.

Table 2.8 Uniaxial compressive strength, E-modulus and the factor m_i in the Hoek Brown failure criterion for rock masses for some typical rocks. Division of rocks according to Goodman (1989), see Table 2.3

Average values from tests of intact rock samples		**Tests of rocks world-wide***				**Scandinavian rocks tested at NTNU / SINTEF *****				Rating of the factor m_i **
		σ_c	E	E / σ_c	Number of tests	σ_c	E	E / σ_c	Number of tests	
ROCK		MPa	GPa			MPa	GPa			
Crystalline texture	Dolomite	86	38	443	8	110	49	443	2	10.1
	Limestone	107	47	441	81	74	71	961	25	8.4
	Marble	133	63	474	20	66	71	1074	4	9.3
	Greenschist	-	-	-	-	93	44	472	3	
	Clay schist / -stone	68	38	563	2	40	21	537	6	
	Micaschist	104	39	374	16	71	30	422	21	15?
	Gneiss	130	53	406	27	130	50	385	107	29.2
	Micagneiss	-	-	-	-	89	29	330	5	30?
	Granitic gneiss	-	-	-	-	89	29	330	5	30?
	Granulite	90	41	451	4	-	-	-	-	
	Amphibolite	212	101	474	7	107	70	660	16	31.2
	Greenstone	281	101	359	1	105	53	503	7	20?
	Quartzite	209	58	276	28	172	56	328	7	23.7
	Anorthosite	228	90	395	2	157	86	545	2	
	Diorite	173	64	368	6	130	52	403	6	27?
	Gneissgranite	-	-	-	-	117	42	354	5	30?
	Granite	154	48	313	71	169	42	250	20	32.7
	Granodiorite	160	51	319	2	171	20	118	2	30?
	Gabbro	228	106	466	5	248	76	306	1	25.8
	Norite	229	82	356	8	-	-	-	-	21.7
	Dunite	-	-	-	-	87	113	1307	5	
	Peridotite	197	55	280	1	109	164	1502	1	
	Monzonite	110	28	256	8	106	61	580	4	30?
	Andesite	152	31	206	6	-	-	-	-	18.9
	Basalt	145	50	347	25	207	82	395	3	(17)
	Diabase, dolerite	229	88	384	13	152	81	537	5	15.2
	Hyperite	-	-	-	-	245	108	441	2	
Clastic texture	Graywacke	81	25	310	12	-	-	-	-	
	Sandstone	109	28	257	95	147	28	189	5	18.8
	Siltstone	89	31	350	14	-	-	-	-	9.6
Very fine-grained rocks	Hornfels	111	74	668	3	-	-	-	-	
	Claystone	5	2	301	2	-	-	-	-	3.4
	Phyllite	39	26	672	4	61	46	756	12	13?
	Chalk	1	2	1606	2	-	-	-	-	
	Marl, marlstone	17	2	133	9	-	-	-	-	
	Mudstone	11	1	106	4	-	-	-	-	
Organic rocks - coal		30	3	107	14	-	-	-	-	
				Sum of tests =	500			Sum of tests =	281	

Soil materials (from ISRM (1978), see Table 2.7):

Very soft clay	σ_c = 0.025 MPa	Stiff clay	σ_c = 0.1 – 0.25 MPa
Soft clay	σ_c = 0.025 – 0.05 MPa	Very stiff clay	σ_c = 0.25 – 0.5 MPa
Firm clay	σ_c = 0.05 – 0.1 MPa	Hard clay	σ_c = 0.5 – 1MPa

Silt, sand (assumed) σ_c = 0.0001 – 0.001 MPa

* Test results mainly from "Handbook of physical properties of rocks and minerals" by R.S. Carmichael (1989); some test results also from Lama and Vutukuri (1978).

** m_i is the factor for intact rock in the Hoek Brown failure criterion for rock masses. Values in brackets have been estimated by Hoek et al. (1992); values with question mark are assumed by Palmström (1995).

*** NTNU = Norwegian University of Science and Technology
SINTEF = The Foundation of Scientific and Industrial Research at NTNU

Chemical *decomposition*, which involves rock decay accompanied by marked changes in chemical and mineralogical composition, results in:

- Discoloration of the rock.
- Decomposition of complex silicate minerals (feldspar, amphibole, pyroxene, etc.) eventually producing clay minerals. Some minerals, notably quartz, resist this action and may "survive" unchanged.
- *Leaching* or solution of calcite, anhydrite and salt minerals.

The disintegration leads mainly to a greater number of joints in rock masses found in the upper zone of weathering, while decomposition influences the joint condition as well as the rock material.

Leaching is the combined result of chemical weathering and the removal of the soluble constituents from the action of rainwater.

Table 2.9 Engineering classification of the weathering of rocks (from ISRM, 1978) and suggested alteration factor (f_W) (based on Palmström, 1995).

Class	Term	Description	Rating (f_W)
I	**Unweathered**	No visible signs of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	1
II	**Slightly weathered**	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.	1.75
III	**Moderately weathered**	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present, either as a discontinuous framework or as corestones.	2.5
IV	**Highly weathered**	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present, either as a discontinuous framework or as corestones.	10
V	**Completely weathered**	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.	
IV	**Residual soil**	All rock material is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	

Note: The suggested ratings of the factor f_W *are based on few data and should therefore be considered very approximate, especially for high grades of weathering.*

In general, the degree of weathering is usually estimated from visual observations. Table 2.9 shows classification of weathering/alteration. A more precise characterisation of alteration and weathering of the rock can be found from analysis of thin sections in a microscope.

Table 2.9 also shows an *alteration reduction factor* (f_W) introduced by Palmström (1995) as a means to estimate the strength of weathered or altered rocks:

$$I_{s50} = \frac{I_{s50\,fresh}}{f_w}$$

Assuming a similar reduction, the compressive strength can be found from:

$$\sigma_c = \frac{\sigma_{c\ fresh}}{f_w} = k_{50} \times \frac{I_{s50\ fresh}}{f_w}$$

2.5 Rock anisotropy and reduction of strength

Anisotropy in rock materials is caused by schistosity, foliation, layering or bedding. The degree of anisotropy is determined by the arrangement and amount of certain *elastic and anisotropic minerals* such as mica, chlorite, amphiboles, and some pyroxenes. Parallel orientation of these flaky and prismatic minerals is often found in sedimentary and regional metamorphic rocks in which weakness planes may occur along their layers. Where mica and chlorite occur in continuous layers, their effect on rock behaviour is strongly increased. Thus, mica schists and phyllites have strong anisotropic mechanical properties. Also other sheet minerals such as serpentine, talc, and graphite reduce the strength of rocks due to easy sliding along the cleavage surfaces, see Section 2.7.1.

The effect of anisotropy upon strength is roughly given in the *anisotropy factor* (f_A) in Table 2.10. It is based on tests performed on slates, schists, and gneisses with compressive strength ranging between 20 and 285 MPa. The minimum compressive strength of the foliated rock can roughly be assessed as:

$$\sigma_{c\ min} = \frac{\sigma_{c\ max}}{f_A}$$

As an alternative to the anisotropy factor f_A, which is considered by some to be approximate and often unreliable, point load testing can be used for finding the degree of anisotropy, see Section 2.3.2.

Table 2.10 Classification of anisotropy of rocks and proposed rating (from Palmström, 1995 and data published by Tsidzi, 1990).

Classification	**Description** *Typical rocks*	**Rating of the foliation anisotropy factor (f_A)**
Isotropic rock	< 10% flaky and prismatic minerals, which may occur as discontinuous streaks or may be randomly oriented. *Igneous rocks and many high-grade regional metamorphic and contact metamorphic rocks (quartzite, hornfels, granulite, etc.).*	1 - 1.2
Fairly anisotropic rock	10-20% flaky and prismatic minerals, showing mechanically insignificant layering. *High-grade regional metamorphic rocks (quartzo-feltspatic gneiss, mylonite, migmatite, etc.)*	1.2 - 1.5
Moderately anisotropic	20-40% flaky and prismatic minerals, showing thin to thick folia, occasionally discontinuous. Mechanically the foliation has little effect. *Rocks formed by medium to high-grade regional metamorphism (schistose gneiss, quartzose schist, etc.).*	1.5 - 2
Highly anisotropic	40-60% flaky and prismatic minerals occurring as thin wavy continuous folia which are mechanically significant. *Medium-grade regional metamorphic rocks (mica schist, mica gneiss, hornblende schist, etc.)*	2 - 2.5
Very highly anisotropic	> 60% flaky and prismatic minerals which occur as very thin, continuous folia. Foliation is perfect and mechanically significant. *Rocks formed by dynamic or low-grade regional metamorphism (slate, small folded phyllite).*	> 2.5

2.6 Deformation moduli and Poisson's ratio

In elastic deformation, there are various constants that relate the magnitude of the strain response to the applied stress. These elastic constants include the following:

Young's modulus (E) also often referred to as the *modulus of elasticity* or *E-modulus*, is the gradient of the stress-strain curve, in most cases related to uniaxial testing:

$$E = \frac{\sigma}{\varepsilon} \text{ (Hooke's law)}$$

where σ = applied stress (MPa), ε = strain (ms)

Poisson's ratio (ν) is the ratio of the lateral strain (perpendicular to an applied stress) to the axial strain.

As the stress-strain curve generally in very few cases is perfectly linear, the axial load has to be defined for the E and ν-values to be unambiguous. In most cases the E-value is defined as the secant modulus for axial stress 20 MPa.

Shear modulus (G) is the ratio of the applied stress to the distortion (rotation) of a plane originally perpendicular to the applied shear stress; it is also termed the modulus of rigidity.

Bulk modulus (K) is the ratio of the confining pressure to the fractional reduction of volume in response to the applied hydrostatic pressure. The volume strain is the change in volume of the sample divided by the original volume. Bulk modulus is also termed the modulus of incompressibility.

For elastic and isotropic materials, the elastic constants are interrelated. For example:

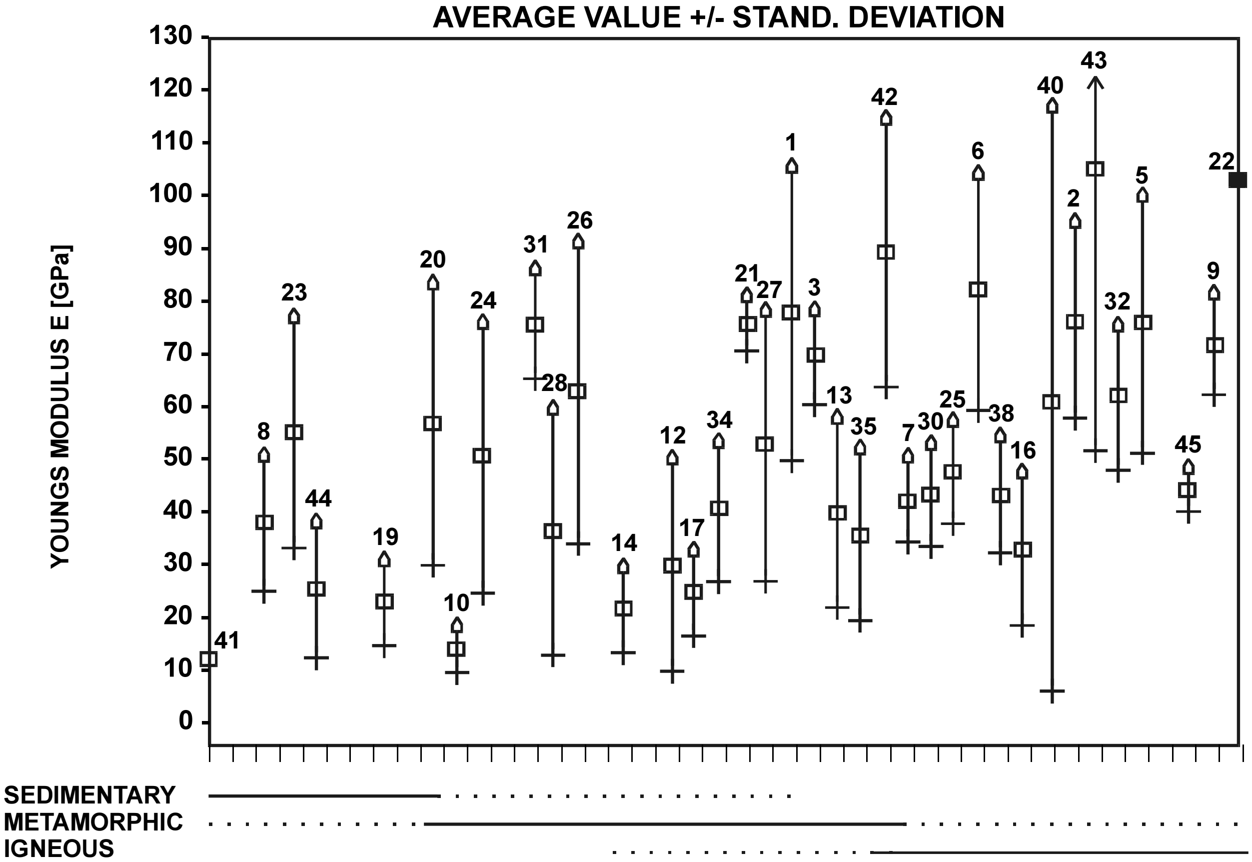
$$\nu = \frac{E}{2G} - 1 \qquad E = 3K\,(1 - 2\nu) \qquad G = \frac{3}{2}K\frac{1-2\nu}{1+\nu} \qquad G = E/2(1 + \nu)$$

Young's modulus for a variety of different rock types are shown in Figure 2.9 (secant values at 20 MPa, based on uniaxial testing). The effect of confining pressure on deformation modulus is illustrated in Table 2.11.

Table 2.11 Some elastic constants (in GPa) with rock type and confining pressure (from Encyclopaedia Britannica, 1998)

Rock Type	At pressure = 0.1 MPa				At pressure = 300 MPa			
	Bulk Modulus	**Young's modulus**	**shear modulus**	**Poisson's ratio**	**bulk modulus**	**Young's modulus**	**shear modulus**	**Poisson's ratio**
Granite	10	30	20	0.05	50	60	40	0.25
Gabbro	30	90	60	0.1	90	80	50	0.2
Dunite	110	150	50	0.3	120	170	70	0.27
Obsidian	40	70	30	0.08	80	120	40	0.25
Basalt	50	80	30	0.23	50	70	30	
Gneiss	10	20	10	0.05	80	70	30	0.3
Marble	10	40	20	0.1				
Quartzite	50	100	40	0.07				
Sandstone	7	20	8	0.1				
Shale	4	10	5	0.04				
Limestone	80	60	20	0.3				

The elasticity modulus for steel is about E = 200 GPa, for concrete E = 25 GPa. Poisson's ratio for steel is about $\nu = 0.28$, for aluminium alloys $\nu = 0.33$.



1. Amphibolite	13. Gneiss	25. quartzite	37. black shale
2. Amphibolitic gneiss	14. Granitic gneiss	26. quartzitic phyllite	38. syenite
3. Anorthosite	15. garnet micaschist	27. quartz sandstone	39. talc schist
4. Arkose	16. granite	28. mudstone	40. augen-gneiss
5. Basalt	17. granitic gneiss	29. marble	41. coral chalk
6. Diabase	18. granodiorite	30. monzonite	42. magnetic ore
7. Diorite	19. greenschist	31. mylonite	43. sulphide ore
8. Phyllite	20. greenstone	32. nepheline syenite	44. zinc ore
9. Gabbro	21. greywacke	33. pegmatite	45. lead ore
10. Mica gneiss	22. hyperite	34. sandstone	
11. Mica quartzite	23. limestone	35. serpentinite	
12. Mica schist	24. conglomerate	36. sparagmite	

Figure 2.9 Variation in the *modulus of elasticity* for some Scandinavian rocks (from Hanssen, 1988).

2.7 Some special features of rocks

For each type of rock, the mechanical properties vary considerably, even within the same rock name. Petrological data can, however, help in predicting mechanical performance, provided that one looks beyond the rock names to the observations on which they are based, and particularly if additional information on anisotropy and weathering is available. For various engineering purposes it is, therefore, still important to retain the names for the different rock types.

2.7.1 The significance of some minerals

The effects on strength of *elastic, flaky and prismatic minerals* (mica, chlorite, amphiboles) in schists, phyllites, amphibolites are discussed in Section 2.5. The same group of minerals also has significant impact on excavation works such as drilling and blasting, see Section 2.7.4.

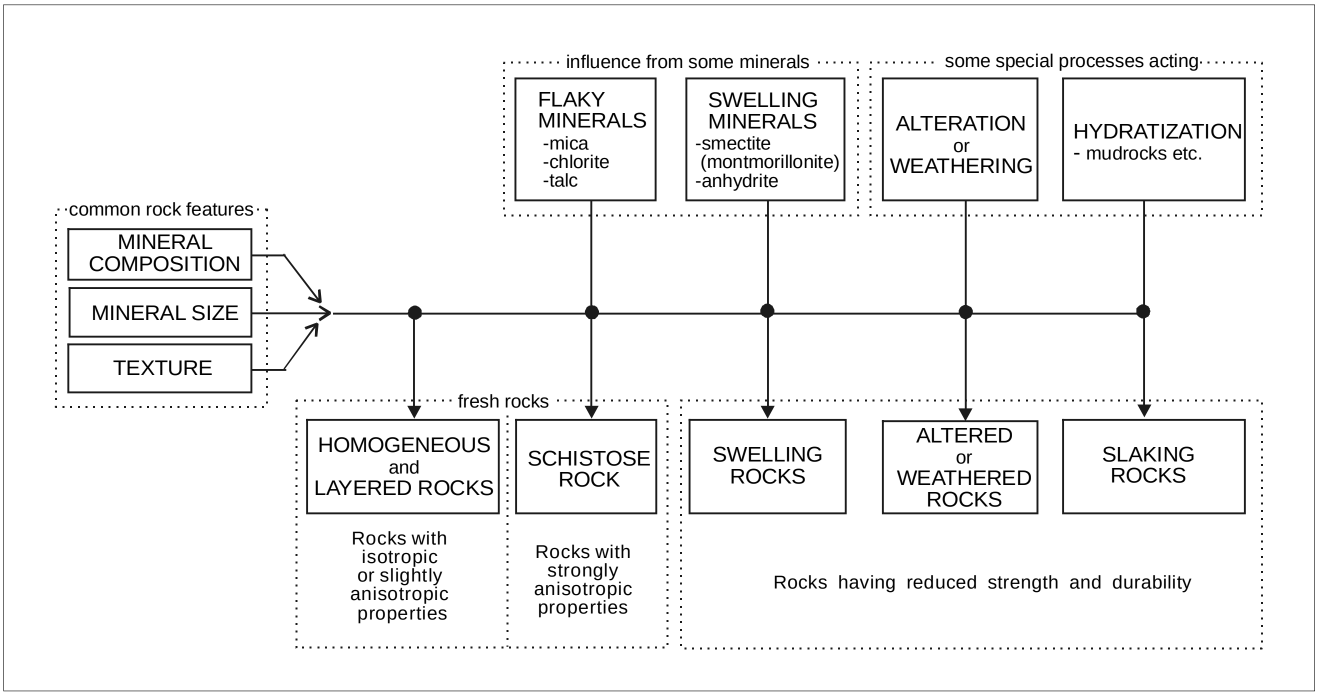


Figure 2.10 The main variables influencing rock properties and behaviour (from Palmström, 1995)

Quartz is another important mineral in rock construction. This mineral has grade 7 in the Moh's scale of hardness, see Table 2.1. Sharp, obtuse-angled edges of the quartz grains are very unfavourable regarding drill bit and cutter wear, while the effect from rounded quartz grains is significantly less, see Section 2.7.4.1.

Swelling minerals can cause significant stability problems with high swelling pressures from water absorption. The swelling clay minerals of the smectite (montmorillonite) group, occurring either as infillings or alteration products in seams or faults, have in addition to expansive properties a low shear strength. This may contribute to rock falls and slides. Among swelling rocks are *montmorillonite*-containing shales, altered or weathered basalts, or other igneous, metamorphic rocks, and also sedimentary rocks containing *anhydrite*. See Section 2.7.3

Some minerals may dissolve or disintegrate, such as *carbonatic rocks* (limestone, marble) and rock salt. Especially joints in carbonate rocks with loose, flaky calcite fillings are subjected to chemical dissolution by water. This may lead to reduced stability in underground openings, and increasing water leakage. See also Section 6.3.1.

2.7.2 Durability of rocks

Durability is the resistance of a rock against slaking or disintegration when exposed to weathering processes. Some rocks may *slake* (hydrate or "swell", oxidise), disintegrate or otherwise weather in response to changes in humidity and temperature consequent to excavation. The abundant group of mudrocks are particularly susceptible to slaking, see Figure 2.11. As also shown, the slake durability may also vary considerably within some rock groups. The slaking process may greatly change the mechanical properties of the rock, and hence dramatically influence the stability.

The slake durability test is intended to assess the resistance of the rock material to weakening and disintegration when subjected to cycles of wetting and drying. The method is described in ISRM (1972). The slake-durability index is defined by: I_{d2} = final weight/original weight expressed as a percentage.

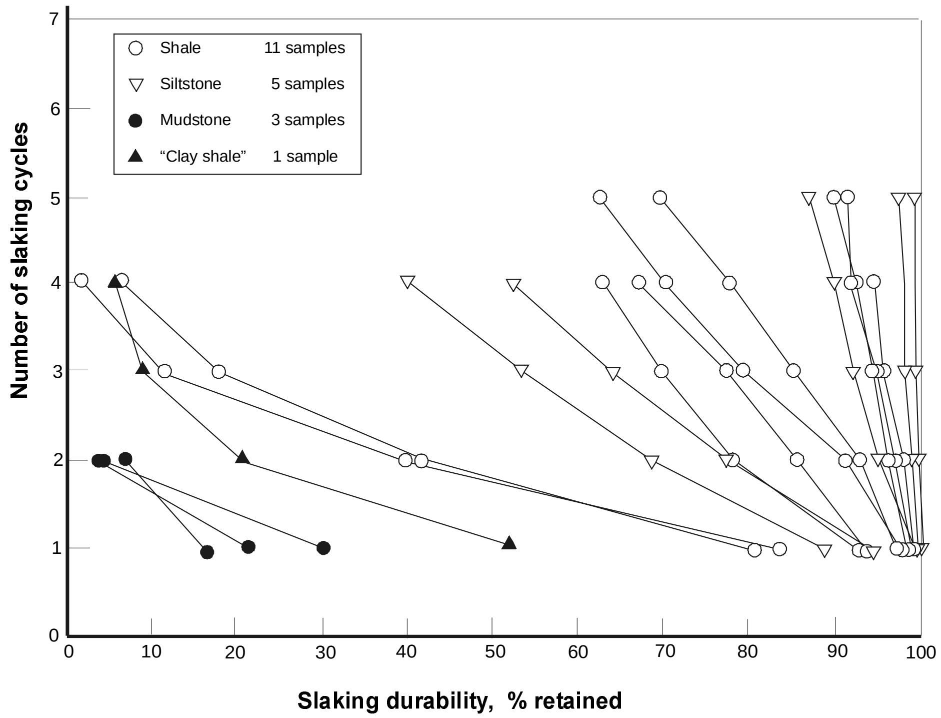


Figure 2.11 Influence of the number of slaking cycles on slake-durability (ISRM, 1972).

Table 2.12 ISRM suggested classification for two slaking cycles.

Term for slaking	Slake durability index (I_{d2})
Very low	0-30 %
Low	30-60 %
Medium	60-85 %
Medium high	85-95 %
High	95-98 %
Very high	98-100 %

2.7.3 Swelling and deterioration of rocks

A special property of rocks and soils is the expansion of certain materials upon access to water. The swelling pressure will exert extra loads on the support. *Rock swelling* can be caused by the following minerals:

1. Smectite (montmorillonite, vermiculite, etc.).
2. Anhydrite.
3. Some pyrrhotite-containing schists and shales.

Smectite is a group of minerals where montmorillonites, vermiculites and mixed-layer *swelling* minerals are most common. They are secondary minerals formed from alteration, either of in situ, settled, rock-forming minerals in shales, or of solution deposits. These very small sheet minerals are different from other sheet minerals (such as mica and chlorite) in their ability to take up and release water in accordance with the external pressure. Swelling clays are dealt with in Section 3.5.3.1

Anhydrite ($CaSO_4$) is a major constituent of sedimentary and metamorphic rocks of evaporitic origin. Hydration of anhydrite produces gypsum with a theoretical, maximum volume increase of about 60% if anhydrite is exposed to water in an open system, i.e., if water is continuously supplied. However, in tunnels with usual temperatures, anhydrite will not directly be transformed into gypsum. It is first dissolved and then re-precipitates as gypsum, usually taking on a thin coating of white gypsum. This coating will often limit the access of water, thus reducing the volume increase somewhat.

Certain types of *pyrrhotite* oxidise when exposed to atmosphere or water rich in oxygen. In such cases, buildings and underground openings can be severely damaged from expanding bedrock and the disintegration of concrete foundations and linings. See also Section 6.3.1.

The term pyrrhotite is used as a collective denomination for several varieties of iron sulphide with compositions ranging from Fe_6S_7 through $Fe_{11}S_{12}$, the latter being very near to pure FeS. Pyrrhotite occurs in certain petrographic environments and its possible occurrence may be predicted from the geology of a region. For example pyrrhotite is commonly found in certain Cambrian sedimentary strata (the so-called alum shale) and in certain Archean schists and gneisses in Scandinavia.

The *swelling* of certain *argillaceous rocks* can be caused by the content of one or more of the minerals listed above, and in addition mechanical swelling from stress relief leading to increased inflow of pore water.

ISRM (1983) defines *swelling* as "time dependent volume increase, involving physio-chemical reaction with water". Often it can be difficult to prove whether an invert heave or wall movement in a tunnel in argillaceous rock is caused by swelling or squeezing, as both processes may act simultaneously.

2.7.4 Drillability and blastability

For conventional excavation (drill and blast), costs related to drilling and blasting often represent the major part of the total costs. In mechanical excavation, boreability is the key factor. The properties governing *drillability*/boreability and *blastability* vary considerably from rock type to rock type, and also may vary considerably even within a certain rock type. In the planning process it is therefore important to collect information based on sampling and testing in each individual case.

2.7.4.1 Drillability

Drilling rate is governed not only by the capacity of the equipment, but also, to a great extent, by the structure of the rock mass. For hydraulic drilling in Scandinavian hard rocks, a variation of drilling rates between 90 cm/min and more than 200 cm/min has been measured (in 45 mm diameter holes). For bit wear, an even greater variation caused by rock properties has been recorded. Depending on the mechanical character of the rock, and in particular the quartz content, the lifetime of drill bits may vary between 150 and 700 metres bored.

In Norway, a method based on laboratory testing has been used extensively to evaluate the drillability of rocks by percussive drilling since the 1960s. The evaluation is based on the following three tests:

- The *brittleness test*
- The *Siever's J-value* test
- The abrasion test

The basic principles of these tests are shown in Figures 2.12 to 2.14.

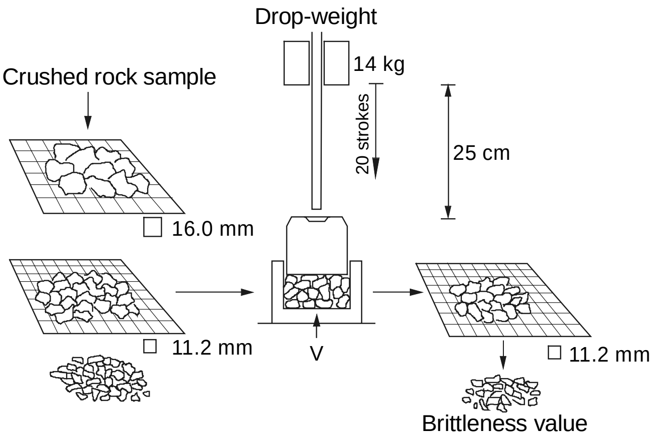


Figure 2.12 The brittleness test. The brittleness value (S_{20}) is the percentage of the material that passes the 11.2 mm sieve after 20 drops of the 14 kg piston.

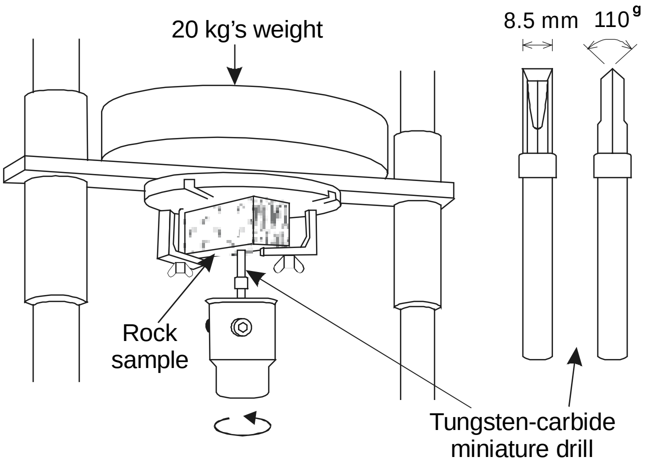


Figure 2.13 The Siever's miniature drill test. The Siever's J-value (SJ) is the penetration measured in 1/10 mm after 200 rotations.

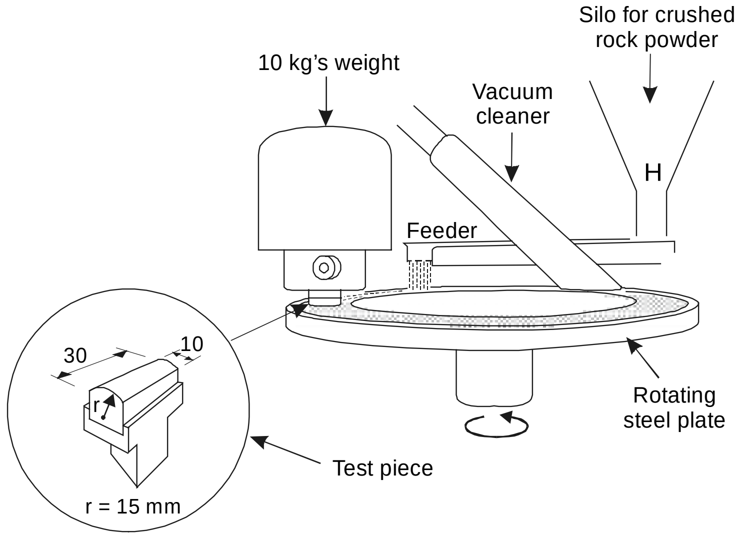


Figure 2.14 The *abrasion test*. The abrasion value (AV) is measured as the weight loss in milligrams of the test specimen.

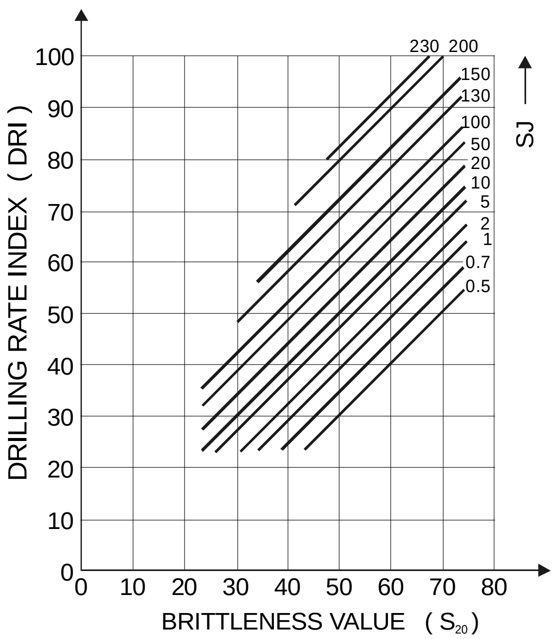


Figure 2.15 Diagram to find the Drilling Rate Index (DRI) (from Selmer-Olsen and Blindheim, 1971).

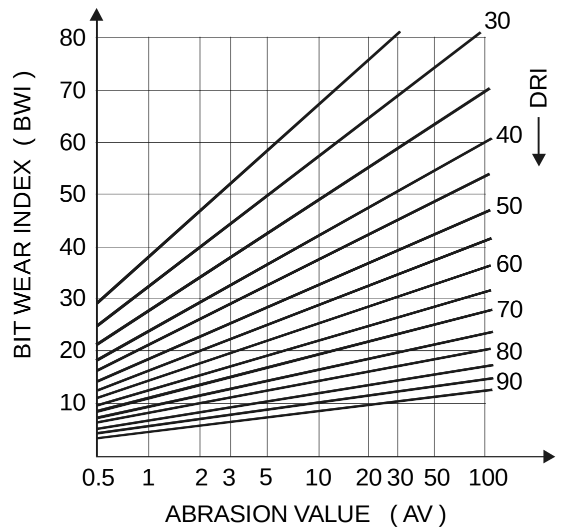


Figure 2.16 Diagram for definition of Bit Wear Index (BWI) (from Selmer-Olsen and Blindheim, 1971).

The "drilling rate index" (DRI) is compiled from the brittleness value and the Siever's J-value as shown in Figure 2.15. The "bit wear index" (BWI) is defined by combining the DRI-value and the abrasion-value, see Figure 2.16

Diagrams have been compiled for correlating the laboratory-indices with field-results obtained by means of various types of equipment. In most cases, however, the DRI- and BWI-values are used primarily for a general characterisation of the drillability of the actual rock. Table 2.13 shows the classification normally used.

In Figure 2.17, DRI and BWI test results for about 200 different rock samples (mainly Norwegian) are presented. As can be seen, the results represent a wide variety in drillability properties. There is, however, very clearly a correlation between DRI/BWI and rock category.

Table 2.13 Classification of the Drilling Rate Index (DRI), the Bit Wear Index (BWI) and the Cutter Life Index (CLI) from NTNU, 1998.

Term	DRI	BWI	CLI
Extremely low	26	< 11	< 5
Very low	26-32	11-20	5-5.9
Low	33-42	21-30	6-7.9
Medium	43-57	31-44	8-14.9
High	58-69	45-55	15-34
Very high	70-82	56-69	35-75
Extremely high	> 82	> 69	> 75

Even within a relatively homogeneous group of rocks such as granite, there may be a considerably variation in DRI/BWI as shown in Figure 2.18. The general relationship between DRI and BWI is, as shown, that rocks with a low DRI tend to have a high BWI, and vice versa.

For evaluating TBM cutter wear, the "Cutter Life Index" (CLI) is used instead of BWI. The main difference from BWI is that in the case of cutter wear an abrasion value (AVS) is defined by using a test specimen made from cutter ring steel instead of hard metal (NTH, 1994). Based on this:

$$CLI = 13.84\ (SJ/AVS)^{0.3847}$$

As its related index for conventional drilling (BWI), the CLI value varies within wide ranges depending on the rock properties. In soft rock such as limestone and shale the CLI-value may be higher than 100, while in very hard rock such as quartzite and diorites it is often lower than 5.

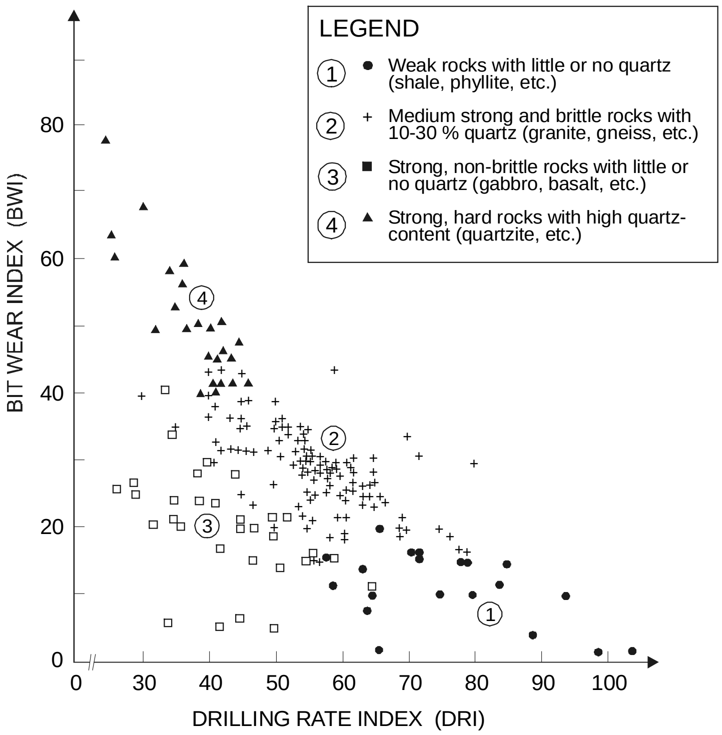


Figure 2.17 DRI and BWI test results for about 200 rock samples tested at NTNU (from Lien, 1979).

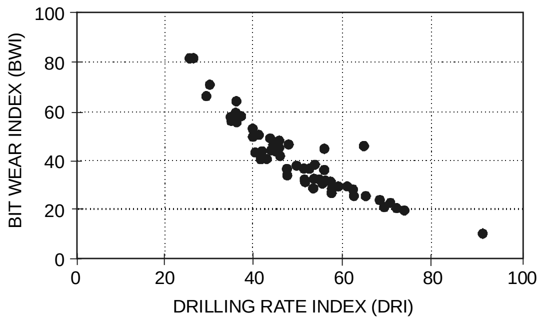


Figure 2.18 DRI and BWI for granites tested at NTNU (from Bruland et al., 1995).

2.7.4.2 Blastability

The result of a blast will depend not only on factors such as type of cut, drill hole pattern, types of explosives and types of detonators. Geological factors are also significant, although in most cases, as indicated in Figure 2.19, the variation of specific charging between "good" and "poor" blastability is within a range of 5-10%.

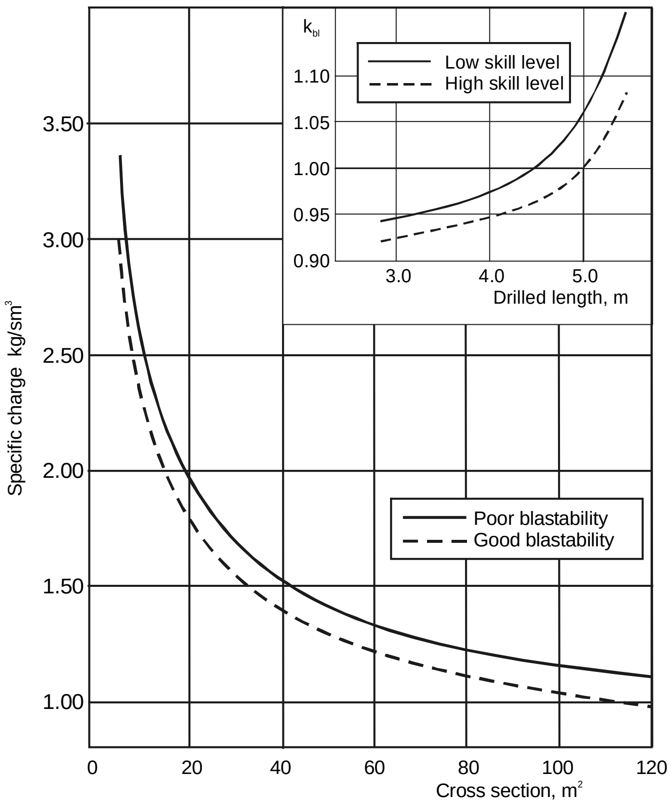


Figure 2.19 Empirical diagram illustrating the relationship between specific charging and the tunnel cross-section for parallel hole cut and 45 mm diameter cartridge explosives. Correction factor for round length. (From NTH, 1995).

In certain cases, the character of the rock may have considerably greater influence on blastability. The net advance per round for "normal" rock conditions will be in the order of 90-100% of the drilled length. In difficult rock it may be much lower and slow down the tunnelling considerably.

The main geological parameters that will influence *blastability*, are:

- The mechanical strength of the rock.
- The degree of jointing.
- The density of the rock mass.
- The anisotropy of the rock mass.

Rock types with a distinct foliation, such as mica *schist* and *phyllite*, are those that most frequently create *blastability problems*. For a hole drilled along the foliation, the situation will, in principle, be as shown in Figure 2.20. In the direction normal to foliation the compressive shock wave will be strongly attenuated, and the tangential stress will haveto overcome a high tensile strength. In the other direction there will be a smaller attenuation, and the tensile strength in the tangential direction is also smaller. The final and unfavourable result may be that only one or a few fissures are initiated along the foliation, and very few normal to this direction.

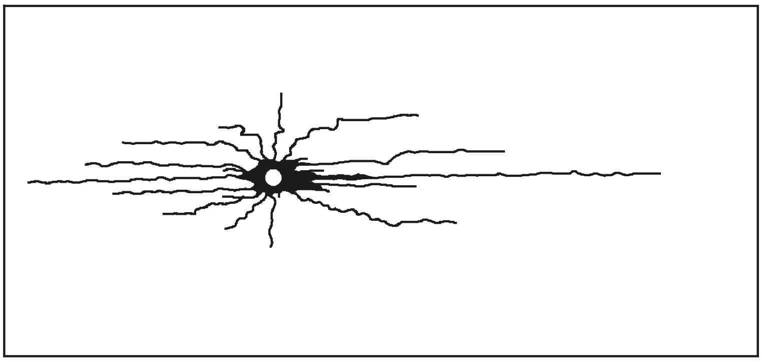


Figure 2.20 Fissures resulting from detonation in a hole drilled along the foliation of an anisotropic rock.

The following are typical characteristics of rocks having favourable blastability properties:
Low to moderate anisotropy (a ratio between max. and min. sonic velocity lower than 1.3).
Moderate mechanical strength (a point load strength within the range 9-14 MPa).
A low density (lower than 2.75 g/cm^3).

Coarse-grained granite, syenite and quartzite are examples of rock types with normally good blastability properties.

2.7.5 Various properties

When rock is subjected to moisture, it will swell or expand, and if originally containing water, it will shrink when drying. Correspondingly, rock will expand when heated. Some figures on *moisture expansion* and *thermal expansion* of hard rocks are given in Table 2.14. As indicated, calcitic rocks tend to have the relatively highest moisture expansion, and concerning thermal expansion, the content of quartz tends to play a major role. The values of thermal expansion (0.5×10^{-5} to 1.2×10^{-5}) correspond to an expansion of 1.1-2.7 mm in a span of 15 m for a temperature increase of 15°C).

Table 2.14 Coefficients of linear moisture and thermal expansion of some Norwegian hard rocks (from Broch & Nilsen, 1996).

Rock	**Moisture expansion (μm from dry to saturated)**	**Thermal expansion (μm/m ×°C)**
Basalt (Permian)	40	8.5
Diorite	100	7.2
Gneiss (Precambrian)		
- cores drilled parallel to foliation	80	5.7
- cores drilled perpendicular to foliation	80	8.8
Quartzite	110	12.1
Marble	280	6.3
Limestone	280	5.0
Sandstone (Devonian)		
- cores drilled parallel to foliation	120	11.0
- cores drilled perpendicular to foliation	160	10.9

Thermal conductivity can be determined in the laboratory or in situ, as in a borehole or deep well, by turning on a heating element and measuring the rise in temperature with time. It depends on several factors, such as:

- Chemical composition of the rock (i.e., mineral content). Often, this feature is mainly governed by the content of quartz, which has a thermal conductivity about three times higher than, for instance, feldspar, mica and amphibole.
- Fluid content (type and degree of saturation of the pore space); the presence of water increases thermal conductivity (i.e., enhances the flow of heat).
- Pressure (high pressure increases the thermal conductivity by closing cracks which inhibit heat flow).
- Temperature.
- Isotropy and homogeneity of the rock.

Typical values of thermal conductivity are given in Table 2.15.

Table 2.15 Typical values of thermal conductivity (from Encyclopaedia Britannica, 1998)

MATERIAL	**THERMAL CONDUCTIVITY** (in 0.001 cal/cm/s per degree Celsius)		
	At 20°C	**At 200°C**	**At other temperatures**
Granite	7.8	6.6	
Gneiss - perpendicular to banding - parallel to banding	5.9 8.2		5.5 (100°C) 7.4 (100°C)
Gabbro	5.1	5.0	
Basalt	4.0	4.0	
Dunite	12.0	8.1	
Marble	7.3	5.2	
Quartzite	15.0	9.0	
Limestone	6.0		
Sandstone - dry - saturated	4.4 5.4		
Shale	3-4		
Rock salt	12.8		
Sand - dry - 30% water	0.65 3.94		
Quartz	20.0		
Feldspars	5.0		
Water			1.34 (0°C) 1.6 (80°C)
Ice			5.3 (0°C) 9.6 (-130°C)

3 Discontinuities

3.1 Discontinuity classification

Discontinuity is any structural or geological feature that changes or alters the homogeneity of a rock. Discontinuities constitute a tremendous range, from structures of up to several kilometres in extent down to a few centimetres, see Figure 3.1.

Discontinuity is the general term for any mechanical discontinuity in a rock mass having zero or close to zero tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistosity planes and faults or weakness zones.

Different types of discontinuities may have completely different engineering significance. The roughness, nature of their contacts, degree and nature of weathering, type and amount of possible gouge and susceptibility to groundwater flow will vary greatly from one type of discontinuity to another since their origin, age and history of development may be fundamentally different. Thus, the effect of a discontinuity on a rock mass varies considerably.

Most discontinuities are a result of *tectonic activity*, i.e., movements in the earth's crust. The two main groups are joints and weakness zones. These are described in the following sections.

3.2 Joint characteristics

If joints are found as separations caused by the formation of rock mass itself, they can be called *foliation joints* in a metamorphic rock or *bedding joints* in a sedimentary rock. When joints extend to long distances (more than 30 m) they are referred to as *major* joints and if not as *minor* ones. These discontinuity planes in a rock mass occur in sets and various joint sets form a *joint system* or *joint pattern* in any area.

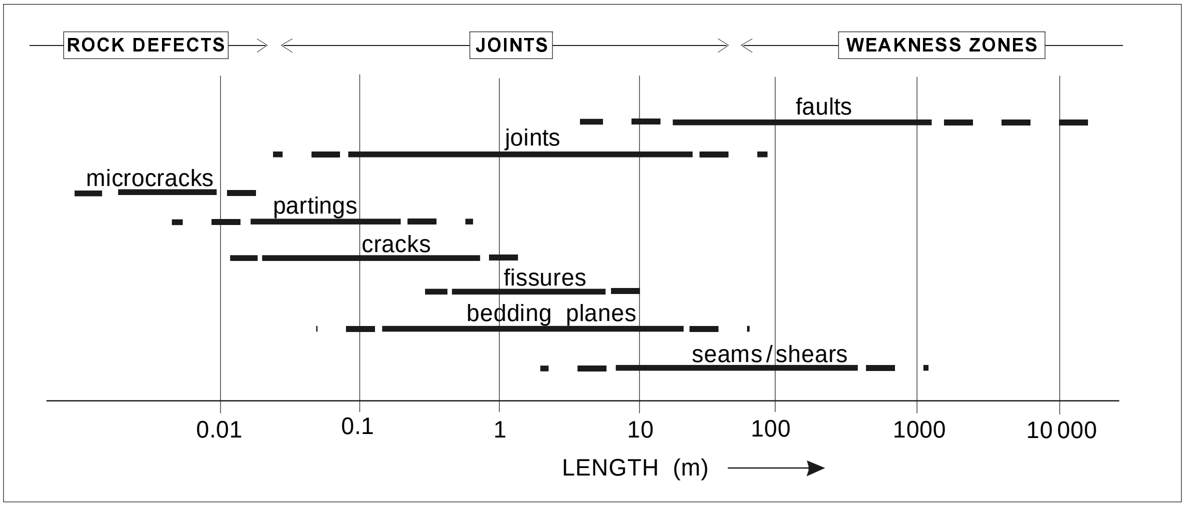


Figure 3.1 The main types of discontinuities according to size. The size range (length) used for joints is indicated (revised from Palmström, 1995).

3.2.1 Definitions

There is a difficulty in giving a precise definition of what constitutes a joint. During the years there have been several discussions as to whether "joint", "fracture" or other terms should be preferred in rock mechanics, engineering geology and rock engineering. ISRM (1975) has chosen "joint" defined as: *"Joint is a discontinuity plane of natural origin along which there has been no visible displacement."*

3.2.1.1 Definitions based on size and composition

The terms for the various types of joints in Figure 3.1 are generally chosen from their size and composition. Some supplementary definitions to these and some others are given as follows:

Crack is a small, partial or incomplete discontinuity. (ISRM, 1975)

Fracture is a discontinuity in rock due to intense folding or faulting. (Dictionary of geological terms, 1962) Fracture is a general term used in *geology* for all kinds of discontinuities caused by mechanical stresses in the bedrock. Fractures include joints and cracks and faults. The use of this term in rock engineering and engineering geology is discouraged.

Parting is a plane or surface along which a rock is readily separated or is naturally divided into layers, e.g., bedding-plane parting. (Glossary of geology, 1980).[2]

Rupture is a fracture or discontinuity caused by excavation works or other human activities.

Seam 1) a minor, often clay-filled zone with a thickness of a few centimetres. When occurring as a weak clay zone in a sedimentary sequence, a seam can be considerably thicker. Otherwise, seams may represent very minor faults or altered zones along joints, dikes, beds or foliation. (Brekke and Howard, 1972).

2) a plane in a coal bed at which the different layers of coal are easily separated. (Dictionary of geological terms, 1962.)

[2] Partings, which often occur as bedding plane and foliation partings, are separations parallel to a mineralogically defined structural weakness in the rock. They are most often tight and rough except where flaky minerals (mica, chlorite) occur.

Shear is a seam of sheared and crushed rock usually spaced more widely than joints and is marked by several millimetres to as much as a metre thickness of soft or friable rock or soil.

Singularity is used as a general term for seams, filled joints, shears or other persistent discontinuities which are not considered as belonging to the *detailed jointing*.

3.2.1.2 Definitions based on origin

Bedding joints/Bedding partings are discontinuities developed along the bedding planes in sedimentary rocks.

Cooling joints are discontinuities formed as a result of the cooling of igneous rocks.

Exfoliation joints are discontinuities developed by a splitting off from bare rock surfaces due to the action of chemical or physical forces, such as differential expansion and contraction during heating and cooling over the daily temperature range. They also include sheeting joints.

Foliation partings/Foliation joints are discontinuities developed along the foliation planes in metamorphic rocks.

Sheeting joints are set of joints developed more or less parallel to the surface of the ground, especially in plutonic igneous intrusions such as granite; probably as a result of the unloading of the rock mass, for example when the cover is removed from erosion.

Tectonic joints3 are discontinuities formed from the tensile stresses accompanying uplift or lateral stretching, or from the effects of regional tectonic compression (ISRM, 1975). They commonly occur as planar, rough-surfaced sets of intersecting joints, with one or two of the sets usually dominating in persistence.

3.2.2 Main joint characteristics

A joint is a three-dimensional discontinuity composed of two matching surfaces called *joint walls*. It is composed of several *characteristics* of which the main are, see Figure 3.2:

- Roughness, waviness (or planarity) of the joint wall.
- Condition of the joint wall, (alteration of wall rock or occurrence of coating).
- Presence of possible filling.
- Length and continuity of the joint

These parameters, which are described in the following, influence the shear strength of the joint as well as the amount of water that can flow through the rock mass.

[3] ISRM (1975) advises against the use of the terms *tension joint* and *shear joint*, since there are many possible ways that they can be developed. For example, tension joints can be developed from cooling of igneous rock, from shrinkage of sediments, from folding, or from ice retreat.

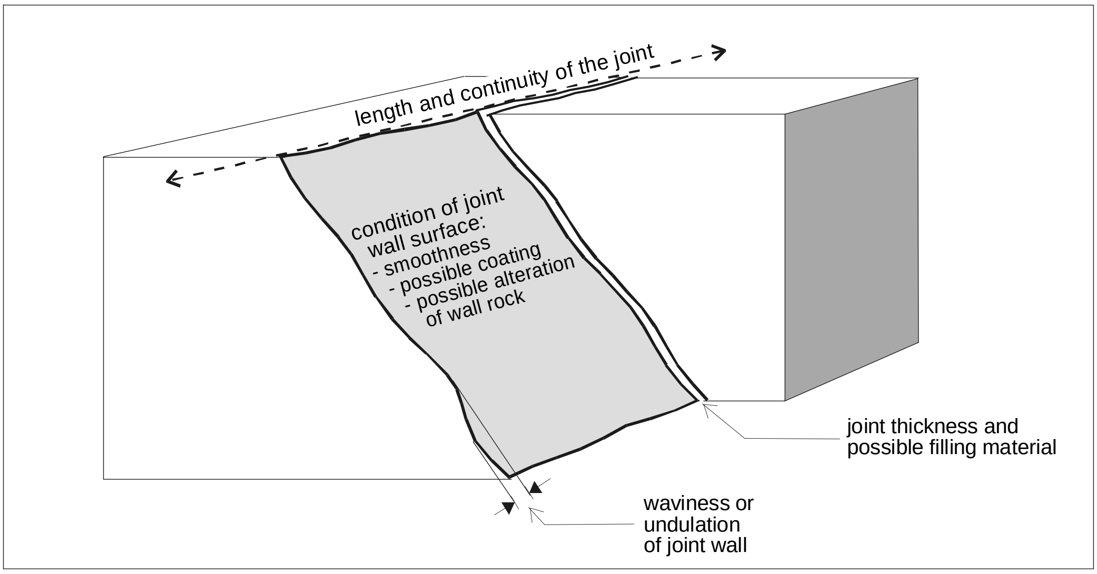


Figure 3.2 Sketch showing the main features of a joint.

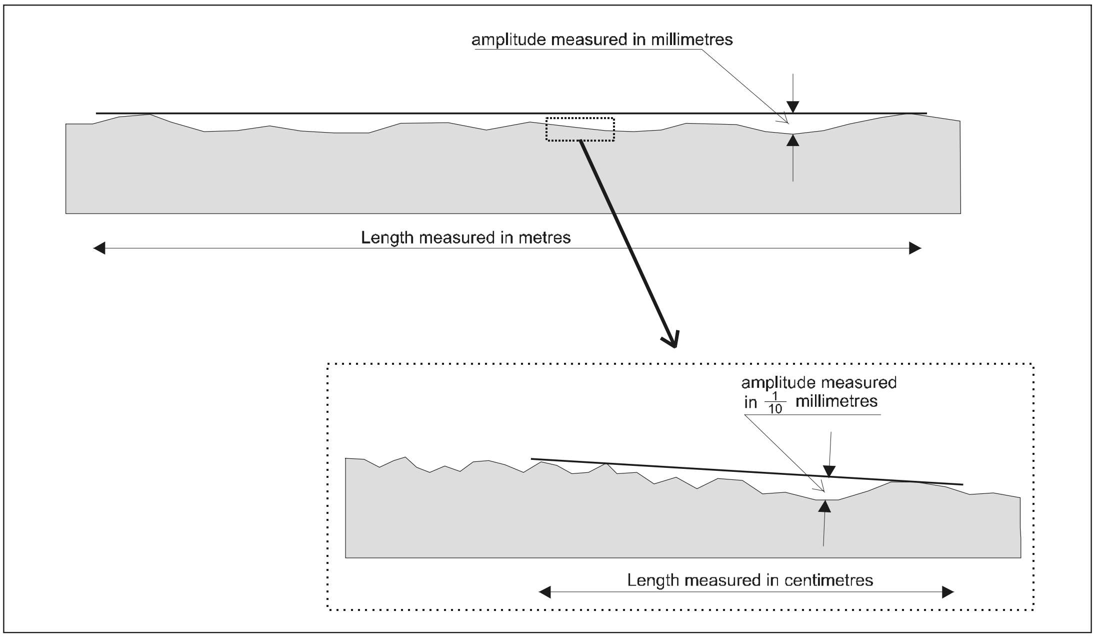


Figure 3.3 The joint wall features can be characterised by the large scale waviness and the small scale smoothness (or unevenness).

3.2.2.1 Roughness

The *roughness of joint* walls is characterised by a large scale waviness and a small scale smoothness or unevenness, ISRM (1978), see Figure 3.3. During shear displacement the waviness undulations, if locked and in contact, cause dilation since they are too large to be sheared off, while the asperities of the smoothness tend to be damaged unless the joint walls are of high strength and/or the stress level is low.

Ideally, the *joint waviness* should be measured as the ratio between max. amplitude and joint length. As it is seldom possible to observe the whole joint plane, a simplified measurement is normally carried out, the so-called undulation factor, representing the ratio between max. amplitude (A) and a reduced, measured length (L) along the joint plane:

$$u = \frac{\text{amplitude from planarity (A)}}{\text{measured length along joint (L)}}$$

The classification of the waviness of the joint plane is shown in Table 3.1.

Table 3.1 Classification of joint plane undulation based on measurements related to 1 m profile length (from Milne et al., 1992)

Classification	**Undulation**
Wavy joints	u > 2%
Planar to wavy joints	u = 1 - 2%
Planar joints	u < 1%

The longest possible ruler should be used in the measurement of waviness. In many cases, however, (jw) is determined from visual observations alone.

Small asperities or second order projections are designated *smoothness* or *unevenness*. If the joint surfaces are clean and closed, these small asperities interlock and strongly contribute to shear resistance especially at low stresses. Smoothness asperities usually have a base length of some centimetres and amplitude measured in tens of millimetres and are readily apparent on a core-sized exposure of a discontinuity.

As indicated in Figure 3.4 the "sample length" for smoothness is in the range of a few centimetres. There is a general problem to arrive at a quick, numerical estimate of joint smoothness from measurements or visual observations of the joint wall surface. A possible solution is to simply touch the surface with the finger and compare it with a reference surface of known roughness, for example sandpaper of various abrasivity (grain) as indicated in Table 3.2.

Table 3.2 Classification and rating of joint smoothness. The terms are based on Jr in the Q-system, the description partly on Bieniawski (1984).

Term	**Description**
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface.
Rough	Some ridges and steps are evident; asperities are clearly visible; joint surface feels very abrasive (rougher than sandpaper grade 30)
Slightly rough	Asperities on the joint surfaces are distinguishable and can be felt (like sandpaper grade 30-300).
Smooth	Surface appears smooth and feels smooth to touch (smoother than sandpaper grade 300).
Polished	Visual evidence of polishing exists, as often seen in chlorite and specially talc coatings.
Slickensided	Polished and striated surface that results from shearing along a fault surface or other discontinuity.

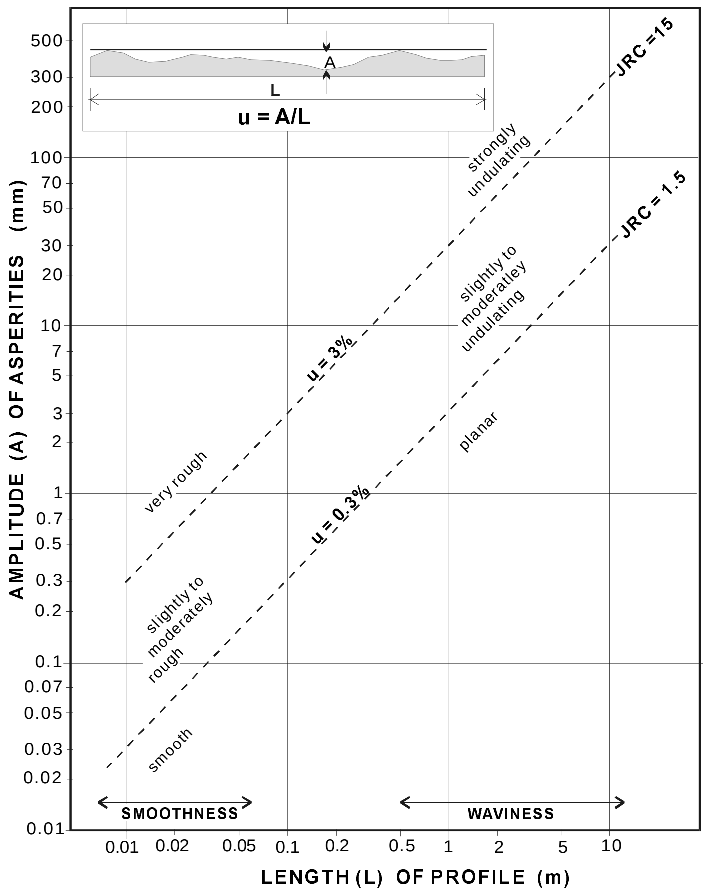


Figure 3.4 Large and small scale roughness measured as waviness smoothness (for JRC see Section 3.2).

3.2.2.2 Alteration and filling

The *character of the joint surface* (wall) which may be fresh, weathered, coated, stained, etc., may strongly influence the roughness and frictional properties of the joint, and hence the strength of the joint surface. Possible weathering or alteration often strongly affects the condition of the joint and may completely change its behaviour. The main types of joint fillings and their properties are shown in Table 3.3.

Filling includes materials derived from breakage of the country rock due to movements (as in crushed zones and breccias), in situ weathered materials (i.e., alteration products), infilling materials deposited between the structural planes (such as calcite), and also intruded igneous materials that are different from the host rock. A filling can, therefore, consist of several different minerals and materials. The main groups are:

- *Hard and resistant minerals* (quartz, epidote and serpentine).
- *Soft minerals* (clay, mica, chlorite, talc and graphite).
- *Soluble minerals* (calcite, gypsum).
- *Swelling minerals* [swelling clays (smectites), anhydrite].
- Loose materials (silt, sand and gravel).

Joints, seams and sometimes even minor faults may be *healed* through precipitation from hydrothermal solutions of quartz, epidote or calcite. This may be the case for layered, igneous and metamorphic rocks in which the layers are strongly *welded* together; therefore such planes are not planes of weakness in the way bedding planes in sedimentary rocks are, but can be regarded more appropriately as planes of reduced strength.

Major discontinuities (*singularities*), such as shears and seams, should be recorded on an individual basis.

Table 3.3 Main joint characteristics and their properties (based partly on Brekke and Howard, 1972).

Type of Joint	Characteristics	Type and properties of material
Clean joints Joints without fillings or coatings	*Healed or welded joints* The joint plane may be regarded often as a plane of reduced strength.	Discontinuities may be healed through precipitation from solutions of quartz, epidote or calcite. Note: Quartz, epidote and calcite may well be present in a joint without healing it.
	Fresh rock walls	These are joint walls of unweathered or unaltered rock. They may, however, show staining (rust) on the surfaces.
	Altered or weathered rock walls The degree of weathering is usually estimated from visual observations (see Table 2.9).	Alteration of the rock material along the joint surface. When *weathering* or *alteration* has taken place it is often more pronounced along the joint surface than in the rock. The wall strength is considerably lower than that of the fresher rock found in the interior of the rock blocks.
Coated joints The joint surfaces have a thin layer or "paint" with some kind of mineral.	Coating will affect the shear strength of joints especially if they are planar, and have a wet coating of chlorite, talc or graphite.	Joint coating, which is not thicker than a few millimetres, can consist of various kinds of minerals, such as chlorite, mica, calcite, epidote, clay, graphite, zeolite.
Filled joints *Filling* or *gouge* is thicker than coating	Chlorite, talc, graphite filling	Very low friction materials, in particular when wet.
	Inactive clay materials filling	Weak, cohesive materials with low friction.
	Swelling clay filling	Exhibits very low friction and swelling with loss of strength. Exerts considerable swelling pressure when confined.
	Calcite filling	May, particularly when being porous or flaky, dissolve during the lifetime of the project, and strongly reduce the shear strength of the joint.
	Gypsum filling	May behave in the same way as calcite.
	Filling of sandy or silty materials	Cohesionless, friction materials. A special occurrence is the thick fillings of *altered or crushed* (sand-like) *materials* that may run or flow immediately after exposure by excavation.
	Filling of epidote, quartz and other hard materials	May cause healing or welding of the joint, resulting in increased shear strength.

3.2.2.3 Size, continuity and persistence

Discontinuous joints terminate in massive rock. Such joints can be foliation partings, *en echelon joints* in addition to many of the smaller joints (less than 1 metre long).

Continuous joints terminate at another joint.

Persistence implies the size, length or area extent, within a joint plane. Joints rarely exceed some hundreds of metres. The size of a joint may vary within the same joint set. This is specially the case for foliation joints where small joints or partings often occur between longer, continuous joints. Persistence can be crudely quantified by observing the joint trace lengths on the surface exposures. Often, rock exposures are small compared to the area or length of persistent joints, and in such cases the real persistence can only be guessed.

Joint continuity or persistence can be distinguished by the terms *persistent, sub-persistent* and *non-persistent* (ISRM, 1978), or more simply as *continuous* and *discontinuous*.

Table 3.4 Classification of joint persistence (from Bieniawski, 1984).

Persistence	**Joint length**
Very low	< 1 m
Low	1 - 3 m
Medium	3 – 10 m
High	10 - 20 m
Very high	> 20 m

3.2.2.4 Friction

The distance between the two matching joint walls controls the extent to which these can interlock. In the absence of interlocking, the properties of the joint filling determine the shear strength of the joint. As separation decreases, the asperities of the rock wall gradually become more interlocked, and the rock wall properties are the main contributor to shear strength.

The terminology *first* and *second order irregularities,* introduced by *Patton* (1966), correspond to large scale undulations (waviness) and small scale irregularities or unevenness (smoothness) on the joint wall surfaces. At very low normal stresses the small scale roughness (smoothness) is mainly contributing to the shear strength of joints, whereas at higher stress the waviness is of main importance.

The *peak friction angle of joints* can be expressed as:

$$\phi = \phi_b + i$$

where ϕ_b = the *basic friction angle* of the joint, and
i = the (peak) *dilation angle* (or Patton's dilation, second order, angle)

The value of ϕ_b is mainly within the range 20-40°; for most smooth unweathered rock surfaces it varies between 25° and 35° as shown in Table 3.5. The values of ϕ_b can normally be estimated based on this table, unless the joint walls are strongly weathered or coated, or the joint contains filling. ϕ_b can be much lower than shown in Table 3.5 if mica, talc, chlorite or other sheet silicate minerals occur on the sliding surface, or when filling is present. Values as low as 6° have been reported in saturated fillings of montmorillonite clay.

Both waviness and smoothness contribute to the dilation or roughness angle (i) which can have any value between 0° and 40°, and even more or more at low pressures. The i-value can be found by applying the *joint roughness coefficient (JRC)* as described by Barton and Bandis (1990), see also Section 10.3.2.1:

$$i = JRC \times \log_{10}\left(\frac{JCS}{\sigma_n}\right)$$

Here JCS = the *joint wall compressive strength* (for fresh rocks JCS = σ_c)
σ_n = the normal stress across the joint. For the peak dilation angle the normal stress is very low (approximately 0.001 MPa, as in the tilt test).

Table 3.5 *Basic friction angles* of various unweathered rocks obtained from flat and residual surfaces (from Barton and Choubey, 1977)

Sedimentary Rocks	Basic friction angle (ϕ_b)	**Metamorphic rocks**	Basic friction angle (ϕ_b)	**Igneous rocks**	Basic friction angle (ϕ_b)
Sandstone wet Sandstone dry	25-34 (31)* 26-35 (32)	Amphibolite dry	32	Basalt wet Basalt dry	31-36 35-38
Siltstone wet Siltstone dry	27-31 31-33	Gneiss wet Gneiss dry	23-26 26-29	Granite, fine grained wet Granite, fine grained dry	29-31 31-35
Shale wet	27	Slate wet Slate dry	21 25-30	Granite, coarse grained wet Granite, coarse grained dry	31-33 31-35
Conglomerate dry	35			Porphyry wet Porphyry dry	31 31
Chalk wet	30			Dolerite wet Dolerite dry	32 36
Limestone wet Limestone dry	27-35 31-37				

* numbers in parenthesis indicate average values

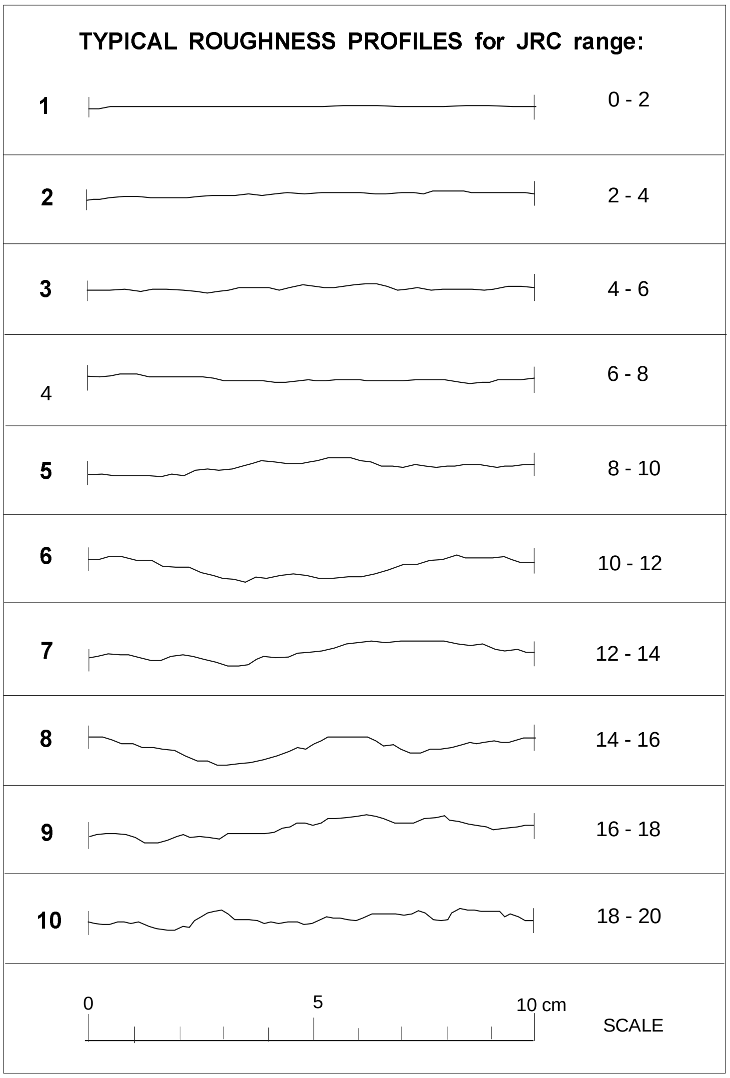


Figure 3.5 Typical *roughness profiles* for various JRC ranges (from Barton and Choubey, 1977)

JRC ranges from less than 5 for smooth, planar joints to 20 for rough, undulating joints. It is subjectively estimated from comparison with standard roughness profiles as shown in Figure 3.5. The factor $\log_{10}(JCS/\sigma_n)$ in the equation corrects JRC for asperity shearing.

A rough estimate of the joint friction angle can be made from the ratio:

$$\frac{Jr}{Ja} \approx \tan^{-1}\phi = \tan^{-1}(\phi_b + i)$$

3.2.2.5 Some other joint features

The peak *shear stiffness of jointed rocks* defined by Goodman (1970) is:

$$Ks = \sigma_n{}' \frac{\tan\phi'}{\delta_{peak}} \quad \text{(MPa/mm)}$$

where σ_n = the normal stress (MPa)
δ = slip magnitude required to mobilise peak strength (mm)

The shear stiffness depends on the normal stress as shown in Figure 3.6.

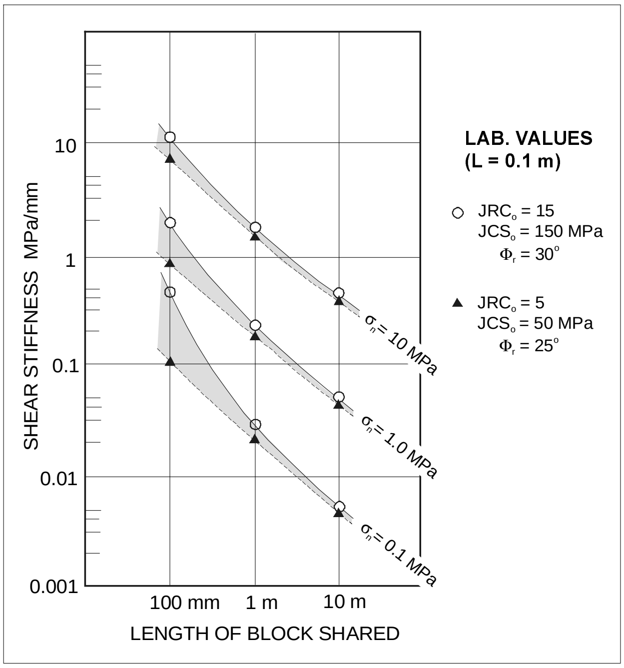


Figure 3.6 Variation of shear stiffness with joint size (block length) and joint roughness for clay-free joints (from Barton 1990).

Thickness of joints is the maximum distance between joint walls. It is generally small, usually less than a millimetre, except for filled joints, seams or shears. The size of the joint is often proportional to the thickness or separation of the joint.

Aperture is the perpendicular distance separating the adjacent rock walls of an ***open*** discontinuity, in which the intervening space is air or water filled. Aperture is thereby distinguished from the width of a filled discontinuity (ISRM, 1978).)

Table 3.6 Classification of separation of joints (from Bieniawski, 1984)

Term	Separation
Very tight	< 0.1 mm
Tight	0.1 - 0.5 mm
Moderately open	0.5 - 2.5 mm
Open	2.5 - 10 mm
Very open	10 - 25 mm

3.2.2.6 Statistical distribution of joints

The most commonly measured geometric properties of jointing are spacing (or density), trace length, and orientation. Based on results of several research projects, the *statistical distribution* of joint density can often be modelled by the following exponential function:
Joint spacing by an exponential function
Joint trace length by an exponential or log-normal function

3.3 Jointing

Jointing is the occurrence of joint sets forming the system or pattern of joints as well as the amount or intensity of joints. The network of joints in the massifs between weakness zones is often referred to as the *"detailed jointing*".

Joints are found in certain, preferred directions. One to three prominent sets and one or more minor sets often occur; in addition several individual or *random joints* may be present.

The joints delineate *blocks*. Their dimensions and shapes are determined by the joint spacings, by the number of joint sets and by the random joints. The block size is as an extremely important parameter in rock mass behaviour. A number of scale effects from compressive strength, deformation, shear strength, etc., in rock engineering can be explained by this feature.

Different methods are used for measuring the degree of jointing (jointing density). The most common are:

- Joint spacing, either on surfaces, in drill cores, or along scan lines.
- Density of joints, either on surfaces, in drill cores, or along scan lines.
- Block size, on surfaces.
- Rock quality designation (RQD), in drill cores.

These are further outlined in Section 3.4, where correlation equations between them are also given.

Strike and dip definition and measurement are described in Section 8.3.3.

3.3.1 Joint sets

The conditions of the joints in the various sets can vary greatly depending on their mode of origin and the type of rocks in which they occur. Not only can the size and average spacing of joints vary, but also all other characteristics mentioned in Section 3.2. Variations in these properties can cause one *joint set* to have a stronger effect on the shear strength characteristics than another joint set.

Although some characteristics are often common for joints of different sets in a structural region, there is seldom any general connection between all joint conditions in the different types of rock. Thus, for each of the joint sets within a structural region with similar jointing, the various properties of each set must be considered individually.

In many cases one joint set is dominant, being both larger and/or more frequent than joints of other sets in the same locality. This set is often referred to as the *main joint set* (or by geologists as primary joints).

3.3.2 Jointing pattern, blocks types and shapes

The joint sets and possible random joints divide the rock volumes into characteristic blocks. The *jointing pattern* and the difference in spacing between the joints within each joint set determine the shape of the resulting blocks, which can take the form of cubes, rhombohedrons, tetrahedrons, plates, etc. The *unit block* can be described by its volume, and in addition the type and shape as shown in Figure 3.7 and Tables 3.7 and 3.8.

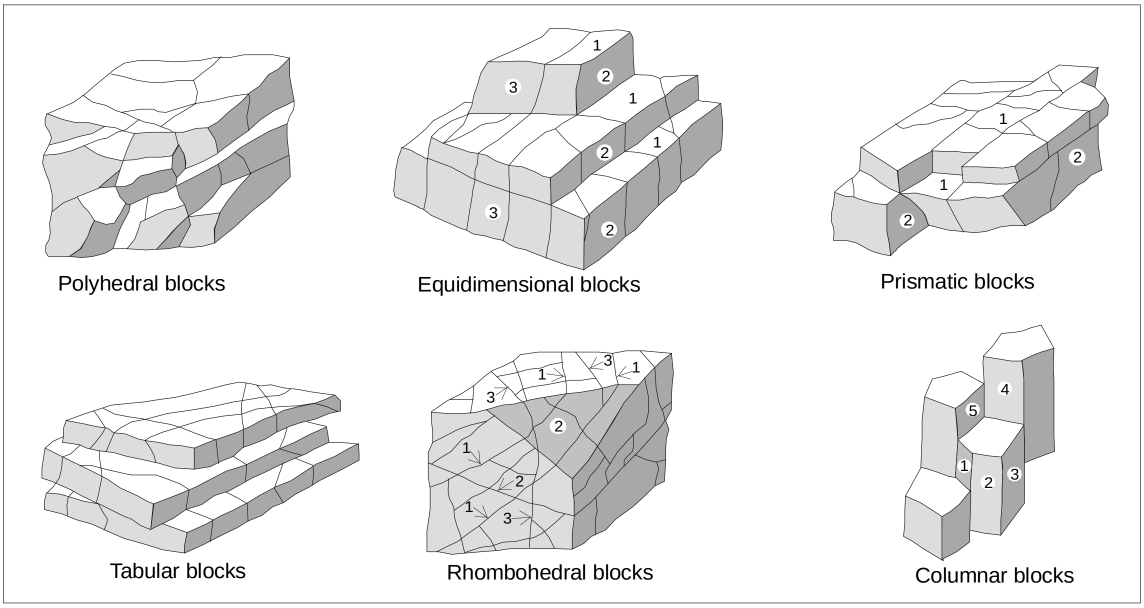


Figure 3.7 Various types of rock blocks. The numbers shown refer to various joint sets (modified from Dearman, 1991).

Table 3.7 Simplified division of *block shapes* and jointing patterns (from Palmström, 1995).

Block shape	**Type of jointing pattern and differences in joint spacing**
Compact (equi-dimensional)	3 or more joint sets with approximately the same joint spacings
Long (columnar)	2 joint sets with similar spacing plus random joints, or 2 joint sets with similar spacing and additional sets with significantly larger spacings
Flat (tubular)	1 joint set and random joints, or 1 joint set with considerably smaller spacing than the other sets
Long and flat	This term is applied for long blocks where the two sets have significantly different spacings.

The block shape can be characterised by the *block shape factor* which is an expression involving the differences between dimensions of the block. The block shape factor can be found from the simplified expression:

$$\beta \approx 20 + 7\ (L_{max} / L_{min})$$

where L_{max} and L_{min} are the longest and shortest dimension of the block

The block shape factor, which varies between 27 and 1000 or more, can be roughly determined from observations in the field or in an underground opening after some training. Normal variations for the various types of blocks are shown in Table 3.8.

Table 3.8 The block shape factor for various types of blocks.

Type of block	**Block shape factor**	**(Average)**
Compact (equi-dimensional) blocks	β = 27 - 32	30
Slightly long or flat blocks	β = 32 - 50	40
Moderately long or flat blocks	β = 50 - 100	72
Very long or flat blocks	β = 100 - 500	270
Extremely long or flat blocks	β > 500	720

3.3.3 Joint spacing and block size

Joint spacing is the perpendicular distance between two joints within a joint set. It may vary from some millimetres to many metres. General trends are:

- Rock masses that have undergone tectonic disturbance often present clusters of joints (joint zones). Often the joint spacing decreases near faults and shear zones.
- Spacing is influenced by weathering as there often is an increase of joints within the zone of weathering, especially where mechanical disintegration has taken place.

Block size is a volumetric expression for jointing density. It is a result of the *detailed jointing* mainly formed by the small and moderate joints. The block dimensions are determined by joint spacings and the number of joint sets. Individual or random joints and other possible planes of weakness may further influence the size and shape of rock blocks, as well as impact from excavation works.

The connection between soil diameter (d) and block volume (Vb) is shown in Table 3.9.

Table 3.9 Classification of *density of joints*, *volumetric joint count*, and *block volume*, related to particle size of soil (from Palmström, 1995)

Degree of jointing	**Volumetric joint count**		**Block volume**		**Soil particles***	
(or density of joints)	**term**	**Jv**	**term**	**Vb**	**term**	**Volume**
Massive/no joints	Extremely low	<0.3	Extremely large	>1000 m^3		
Massive/very weakly jointed	Very low	0.3-1	Very large	30-100 m^3		
Weakly jointed	Low	1-3	Large	1-30 m^3		
Moderately jointed	Moderately high	3-10	Moderate	0.03-1 m^3	Blocks	>0.1 m^3
Strongly jointed	High	10-30	Small	1-30 dm^3	Boulder	5-100 dm^3
Very strongly jointed	Very high	30-100	Very small	0.03-1 dm^3	Cobbles	0.1-5 dm^3
Crushed	Extremely high	>100	Extremely small	< 30 cm^3	Coarse gravel	5-100 cm^3

*$V = 0.58\ d^3$ has been applied for the correlation between particle volume and particle diameter

3.4 Methods to determine the degree of jointing

Several methods have been developed to measure the quantity or density of joints in the rock mass. The method or methods to be applied at an actual site are normally selected according to the availability of observable rock and jointing in an exposure, requirements regarding the necessary quality of collected data, the type and cost of the investigation or survey, and the experience of the engineering geologist.

As joint spacings generally vary greatly, the difference in size between smaller and larger blocks can be significant. Therefore, the characterisation of block volume should be given as an interval rather than as a single value.

Where less than three joint sets occur, defined blocks are often not expected to be found. However, in most cases random joints or other weakness planes will contribute to block definition. Where the jointing is irregular or many of the joints are discontinuous, the actual size and shape of individual blocks may also be difficult to recognise. Therefore, the block size and shape may sometimes have to be determined from reasonable simplifications where an *equivalent block volume* is assumed. Simplifications may also be necessary to keep the amount of work necessary to collect the data within reason.

3.4.1 The rock quality designation (RQD)

Rock quality designation is probably the most commonly used method for characterising the degree of jointing in borehole core. *RQD* can be regarded as an indirect measurement of block size. It is defined as the percentage of core bits longer than 0.1 m along the measured length of the core. An increase in the number of joints in a rock mass causes a decrease in RQD. The RQD-classification, developed by Deere (1966), is shown in Table 3.10.

RQD is rapid and easily learned. Today, RQD is used in the main classification systems as an input parameter for block size or jointing density. RQD is one-dimensional; and therefore it has the weakness of being strongly directional.

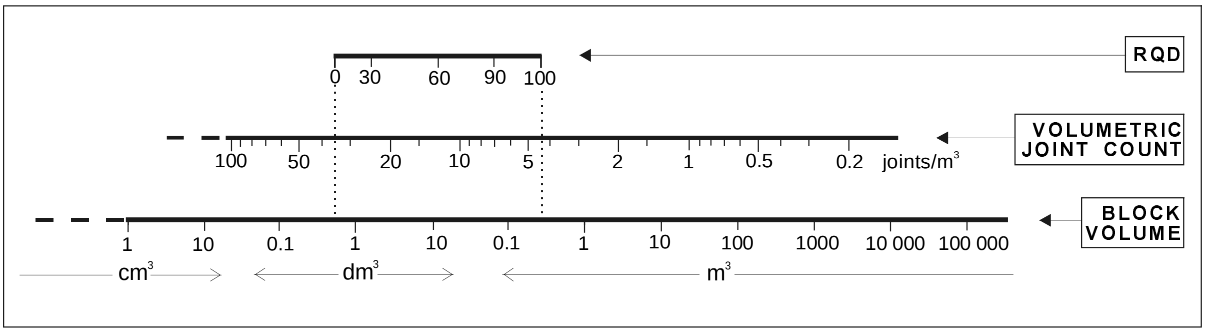


Figure 3.8 Range of jointing covered by RQD, block volume (Vb), and volumetric joint count (Jv).

Figure 3.8 shows that RQD only covers a small part of the range of block sizes possible in a rock mass, which means that RQD is not able to express variations for a high or low degree of jointing.

The weighted jointing density method, which is fairly rapid and simple, offers an attractive improvement to core characterisation, see Section 3.4.2.

Table 3.10 Classification of the RQD (from Deere, 1966).

Term	RQD
Very poor	< 25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

3.4.2 The volumetric joint count (Jv)

The volumetric joint count (Jv) is the number of joints intersecting a rock mass volume of 1 m³. Where the jointing is formed only by joints sets it can be found from:

1. Measuring the spacing of each joint set:

$$Jv = \Sigma\, 1/S = 1/S1 + 1/S2 + 1/S3 + \ldots..$$

   where S1, S2, S3, etc., are joint spacings for the various joint sets given in metres

2. Where joint sets and additional random joints occur, a "spacing" of 5 m is applied for each random joint seen in 1 m³ volume:

$$Jv = \Sigma\,(1/S) + Nr/5) = (1/S1 + 1/S2 + 1/S3 + \ldots) + Nr/5$$

   where Nr = the number of random joints

3. From frequency in drill cores:

$$Jv = kl \times Nl$$

   where Nl = the number of joints per metre along the core;
   kl = a correlation factor, commonly kl ≈ 2

4. From rock quality designation measurements (RQD):

$$Jv = 35 - RQD/3.3$$

5. From *weighted joint density (wJd)* observations:
   - in boreholes: $Jv \approx wJd = \Sigma \frac{n_i \times f_i}{L}$ (L is measured length in m)
   - on surfaces: $Jv \approx wJd = \Sigma \frac{n_i \times f_i}{\sqrt{A}}$ (A is size of observation area in m²)

   Here, f_i is a factor given for the intersection angle between the joint and the plane or borehole with ratings as given in Table 3.11. For every joint the angle is estimated and the rating of f_i is noted. The wJd is found as the sum of the ratings divided by the interval length or the measurement area.

Table 3.11 Ratings of the angle factor f_i (from Palmström, 1995)

Angle	Rating of f_i
< 15°	6
15-29°	3.5
30-59°	1.5
> 60°	1

Example

From 5 m of borehole cores are found:
9 joints intersect at > 60°, 6 joints at 30-59°, 4 joints at 15-29°
And 2 joints intersect at < 15°
This gives wJd = $(9 \times 1 + 6 \times 1.5 + 4 \times 3.5 + 2 \times 6)/5 = 8.8$

Classification of the Jv is shown in Table 3.9.

3.4.3 Block size

The *block volume (Vb)* may be found by using different methods in the underground opening, on the surface, in cuttings, or in drill cores.

Direct measurement can be made where the rock masses can be observed on surfaces, in cuttings and in underground excavations. The volumes will generally vary considerably at each site, and it is generally recommended to record the variation in volumes in addition to the average volume.

Often, it is not possible to observe the whole individual block, especially where less than three joint sets occur. As fissures formed during the excavation process or random joints will often result in defined blocks, a *random spacing* Sf = $5 \times$ S1 to $10 \times$ S1 can be used to estimate a block volume. Here S1 is the spacing of the main joint set.

The block volume can then be found from the volumetric joint count using the expression:

$$Vb = \beta \times Jv^{-3}$$

For the frequent (common) value of $\beta = 36$ the block volume $Vb = 36\ Jv^{-3}$
For more information on β, see Section 3.3.2.

Example
In a bedded limestone with 20 cm average between the bedding joints plus some random joints, the block volume Vb = $0.2 \times (0.2\times5) \times (0.2\times10) = 0.4\ m^3$

3.4.4 Refraction seismics

The *degree of jointing* can be estimated along a seismic profile by means of mathematical correlation. This is often of particular interest at an early stage in the planning of a project, before information on jointing from core drilling or surface mapping is available.

The velocities of seismic waves vary considerably with rock type, but also with rock stress, groundwater conditions, rock anisotropy and weathering, as well as local topographical conditions, see Section 8.3.5. The calculation of block size from seismic velocities is therefore very approximate. However, using the principles in Section 3.4.4.2 may provide satisfactory results since they are linked to real field conditions.

3.4.4.1 No information available on the jointing

The following, approximate expressions may be used for calculating the joint frequency/number of joints per metre (Palmström, 1995):

$$Nl = V_0^{3.4} \times v^{-2.8} \text{ or } Nl = 3\left(\frac{V_0}{v}\right)^{\frac{V_0}{2}}$$

where V_0 = the *basic velocity* (km/s) of intact rock under the same conditions as in situ
v = the measured in situ seismic velocity (km/s)

Important in both expressions is the use of basic velocity (V_0) for intact rock under the same conditions as in the field (wet or dry conditions, orientation of rock anisotropy, stress level, etc.). Joint openness and possible joint fillings may, however, disturb the accuracy of this method as V_0 for surface measurements is assessed from laboratory measurements, or from textbook tables such as Table 3.12. Therefore, the method described in the next section, which includes site-dependent features, gives more accurate results.

3.4.4.2 At least two correlations between jointing and seismic velocities are known

The method, which has been developed by Sjögren et al. (1979), makes use of the following relation for joint frequency:

$$Nl = \frac{\frac{1}{v} - \frac{1}{N_n}}{ks}$$

where V_n = the maximum or *"natural" velocity* (km/s) in crack- and joint-free rock. The "natural" velocity for some rocks measured in the laboratory are shown in Table 3.12
ks = a constant representing the actual in situ conditions

The data on the jointing can be found from observation of joint frequency along the seismic profile, and/or logging of the frequency in cores from boreholes located in or near the seismic profile. Data from at least two different locations are required to perform the assessments.

Table 3.12 Approximate (natural) velocities of fresh rocks, free from cracks and pores (from Goodman, 1989, based on data from Fourmaintraux, 1976).

Rock	**V_n** (km/s)	**Rock**	**V_n** (km/s)
Gabbro	7	Basalt	6.5-7
Limestone	6-6.5	Dolomite	6.5-7
Sandstone and quartzite	6	Granitic rocks	5.5-6

The number of joints per metre, Nl, can best be found from the calculation of the two unknown constants (ks) and (V_n), applying two data sets of measured values of (Nl) and the corresponding (v) in the following equations (derived from the equation above):

$$V_n = \frac{v_1 \times v_2 (Nl_2 - Nl_1)}{Nl_2 \times v_2 - Nl_1 \times v_1} \quad \text{and} \quad ks = \frac{1}{Nl_1}\left(\frac{1}{v_1} - \frac{1}{V_n}\right)$$

Here, Nl_1, v_1 and Nl_2, v_2 are corresponding values of joint frequency and measured in situ seismic velocity, respectively, for the two pairs of measurements.

When ks and V_n have been found the joint frequency (given as joints/m) is

$$Nl = \frac{V_n - v}{V_n \times v \times ks}$$

According to Sjögren et al. (1979), these theoretical calculations of average jointing frequencies have shown a satisfactory agreement with those obtained empirically. The discrepancies between them have been less than 0.5 joints/m.

3.4.4.3 Example

The data in Table 3.13, retrieved from one core drilling and surface joint observations along a seismic profile, are used to establish the relation between the degree of jointing (given as Nl) and the seismic velocities (v).

Table 3.13 The data used from drill cores and seismic measurements.

Seismic velocity	**Joints/m**	**Comment**
v_1 = 4.5 km/s v_2 = 3.5 km/s	Nl_1 = 4.5 Nl_2 = 11	Average for joints measured where v = 4.5 km/s in the profile Average found in a borehole (at 3.5 km/s seismic velocity)

Combining the data sets 1 and 2 in Table 3.13, the two unknown constants, ks and V_n are found as:

$$V_n = \frac{v_1 \times v_2 (Nl_2 - Nl_1)}{Nl_2 \times v_2 - Nl_1 \times v_1} = \frac{4.5 \times 3.5 (11 - 4.5)}{11 \times 3.5 - 4.5 \times 4.5} = 5.61 \, \text{km/s}$$

$$ks = \frac{1}{Nl_1}\left(\frac{1}{v_1} - \frac{1}{V_n}\right) = \frac{1}{4.5}\left(\frac{1}{4.5} - \frac{1}{5.61}\right) = 0.010$$

From this, the correlation between joint frequency, given as Nl (joints/m), and velocities (v) along the profile is:

$$Nl = \frac{V_n - v}{V_n \times v \times ks} = \frac{5.61 - v}{5.61 \times 0.01v} = 17.8\frac{5.61 - v}{v}$$

From the expression above, the value of Nl can be found for the various seismic velocities along the profile. The block volume can be found from
$Vb = \beta Jv^{-3} = \beta(kl \times Nl)^{-3} \approx 0.125\, \beta \times Nl^{-3}$ (for kl ≈ 2, see Section 3.4.2)

Most frequently $\beta = 36$, which gives $Vb \approx 4.5\, Nl^{-3}$

Note that local differences such as composition of rocks, joint characteristics, stress level, etc., are averaged in the calculations above.

3.5 Weakness zones and faults

3.5.1 Definitions

Fault Major rupture zone ranging in width from a decimetre to more than a hundred metres, occasionally more than a thousand metres. The walls are often striated and polished (slickensided) as a result of shear displacement. Frequently rock on both sides of a fault is shattered and altered or weathered, resulting in fillings such as breccia and gouge. In order to be characterised as “fault”, there must be proof of movement.

Normal fault Dipping fault in which the overlying face or wall appears to have moved downward relative to the underlying face. The fault angle is usually 45° to 90°.

Reversed fault Fault, resulting from the action of compressional forces, where the overlying face or wall appears to have moved upward relative to the underlying face. The fault plane usually dips at a low angle.
Shear fault or *strike-slip fault* occurs where the movement is usually horizontal along the strike of the fault.
Wrench fault is a nearly vertical strike-slip fault.
Weakness zone is a part of the rock mass where mechanical properties are significantly lower than those of the surrounding rock mass. Weakness zones can be faults, shears/shear zones, thrust zones, weak mineral layers, etc.
Crushed zone is a weakness zone consisting of a central, crushed part, and a gradual transition, often with filled joints, to adjacent rock.
Gouge is clay-like material occurring between the walls of a fault as a result of the movement along the fault surfaces. (ISRM, 1978)
Filling in this connection is used for the finer, often clay-like material occurring between the walls of a weakness zone or seam (filled joint).

3.5.2 Faults - occurrence and formation

Minor faults normally range in thickness from a decimetre to a metre, major faults from several metres to hundreds of metres. It is important to realise that most fault zones are the result of numerous ruptures throughout geological time, and that they quite often are associated with other parallel discontinuities that decrease in frequency and size in the direction away from the central zone.

Faults and fault zones often form characteristic patterns in the earth's crust, consisting of several independent sets or systems, see Figure 3.9. The main directions, which usually were determined by the state of stress, often have the same orientations as the joint sets within the same structural area.

The fact that faults and weakness zones can have a major impact upon the stability as well as on the excavation process of an underground opening, necessitates special attention during planning, follow-up and investigations often are necessary to predict and if possible avoid such events.

Although all weakness zones basically are composed of mainly rock(s) in addition to joints and seams with or without filling, a great variety exists, see Figure 3.10. Common for all is, however, that they form zones, planes, lenses, veins or layers. Basically, there are two main groups of weakness zones: 1) those which are formed from tectonic events, and 2) those consisting of weak materials formed by other processes. Weathering, hydrothermal activity and alteration are features that may have had a significant impact on the composition and properties of a zone.

Faults and *crushed zones* can vary greatly in composition from mostly brecciated or crushed material with relatively small amounts of clay to highly weathered or hydrothermally altered, highly plastic, swelling clay gouge. The composition of the rock fragments in the zone can be similar to the adjacent rock, or hydrothermal solutions can have altered the original rock material and/or brought

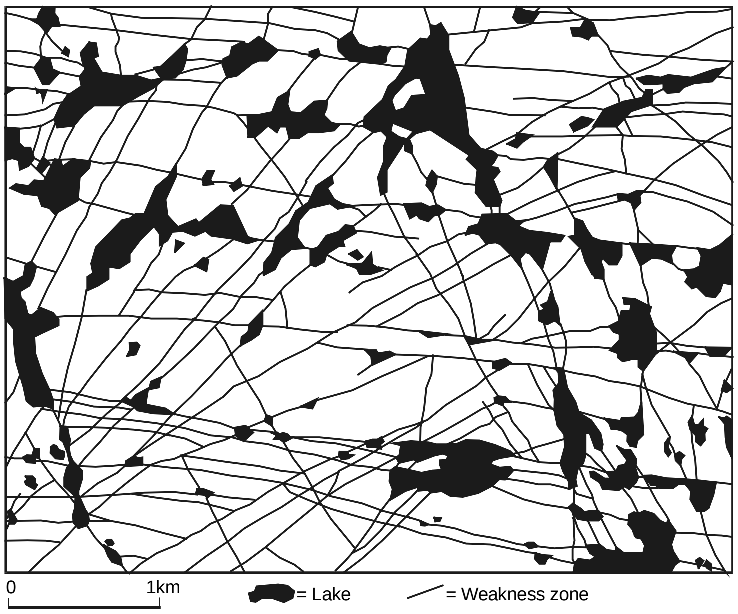


Figure 3.9 Pattern of weakness zones and faults in the earth's surface (from Selmer-Olsen, 1988).

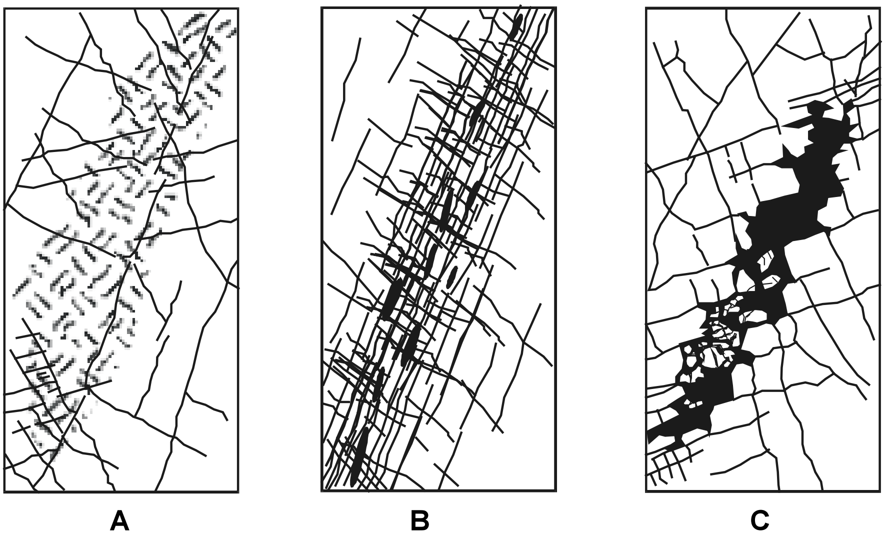


Figure 3.10 Sketches of some types of weakness zones developed as faults. Black indicates filling or gouge. [based on ISRM, 1978 (A-C)]

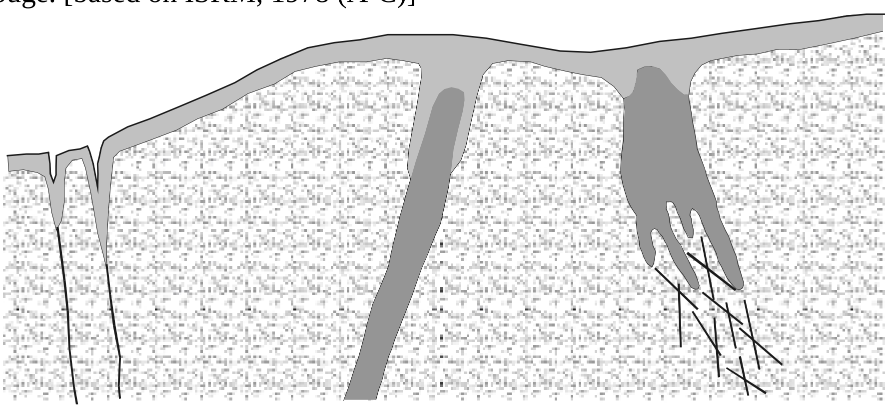


Figure 3.11 The action of weathering along joints, rock boundaries and crushed zones in and near the surface (from Morfeldt, 1976).

Table 3.14 Description of various types of *weakness zones.*

Types of weakness zones	Occurrence and formation
Zones of weak materials *Layers of soft or weak minerals and rocks* - clay materials - mica, talc, or chlorite layers and lenses - coal seams - some dykes - some pegmatites, often heavily jointed - some brecciated zones and layers which have not been "healed" *Weathered rock mass*	Many of the zones of *weak* materials are only regarded as weakness zones if they are surrounded by other, stronger rock masses. The (weak) material in these zones may consist of clay, pegmatite, mica or chlorite, poorly cemented sedimentary layers (for example tuff layers in basalts), or coal layers. The zone has often a sharp boundary to adjacent, stronger rocks. *Weathered* rock mass also belongs to this group. The weathering process has often acted along rock layers, dykes or rock contacts, or along joints, seams, and crushed zones to form zones, layers or pockets of weathered products with low mechanical properties.
Faults and fault zones *Tension fault zones* - feather joints and filled zones, > clay-filled zones > calcite-filled zones *Shear fault zones* - coarse-fragmented, crushed zones - small-fragmented, crushed zones - sand-rich crushed zones - clay-rich, crushed zones, such as: > simple, clay-rich zones > complex, clay-rich zones > unilateral, clay-rich zones -foliation shears *Altered faults* - altered, clay-rich zones - altered, leached (crushed) zones - altered veins/dykes	These zones are the result of numerous ruptures throughout geological time and their composition and magnitude may vary greatly. *Tension fault zones* are often developed with a filling of soft minerals between parallel walls. The filling material can be chlorite, (swelling) clay, porous calcite, silt, etc. Such zone are named according to the dominant filling. Feather or pinnate zones ("*fiederspalten*"), see Figure 3.13, belong to this group. There is generally a sharp boundary to the adjacent rocks. *Shear fault zones* are crushed and brecciated by many intersecting joints and/or seams. Their central part may sometimes be weathered or completely altered to clay. These zones can vary in width from a few centimetres to several metres. Where shear zones occur parallel with the foliation, typically along weak mica-rich schist layers, they are of ten termed *foliation shear zones*. *Alteration* of faults may take place in most types of the zones described above. The alteration processes may occur during the formation of the zone and/or later.
Recrystallized and cemented or Welded zones	Recrystallization may cause significant changes to the composition, properties and behaviour of a weakness zone. These types which probably earlier have been crushed zones, are still geologically named faults or thrust zones. They often have some slickensided and clay filled joints, with secondary formed minerals of epidote, quartz, feldspar, etc. which have "welded" the blocks and “reinforced” the zone.

in and deposited new minerals that are not associated with the petrography of the adjacent rock. Many faults and weakness zones thus contain materials quite different from the “host” rock. The problems related to weakness zones may depend on several factors that may all interact in its final behaviour.

Weathering may greatly influence the character of the zone as a function of depth, see Figure 3.11.

In Table 3.14, a classification of weakness zones is given according to Selmer-Olsen (1964, 1971), who studied many of the weakness zones encountered in the 5000 km of Norwegian tunnels in crystalline Precambrian and Palaeozoic rocks.

3.5.3 Filling materials and gouge

Fault gouge normally constitutes a very complex material both in regard to mineralisation and physical properties. Although the *filling material* or gouge in a fault may only be some centimetres wide, the overall zone affected by open or altered joints may be some metres wide, and the length of the fault zone may be from hundred metres to more than a kilometre.

Table 3.3 gives an overview of the major types of materials that can be found in weakness zones and faults, and the potential behaviour of these materials in excavations. The basic division is made according to the mineral or material that dominates the properties of the filling. This is not necessarily the most abundant material.

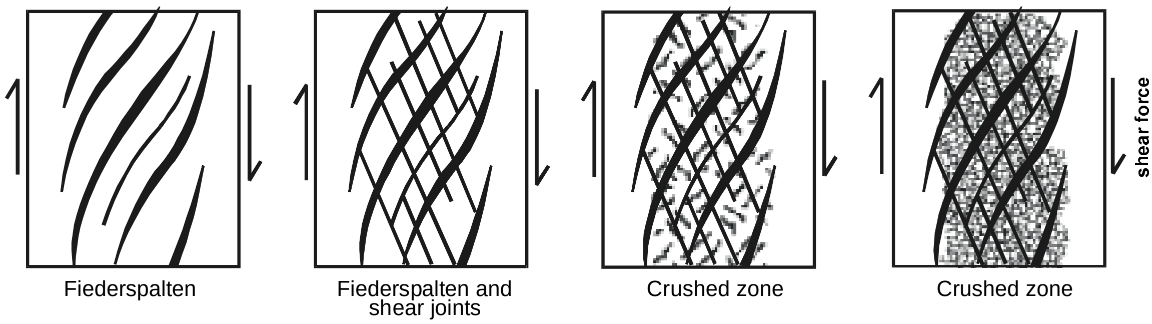


Figure 3.13 Principles in the development of crushed zones. (from Selmer-Olsen, 1964)

3.5.3.1 Swelling clay

Swelling clays connected with weakness zones may occur in two different ways:
1. As fillings strictly associated with joints, veins, fractures or faults.
2. As rock-forming minerals in altered rocks. This type occurs less often.

Swelling materials may be found in only some of the faults or joint systems in a geological province; younger or older systems crossing the same area may not contain swelling clays. Usually rock powder and fragments are found together with the swelling minerals. Other secondary minerals such as calcite, quartz, chlorite, talc, zeolites, kaolinite and hydromica/illite may also be associated with the swelling clay minerals of the fillings.

The type of cation present greatly affects the degree of swelling. Na^+, for example, will cause a high degree of swelling while Ca^{2+} will generally cause a low degree of swelling.

The most important factors affecting the degree of swelling and softening in a zone are:
- Amount and type of swelling minerals.
- Amount and type of mobile cations.
- Degree of consolidation of the material in the zone.
- Access to water.
- Degree of unloading after excavation.

In addition to the factors listed above, the amounts, types and shear properties of the other fine grained, loose materials in the zone will influence the swelling properties and the behaviour of the zone. Important in this connection is the presence of calcite and other minerals that when dissolved, make room for the soft filling to be washed out.

Table 3.15 Measured *swelling properties* of fault clay in Norwegian tunnels. (from Broch and Nilsen, 1996)

Project	**Material < 20 μm** %	**Free swelling** %	**Swelling pressure** MPa
Rana hydropower plant	-	200	1.04
Rafnes water transfer tunnel	23	232	1.05
Sira-Kvina (Duge) hydropower plant	2	170	1.76
Nye Osa hydropower plant	12	140	0.30
Åbjøra (Trøndelag) hydropower plant	13	210	0.89
Øvre Otra (Ormsa) hydropower plant	5	195	0.95
Hjartøy sub-sea tunnel	10	450	0.95
Ormsetfoss hydropower plant (transfer tunnel)	10	167	0.62
Ormsetfoss hydropower plant (headrace tunnel)	46	125	0.34

No swelling will take place under ***dry*** tunnelling conditions. Since possible water in the swelling zone, and groundwater from the rock masses entering the zone will evaporate, it is often nearly impossible to distinguish between non-swelling and swelling materials solely by inspection. Washing the tunnel walls may make detection of the presence of swelling materials easier, especially if the swelling materials are given the opportunity to absorb water.

Under ***wet*** tunnelling conditions it is easier to detect swelling materials. Here, the swelling will squeeze gouge material some mm out of the walls. This feature is often best seen near the lower part of the tunnel wall where clay can more easily absorb water from the ditch.

3.5.4 Description of weakness zones

The main parts of a typical fault or shear zone consist of (see also Figure 3.12):

- The central part. This is where most movement has taken place and resulted in intense jointing or crushing of the rock, with possible hydrothermal activity and deposition of minerals with/without alteration.
- The transition part, i.e., the area disturbed by movement with a higher degree of jointing than in the adjacent rocks. Alteration may also have taken place.
- The surrounding or adjacent rock masses that have been little influenced by movement, but which sometimes are penetrated by seams and minor faults that branch out from the zone.

Larger faults and weakness zones should be mapped and described as independent structural regions. Core drilling from the surface or probing ahead of an advancing tunnel face are the most effective means of collecting information about a zone before it is penetrated. However, it can be difficult to obtain enough data to fully describe the structure, especially in the case of core loss.

When the zone has been encountered during excavation, its composition and structure may be better studied together with its orientation and size (thickness). An adequate description of these features is very important for decisions concerning excavation procedure(s) and for design of the appropriate rock support. Where time is available, descriptions can be backed up by laboratory tests to measure the properties of important features, such as the swelling pressure of clays, etc.

In addition to the orientation and thickness of the zone, the following features should be included in a description of faults:

- Joint and seam characteristics. Filling or gouge type and properties.

- Block sizes and shapes.
- The types of rocks or minerals and their possible alteration.
- The composition of the rock masses in the transition zone between the zone and the adjacent rocks.

For important weakness zones, idealised sketches showing estimates of the principal dimensions may be very helpful. A verbal description of these features should always be given so that the extent and character of the discontinuity is communicated (ISRM, 1975/1978).

4 Rock masses

Rock mass is "rock penetrated by discontinuities", i.e., the structural material which is being excavated and in which the underground opening is located. *Ground* is the in situ rock mass subjected to stresses, groundwater, and other external factors.

4.1 Structure and composition

According to Kirkaldie (1988), the following 28 parameters in rock masses may influence the strength, deformability, permeability or stability behaviour of rock masses:

- 10 rock material properties,
- 10 properties of discontinuities, and
- 8 hydrogeological properties.

It is, of course, difficult or impossible to include all these in a characterisation of the rock mass. Therefore, it is necessary to select only a certain, limited number of representative parameters. For this purpose several classification and design systems have been developed, of which some are described in Section 9.3.

Discontinuities ranging in lengths from less than a decimetre to several kilometres divide the bedrock into units, volumes or blocks of different scales:

1. The regional pattern or first order fault blocks defined by the larger weakness zones or faults.
2. The second order blocks formed by *singularities*, i.e., small weakness zones or *seams.*
3. The third order blocks formed by normal joints.
4. The smallest pattern of interest for engineering purposes, defined by small discontinuities such as bedding or schistosity partings.
5. Small fragments or grains in the rock, defined by microcracks. These discontinuities are, however, usually considered a rock property and therefore generally included in the strength characterisation of the rock material.

Based on this it has been found useful for engineering geological and design purposes to divide the ground into:

a) The "detailed jointing" formed mainly by the third and fourth order blocks (see Section 3.3), and
b) The "coarse pattern of weakness zones" formed by the first order blocks or units by faults and weakness zones.

4.2 Continuous and discontinuous rock masses

The type and size of an underground opening determines the volume of the rock mass influenced by the excavation. The size of the opening compared to the structure and the size of the blocks can be used to find how the ground will behave. For this the *continuity factor* given as the ratio between tunnel diameter (Dt) and block diameter (Db), CF = Dt/Db, can be used to assess whether the ground behaves as a bulk (continuous) material or discontinuous material. The limit between continuous and discontinuous is a matter of judgement. The following division has been suggested by Palmström (1995):

1. For CF = approx. 5-100 the ground is *discontinuous*. The behaviour is likely to be highly anisotropic and dominated by the properties of individual discontinuities.
2. The ground is *continuous*, behaving as a bulk material:
   - in massive rock where CF < 5, the rock properties dominate
   - in highly jointed (*particulate*) ground where CF > 100, the material behaves more like a soil.

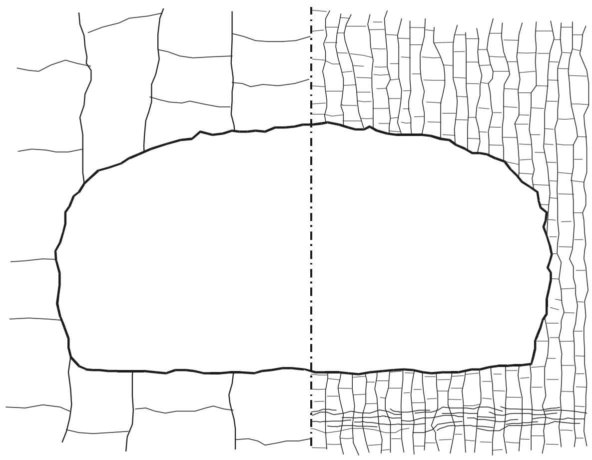


Figure 4.1 The difference between discontinuous (left) and continuous materials (revised from Barton, 1990b). Increasing the number of blocks in the tunnel surface increase the likelihood of blocks to loosen and the volume involved in a possible failure.

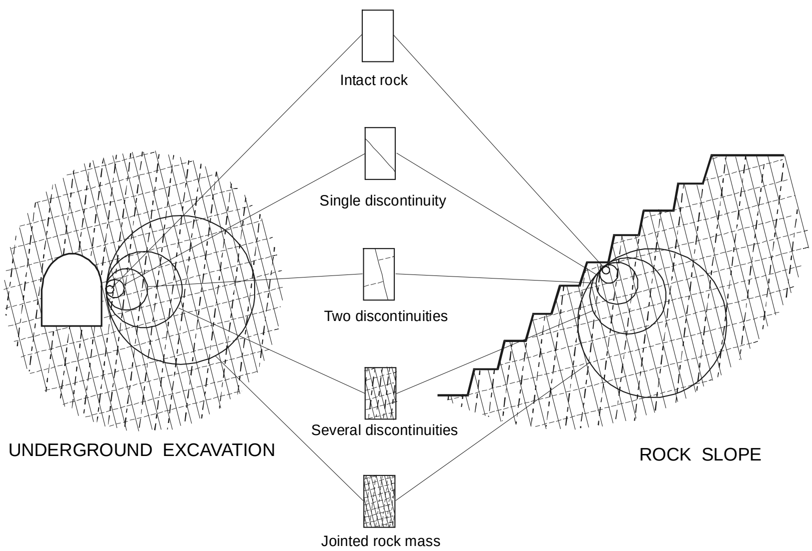


Figure 4.2 Various volumes of rock masses involved in a “sample”. Continuous rock masses are: “intact rock”, “jointed rock mass”, and possibly “several discontinuities”. (From Hoek, 1983) The tunnel shown involves a relatively moderate amount of joints, i.e., the ground is discontinuous.

As continuous or discontinuous materials behave very differently, it is important to determine which type the rock mass in question is. The most appropriate theories and methods of design can then be chosen.

4.3 The Hoek-Brown failure criterion for rock masses

The *Hoek-Brown failure criterion* provides engineers and geologists with a means of estimating the strength of jointed rock masses. Since the criterion was presented in 1980, the ratings of its constants have been adjusted in 1988, 1991 and 1992. A modified failure criterion has later been published by Hoek et al. (1992) as outlined in Section 4.3.2.

4.3.1 The original criterion

In its original form the Hoek-Brown criterion is expressed in terms of the major and the minor principal stresses at failure as

$$\sigma_1' = \sigma_3' + (m \times \sigma_c \times \sigma_3' + s \times \sigma_c^2)^{½}$$

where σ_1' is the major principal effective stress at failure.
σ_3' is the minor principal effective stress.
σ_c is the uniaxial compressive strength of the intact rock material.
s and m are empirical constants representing inherent properties of jointing conditions and rock characteristics. These constants depend on the properties of the rock and the extent to which it has been broken before being subjected to the (failure) stresses. Both constants are dimensionless and *"very approximately analogous to the angle of friction, ϕ_i', and the cohesive strength, c', of the conventional Mohr-Coulomb failure criterion"*. (Hoek, 1983)

In addition to adjustments in the ratings of the constant m, Wood (1991) and Hoek et al. (1992) have introduced the ratio m_b / m_i, where m_i represents intact rock. The constant m_b is the same as m in the original criterion.

For $\sigma_3' = 0$, the failure criterion expresses the unconfined *compressive strength of a rock mass*, given as

$$\sigma_{cm} = \sigma_c \times s^{½}$$

To determine m and s the RMR or Q classification system can be applied. Because these classification systems also include external factors such as groundwater and stresses, they do not best characterise the mechanical properties of a rock mass.

A more direct method is to use the RMi system to find the value of s as $s = JP^2$ (where JP is the jointing parameter, see Section 4.4.1).

The constant m_b can be expressed as:

a) For undisturbed rock masses $m_b = m_i \times JP^{0.64}$
b) For disturbed rock masses $m_b = m_i \times JP^{0.857}$

Replacing s and m in the failure criterion by JP and m_i, the criterion can be written as:

$$\sigma_1' = \sigma_3' + [\sigma_c \times JP^{0.64} (m_i \times \sigma_3' + \sigma_c \times JP^{1.36})]^{½}$$

The values of m_i for some rocks are given in Table 2.8.

4.3.2 The modified criterion

From more than 10 years of experience in using the Hoek-Brown criterion, Hoek et al. (1992) found a need to modify the criterion to the following form:

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

where m_b and *a* are constants which depend on the composition, structure and surface of the jointed rock mass.

m_b can be found from the ratio m_b/m_i in Table 4.1. This ratio varies between 0.001 in crushed rock masses with highly weathered, very smooth or filled joints to 0.7 in blocky rock masses with rough joints. In massive rock $m_b/m_i = 1$. The value of *a* varies between 0.3 and 0.65. It has its highest value for the crushed rock masses with altered, smooth joints and lowest for massive rock masses.

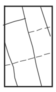
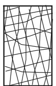
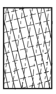
Table 4.1 Estimation of m_b/m_i and *a* based on the degree of jointing (block size) and joint characteristics (from Hoek et al., 1992).

MODIFIED HOEK-BROWN FAILURE CRITERION

$$\sigma'_1 = \sigma'_3 + \sigma_c\left(m_b \frac{\sigma'_3}{\sigma_c}\right)^a$$

σ'_1 = major principal effective stress at failure
σ'_3 = minor principal effective stress at failure
σ_c = uniaxial compressive strength of *intact* pieces in the rock mass

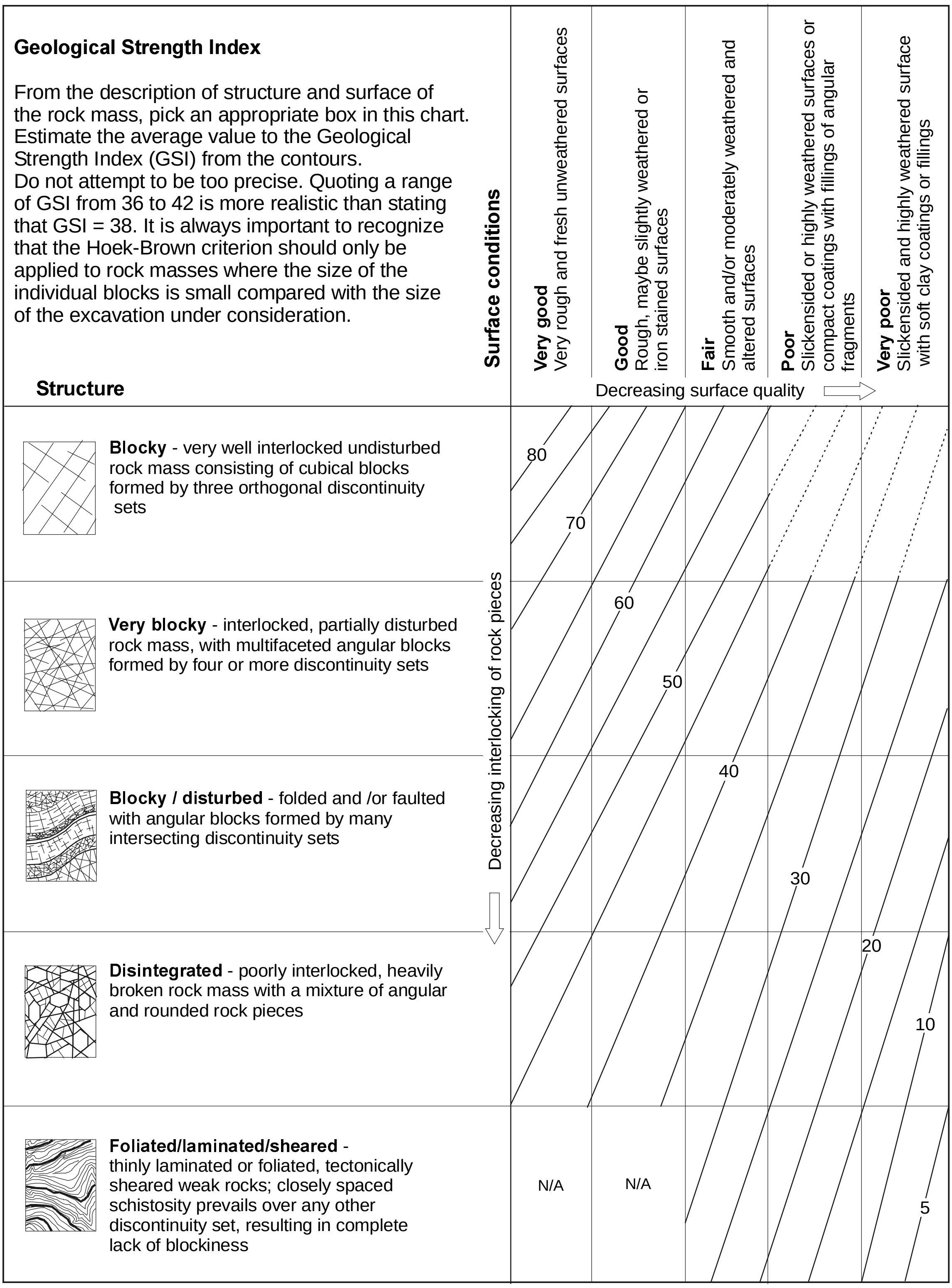
m_b and *a* are constants which depend on the condition of the rock mass

STRUCTURE		SURFACE CONDITION	VERY GOOD Unweathered, discontinuous, very tight aperture, very rough surface, no infilling	GOOD Slightly weathered, continuous, tight aperture, rough surface, iron staining to no infilling	FAIR Moderately weathered, continuous, extremely narrow, smooth surfaces, hard infilling	POOR Highly weathered, continuous, very narrow, polished / slickensided surfaces, hard infilling	VERY POOR Highly weathered, continuous, narrow polished / slickensided surfaces, soft infilling
	BLOCKY - well interlocked, undisturbed rock mass; large to very large block size	m_b/m_i a	0.7 0.3	0.5 0.35	0.3 0.4	0.1 0.45	
	VERY BLOCKY - interlocked, partially disturbed rock mass; medium block size	m_b/m_i a	0.3 0.4	0.2 0.45	0.1 0.5	0.04 0.5	
	BLOCKY / SEAMY - folded and faulted, many intersecting joints; small blocks	m_b/m_i a		0.08 0.5	0.04 0.5	0.01 0.55	0.04 0.6
	CRUSHED - poorly interlocked, highly broken rock mass; very small blocks	m_b/m_i a		0.03 0.5	0.015 0.55	0.003 0.6	0.001 0.65

4.3.3 The Geological Strength Index (GSI)

The *Geological Strength Index (GSI)*, introduced by Hoek (1994), Hoek et al. (1995) and Hoek and Brown (1998) provides a system for estimating the reduction in rock mass strength for different geological conditions as identified by field observations. The rock mass characterisation is straightforward and based on a visual impression of the rock structure in terms of blockiness, and the surface condition of the discontinuities indicated by joint roughness and alteration, see Table 4.2. The combination of these two parameters provides a practical basis for describing a wide range of rock mass types and the value of GSI is estimated from the contours in the same table.

Table 4.2 Characterisation of rock masses on the basis of interlocking and joint alteration (from Hoek et al., 1998)



4.4 Properties of the rock mass

4.4.1 Compressive strength

The many factors involved may reduce the *strength of a rock mass* to only a fraction of the strength of the intact rock. As it is almost impossible to sample the volume needed for a full scale tests of a rock mass, the strength has to be estimated based on tests on some features involved and observation of the others. The data collected can then be combined to arrive at an estimate of the rock mass *compressive strength* applying the *RMi* (*rock mass index*) system.[4] In principle RMi is expressed as

- in blocky rock: RMi = $\sigma_c \times$ JP = $\sigma_c \times 0.2\sqrt{jC} \times Vb^D$ (D = 0.37 jC $^{-0.2}$)
- in massive rock (block volume V_b > approx. 8 m^3): RMi ≈ 0.5 σ_c.

The symbols in the expressions above are:

σ_c = The uniaxial compressive strength of intact rock, measured on 50 mm samples. Some average strength values are given in Table 2.8. For anisotropic rocks the lowest strength should be applied.

JP = $0.2\sqrt{jC} \times Vb^D$ is the jointing parameter, incorporating the main joint features in the rock mass, such as:

- The *block volume* (Vb), measured in m^3. It is a measure of the degree of jointing (the amount of joints in the rock mass). Vb can be measured directly in the field, or from various joint observations as further described in Section 3.4.
- The *joint condition factor* (jC), which includes the following important joint factors:
  - the *joint size and continuity factor* (jL),
  - the *joint roughness factor* (jR), representing the smoothness and waviness of the joint
  - the *joint alteration factor* (jA), representing the character of the joint wall, i.e., the presence of coating or weathering and possible filling.
    Its value can be found from jC = jL × jR / jA using ratings for the three factors in Table 4.3.

The value of JP can also be found using the lower chart in Figure 9.13, which require input of the block volume (Vb) and the joint condition factor (jC).

For the most frequent or common joint characteristics (where jC = 1.75) the expression for JP is simplified to:

JP = 0.25 $\sigma_c \sqrt[3]{Vb}$ = 0.25 σ_c Db (where Db is the equivalent diameter of the block)

4.4.2 Shear strength

The *shear strength* of rock masses can be estimated from Mohr-Coulomb equation:

$$\tau_m = c_m + \sigma_n \times \text{Tan}\ \phi_m$$

where the two shear strength parameters are:

c_m = cohesion of the rock mass
ϕ_m = friction angle of the rock mass

[4] Application of the RMi for estimates of the deformation modulus of rock masses is given in Section 4.4.3 for estimating rock support in Section 9.3.3.

Table 4.3 Ratings of the parameters applied in the joint condition factor (jC) (modified from Palmström, 1995).

UNIAXIAL COMPRESSIVE STRENGTH (σ_c) of intact rock	**value** (in MPa) found from laboratory tests (or estimated from handbook tables etc.)
BLOCK VOLUME (Vb)	**value** (in m³) measured at site (from observations or from bore hole cores)

JOINT CONDITION FACTOR jC (jC = jR x jL / jA)

JOINT ROUGHNESS FACTOR (jR) (The ratings in **bold** are similar to Jr in the Q-system)		*Large scale waviness*				
		Planar	Slightly undulating	Undulating	Strongly undulating	Stepped or interlocking
Small scale smoothness of joint surface	Very rough	2	3	4	6	6
	Rough	**1.5**	2	**3**	4.5	6
	Smooth	**1**	1.5	**2**	3	4
	Polished or slickensided *	**0.5**	1	**1.5**	2	3
	For filled joints jR = 1 For irregular joints a rating of jR = 6 is suggested					

* For slickensided surfaces the ratings given cover possible movement along the lineations. (For movements across lineation, rough or very rough rating should be applied for the surface)

JOINT ALTERATION FACTOR (jA) (the ratings are based on Ja in the Q-system)					
Contact between joint walls	CLEAN JOINTS:	Healed or welded joints	filling of quartz, epidote, etc.		jA = 0.75
		Fresh joint walls	no coating or filling, except from staining (rust)		1
		Altered joint walls	- one grade higher alteration than the rock		2
			- two grades higher alteration than the rock		4
	COATING or THIN FILLING OF:	Frictional materials	sand, silt calcite, etc. (non-softening)		3
		Cohesive materials	clay, chlorite, talc, etc.		4
Some or no wall contact	FILLED JOINTS:			Thin filling (5 mm)	Thick filling
		Frictional materials	sand, silt calcite, etc. (non-softening)	jA = 4	8
		Hard, cohesive materials	clay, chlorite, talc, etc.	6	6 - 10
		Soft, cohesive materials	clay, chlorite, etc.	8	12
		Swelling clay materials		8 – 12	13 – 20

JOINT SIZE FACTOR (jL)		Continuous joints	Discont. joints **
Bedding or foliation partings	length < 1 m	jL = 3	jL = 6
Joints	with length 0.1 - 1 m	2	4
	with length 1 - 10 m	1	2
	with length 10 - 30 m	0.75	1.5
(Filled) joint, seam or shear *	length > 30 m	0.5	1

* Often a singularity (special feature) and should in these cases be treated separately. ** Discontinuous joints end in massive rock.

As for the compressive strength, the two shear strength parameters can be estimated from observations and measurement of rock mass parameters. In addition, information is needed on the stresses at the location of interest. Basically, two alternative approaches as described in the following may be applied here.

4.4.2.1 Estimation based on the Hoek-Brown failure criterion for rock masses

Applying the original Hoek-Brown empirical failure criterion for rock masses, the failure envelope is:

$$\tau = \left(\text{Cot}\,\phi_i\text{'-Cos}\,\phi_i\text{'}\right)\left(m\frac{\sigma_c}{8}\right)$$

where τ = the shear stress at failure, and
ϕ_i' = the instantaneous friction angle.

The value of the *instantaneous friction angle* is given by:

$$\phi_i' = \text{Arctan}\,[4h\ \text{Cos}^{2}(\pi/6 + 1/3\ \text{Arcsin}\ h^{-3/2}) - 1]^{-1/2}$$

$$\phi_i' = \text{Arc}\tan\left[4h\ \text{Cos}^2\left(\frac{\pi}{6}+\frac{1}{3}\text{Arc}\sin h^{-3/2}\right)-1\right]^{-1/2}$$

where σ' = the effective stress

$h = 1 + 16(m \times \sigma' + s \times \sigma_c)/3m^2\sigma_c$

$m = m_i \times JP^{0.64}$ (for undisturbed rock masses); $m = m_i \times JP^{0.857}$ (for disturbed rock masses)

the value of m_i can be found from Table 2.8

$s = JP^2$ (JP = jointing parameter, see Section 4.4.1)

The instantaneous *cohesive strength* is found as

$$c_i' = \tau - \sigma'\ \text{Tan}\ \phi_i'$$

Note that the Hoek-Brown failure criterion is only valid for continuous rock masses.

4.4.2.2 Estimation based on the geological strength index (GSI)

By using the GSI system the angle of internal friction (ϕ) and cohesion (c), can be found from the curves in Figures 4.3 and 4.4. The value of the material constant (m_i) can be found from laboratory testing, or estimated from Table 2.8 or from published tables.

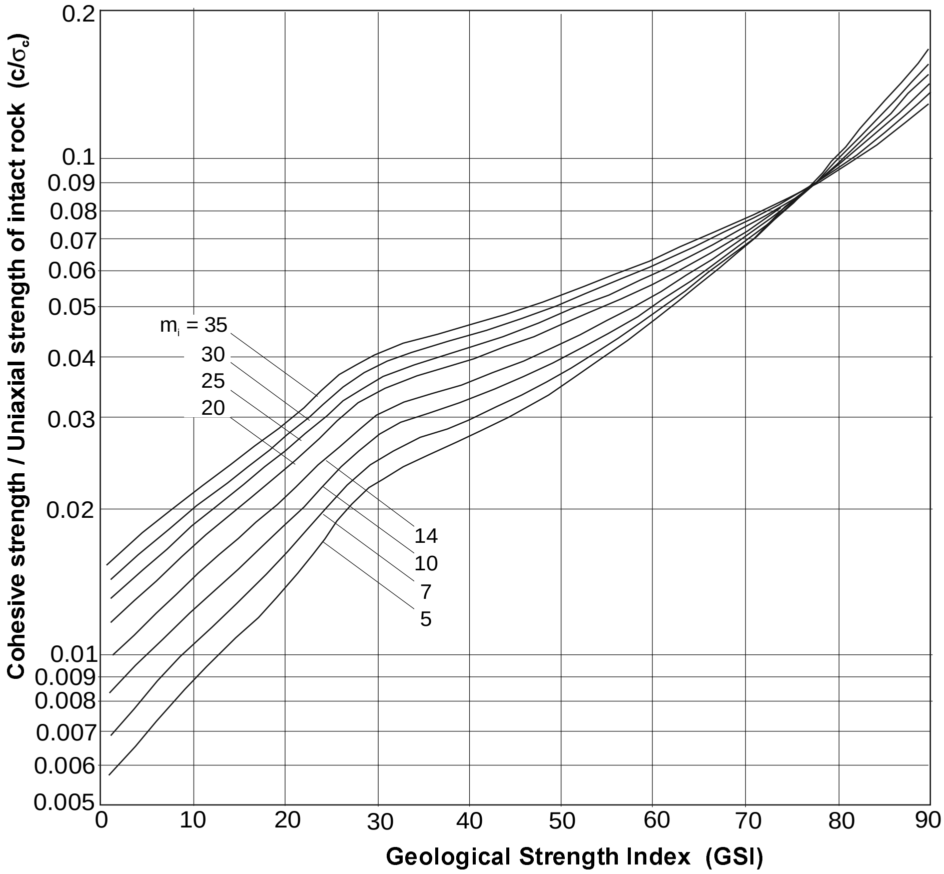


Figure 4.3 Relationship between cohesive strength (c) and GSI.

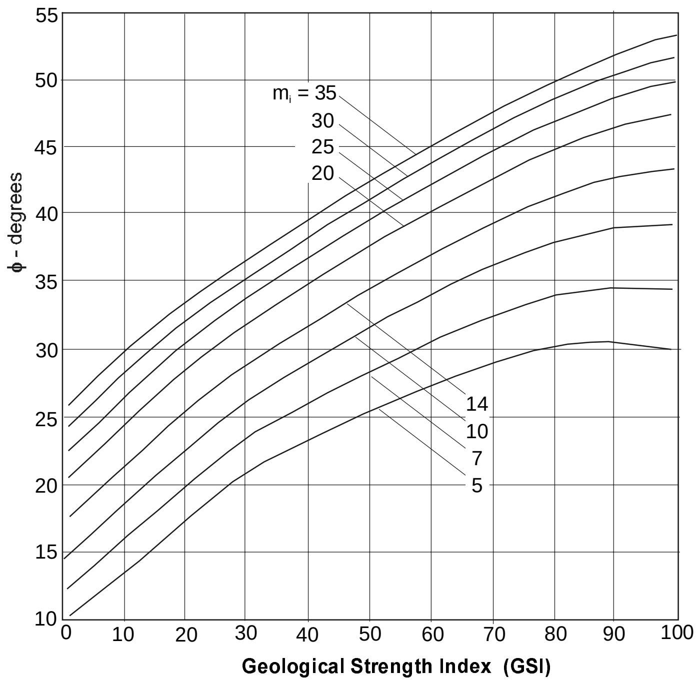


Figure 4.4 Relationship between friction angle (ϕ) and GSI.

4.4.3 E-modulus

The static *modulus of deformation of a rock mass* is the geomechanical parameter that best describes the mechanical behaviour of rock mass. All direct measurements of this parameter are time-consuming and imply notable cost and operation difficulties. The different procedures used for direct measurement (plate bearing, dilatometer test, flatjack test, hydraulic chamber, etc.) provide values that often differ from one another by as much as 100%. This is inevitable, not least due to the fact that the rock mass volume concerned differs from one test to another. Also, the parameter is notably sensitive to a scale effect because of the discontinuities.

Because of such difficulties, the value of the modulus of deformation is often estimated from observations of relevant parameters that can be easily acquired at a low cost. These parameters are then applied in approximate equations, for example:

$E_m = 2RMR - 100$ (GPa) for RMR > 50 (see Figure 4.5) [Bieniawski, 1978]

$E_m = 10^{(RMR - 10)/40}$ (GPa) for RMR < 50 (see Figure 4.5) [Serafim and Pereira, 1983]

$E_m = 25 \log_{10} Q$ (GPa) for Q > 1 [Grimstad and Barton, 1993]

$E_m = 5.6 \, RMi^{0.375}$ (GPa) for RMi > 0.1 (see Section 4.4.1) [Palmström, 1995]

$Em = \sqrt{\frac{\sigma_c}{100}} \times 10^{\left(\frac{GSI-10}{40}\right)}$ (GPa) for $\sigma_c < 100$ MPa (see Section 4.3.3) [Hoek and Brown, 1998]

The use of more than one indirect procedure is undoubtedly preferable, so that the results obtained may be compared and reliability checked. The procedure including the RMR parameter, has probably been applied most frequently. The equation developed by Serafim and Pereira (1983) has proven to give values less than ± 15% from values measured in situ.

The indirect procedure is simple and cost-effective, especially as compared with direct procedures, although it is obvious that direct procedures should be preferred whenever time and means available allow for them.

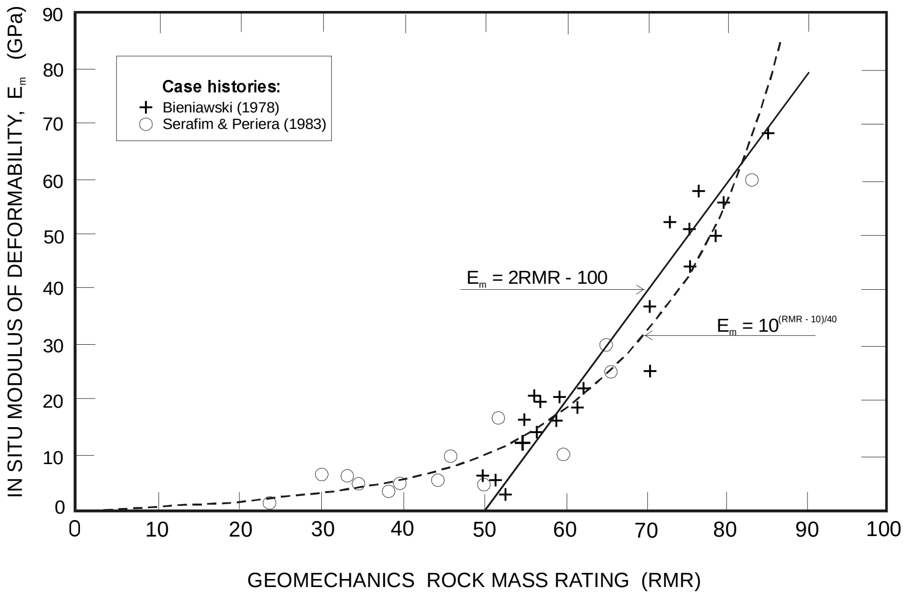


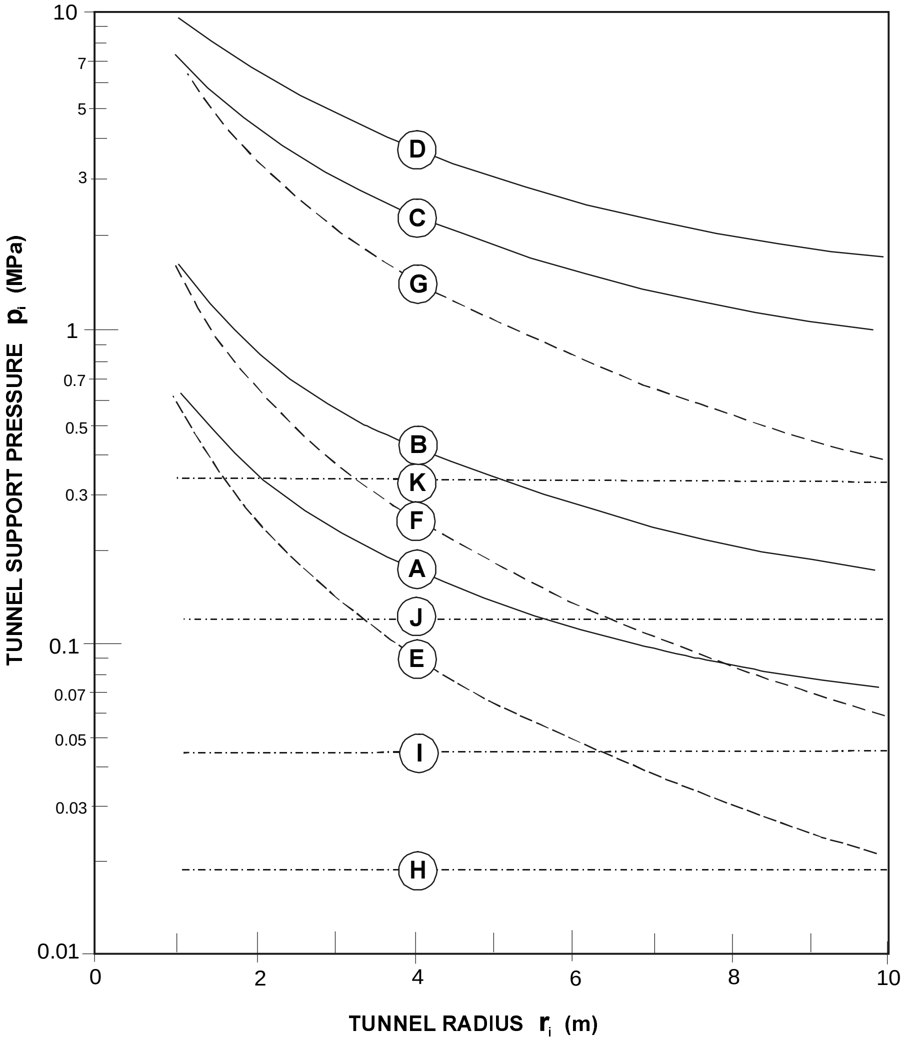
Figure 4.5 Correlation between the in situ modulus of deformation and the RMR system (from Serafim and Pereira, 1983)

4.4.4 Ground response to excavation works

Ground response interaction diagrams are well-established aids to the understanding of rock mass behaviour and support behaviour. They are limited to continuous materials, i.e., massive rock or highly jointed and crushed (particulate) rock masses. They may also be used quantitatively for designing tunnel support. For this use it is essential to be able, from field observations and assessment of the stresses and moduli, to predict the ground response curve for the actual rock mass, stress regime and tunnel geometry.

Many approaches to the calculation of *ground response curves* have been proposed. Most use closed-form solutions to problems involving simple tunnel geometry and hydrostatic in situ stresses, but some use numerical methods for more complex excavation geometry and stress fields. However, with improved knowledge of the engineering behaviour of rock masses and the use of computers it is now possible to incorporate more complex and realistic models of rock mass behaviour into the solutions.

A basis for determining reaction to support is given in Figure 4.6, and a typical ground response curve with the effects of various support methods is shown in Figure 4.7.



	Type of support			Design	Strength	
A B	Shotcrete			50 mm thick 50 mm thick	$\sigma_{c\ con}$ = 14 MPa after 1 day $\sigma_{c\ con}$ = 35 MPa after 28 days	
C D	Concrete lining			300 mm thick 500 mm thick	$\sigma_{c\ con}$ = 35 MPa after 28 days $\sigma_{c\ con}$ = 35 MPa after 28 days	
E F G	Steel sets, blocked $2\theta = 22\frac{1}{2}{}^{o}$		(6 I 12) (8 I 12) (12 I 12)	spaced 2 m spaced 1.5 m spaced 1 m	Yield strength	σ_{ys} = 248 MPa σ_{ys} = 248 MPa σ_{ys} = 248 MPa
H I J K	Rock bolts	16 mm 19 mm 25 mm 34 mm	mechanical anchor mechanical anchor mechanical anchor resin anchored	spaced 2.5 m spaced 2 m spaced 1.5 m spaced 1 m	Bolt capacity	Tbf = 0.11 MN Tbf = 0.18 MN Tbf = 0.267 MN Tbf = 0.345 MN

Figure 4.6 Tunnel support pressures for various systems and tunnel sizes (from Hoek and Brown, 1980).

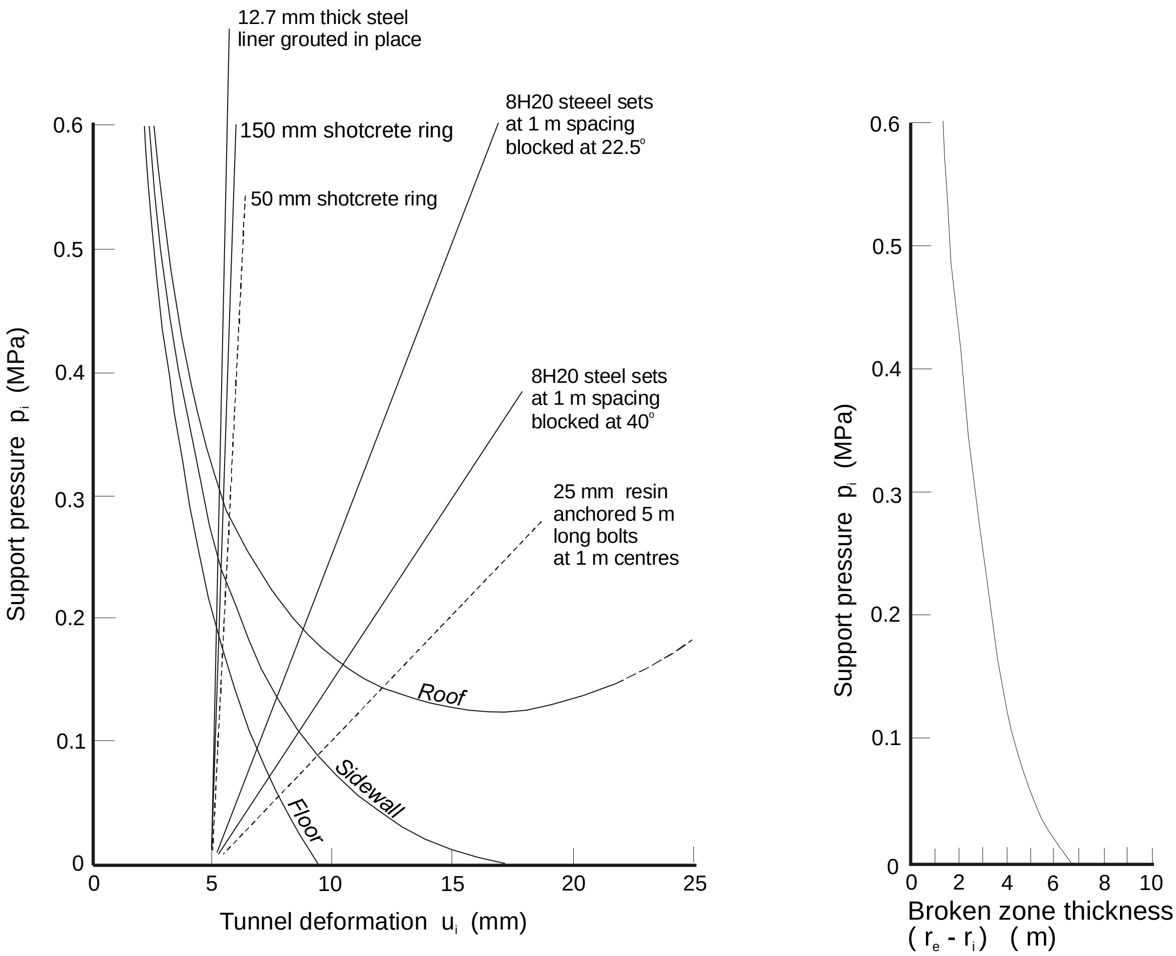


Figure 4.7 Example of a ground response curve in a 3.3 m diameter tunnel in mudstone showing the effect of various support systems given as support curves. The thickness of broken zone is also indicated (from Hoek and Brown, 1980).

4.5 Zoning of rock masses into geotechnical units

To facilitate the characterisation of the variation of rock masses within a region or along a borehole, it is often necessary and convenient to define a number of *structural regions*, wherein the rock and joints have similar composition. Each part selected can then be considered and treated individually for its particular characteristics. ISRM (1980) have applied the term *zoning of rock masses* into structural regions in their proposed method for basic geotechnical description (BGD) of rock masses. Designation of a structural region implies that the detailed jointing and the rock properties within the region selected are similar, assuming that the individual joint sets have similar characteristics.

A zone or geotechnical unit may include differing volumes of rock masses, such as interbedded layers of sedimentary or volcanic formations exhibiting the same geotechnical characteristics (ISRM, 1980). In the case of rock masses, which vary continuously from place to place, for example due to weathering, ISRM advises delineating zone boundaries in such a way that the properties of each zone may be considered relatively uniform. Generally, it is found that structural regions of similar jointing will juxtapose at major geological structures. The boundaries of a zone will therefore often be defined by faults, dykes, rock boundaries or jointing differences.

An example of ground zoning is shown in Figure 4.8.

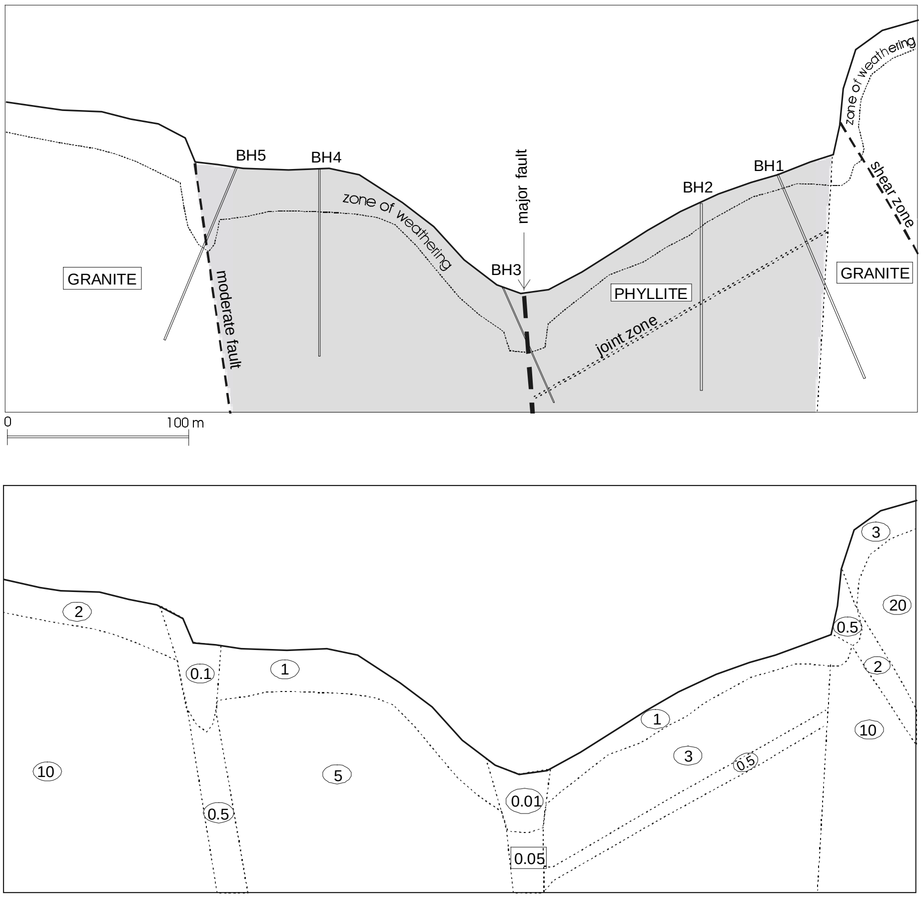


Figure 4.8 Simplified example of zoning into rock classes in a profile. Upper figure shows the geological features, the lower shows the zoning of the rock masses. The numbers refer to RMi values.

4.6 Description of rock masses

A concise *description of rock masses* is important for the exchange of engineering geological data and to inform other people involved. As most of the input parameters in rock engineering and rock mechanics are the results of observations, errors may be introduced from poorly defined descriptions and methods for data collection.

A *numerical description* of rock masses can be made using the RMi characterisation system described in Section 4.4.1 Although the RMR and Q classification systems described in Section 9.3 are worked out for rock support estimate, they are also often used for rock mass descriptions.

The *verbal description* included in the characterisation should include a description of the composition and structure of the rock mass with special emphasis on parameters of importance for engineering properties. The rock masses should be grouped in such a way that the parameters of most universal concern, are emphasised. At the same time the number of parameters should be kept to a practical minimum. The main features are:

- The rock: name or type, structure, orientation, appearance and uniaxial compressive strength
- The jointing: joint characteristics (orientation, roughness, alteration and size), and block size or degree of jointing.

For many rocks the name of the rock, its homogeneity and continuity can be established by visual observation in the field. As alteration and weathering with deterioration of the rock material may have a significant effect in reducing the strength and deformation properties, the description and characterisation should pay particular attention to such features. The description of the rock could also contain information on texture, colour, lustre, etc., which can provide information for a better understanding of the rock conditions as well as the jointing.

For the jointing, the variation in block sizes and their shape, possible coating of joint walls or filling is additionally of particular significance.

Some of the terms, which may be applied to rock masses, are outlined in Table 4.4. These terms may be useful in assessing numerical values of the actual parameters, as most of them, except some terms for rocks, are connected to ratings given in this book.

Table 4.4 Terms and features that may be useful in a rock mass description.

Rock Material					
Rock type	**Appearance**	**Strength**	**Weathering or alteration**	**Anisotropy or structure**	**Orientation**
Geological name	Grain size Grain shape Colour Lustre	Extremely weak Very weak Weak Medium strong Strong Very strong Extremely strong	Fresh Slightly weathered Moderately weathered Strongly weathered Completely weathered or Discoloured Disintegrated Decomposed	-Bedded -Schistose -Foliated Homogeneous Striped Veined Banded Layered Folded Porous	Strike/dip Fold axis/plunge
Joint Conditions					
Smoothness	**Waviness**	**Alteration**	**Separation**	**Size (length)***	**Joint type**
Slickensided Polished Smooth Slightly rough Rough Very rough Interlocked	Planar Slightly undulating Strongly undulating Stepped Interlocked	Welded or Healed Fresh Weathering grade Type of coating Type of filling	Closed Small separation Moderate No wall contact	Very small Small medium Large Very large	Crack Parting Joint Seam** Filled joint** +Fracture +Fissure
Block size, Jointing density or Degree of jointing					
Block volume	**Term (for jointed)**	**Block type**	**Alternative description instead of block volume using: Joint spacing,**	**or Joint frequency**	
Extremely small Very small Small Moderate Large Very large Extremely large	Slightly Moderately Strongly Extremely	Compact or Blocky Long Flat Long and flat	Very small Small Moderate Large Very large	Very low Low Moderate High Very high	

* In addition, *continuity* of the joint (i.e., continuous or discontinuous) should be given
** In addition, information about the type of *filling material* should be given
\+ must be defined
\- additional characteristic term (small, moderate, large, etc.) should preferably be given

5 Rock stresses

Rock stresses are the stresses (force per unit area) which exist within the rock mass. They are characterised by their directions and magnitudes. According to convention, compressive stress is positive and tensile stress is negative. The *principal stresses* are the normal stresses on planes with no shear stress, and are referred to as:

- *major principal stress*, σ_1 (highest)
- *intermediate principal stress*, σ_2
- *minor principal stress,* σ_3 (lowest)

The *in situ stresses* at the location of an underground excavation may have great impact on stability where the stresses set up around the excavation exceed the strength of the rock mass. However, not only high stresses may cause instability. A low stress level may reduce the stability in jointed rock masses because of low normal stresses on joints.

When the in situ stresses and the geometry of the opening are known, it is possible to evaluate the magnitudes and the directions of the stresses surrounding the opening. If the rock mass properties are known, it is possible also to analyse potential stability and leakage problems caused by stresses, the need for rock support and the possibilities of optimising the excavation geometry.

5.1 Origin and magnitude of rock stresses

Generally, the stresses surrounding underground openings are defined by:

1) The stress situation prior to excavation (the "virgin" stresses).
2) The geometry of the opening.

The *virgin rock stress* is the result of the following components:

- Gravitational stresses.
- Topographic stresses.
- Tectonic stresses.
- Residual stresses.

In the following, the nature and the importance of each of these components will be briefly discussed.

5.1.1 Gravitational stresses

The *gravitational stress* is a result of gravity alone. When the surface is horizontal, the vertical gravitational stress at a depth z is:

$$\sigma_z = \rho \times g \times z$$

where $\rho \times g$ = specific gravity of the rock.

The magnitude of the total vertical stress may be identical with the magnitude of the gravitational vertical component. However, in several cases, and particularly at great depths, there are also considerable deviations from this as illustrated in Figure 5.1.

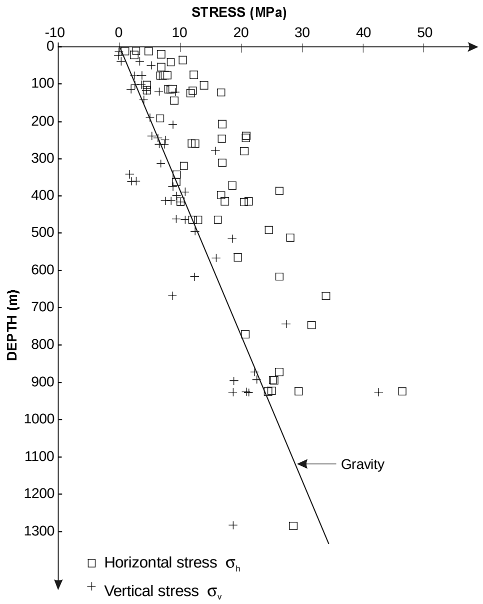


Figure 5.1. Vertical and horizontal rock stresses in Scandinavia as functions of depth below the rock surface (after Hanssen, 1988).

In an elastic rock mass with a *Poisson's ratio* of ν, the horizontal stresses induced by gravity are:

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu}\,\sigma_z$$

For a Poisson's ratio $\nu = 0.25$, which is fairly common for rock masses, this means that the horizontal stress induced by gravity is 1/3 of the vertical stress. However, the horizontal stress induced by gravity normally constitutes only a small part of the total horizontal stress.

In jointed rock masses, this idealised elastic relation does not correspond very well with reality.

5.1.2 Topographic stresses

When the surface is not horizontal, the topography will have a considerable influence on the rock stress situation. Stresses, which are caused by topographic effects, are generally referred to as topographically induced stresses or simply *topographic stresses*.

In high valley sides, where underground excavations often are located, the stress situation will be totally dominated by the topographic effects. Near the surface in such cases, σ_1 will be more or less parallel to the slope of the valley, and σ_3 will be perpendicular, approximately, to the slope as shown by the example in Figure 5.2. The example, which is based on finite element analysis (see Chapter 12), also illustrates the typical stress anisotropy of high valley sides.

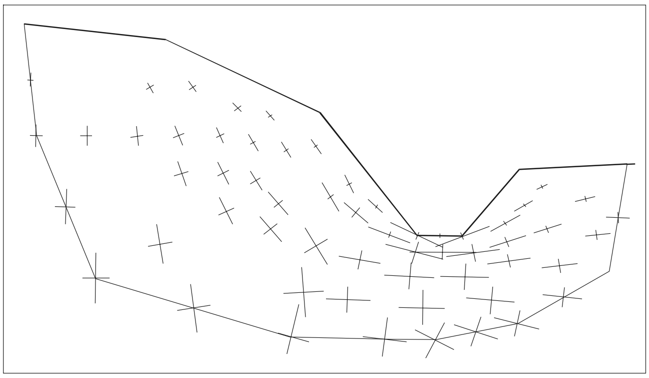


Figure 5.2 Magnitudes and directions of the major and minor principal stresses in a valley side as computed by a finite element analysis. The length and directions of the crosses indicate the magnitude and directions of the major and minor principal stresses (from Nilsen, 1979).

5.1.3 Tectonic stresses

Tectonic stresses are responsible for incidents such as faulting and folding. The main cause of tectonic stress is *plate tectonics*, the drifting and tectonic activity along the margins of the about 20 rigid plates that constitute the earth's outer shell.

Due to the existence of tectonic stresses, the total horizontal stress is in most cases much higher than the horizontal stress induced by gravitation. This is particularly the case at shallow and moderate depths as illustrated by Figure 5.3, which summarises the results of a considerable number of rock stress measurements from various parts of the world. The figure also illustrates that the relationship between horizontal and vertical stress may vary within very wide ranges, and hence underlines the importance of measuring the rock stresses in each individual case.

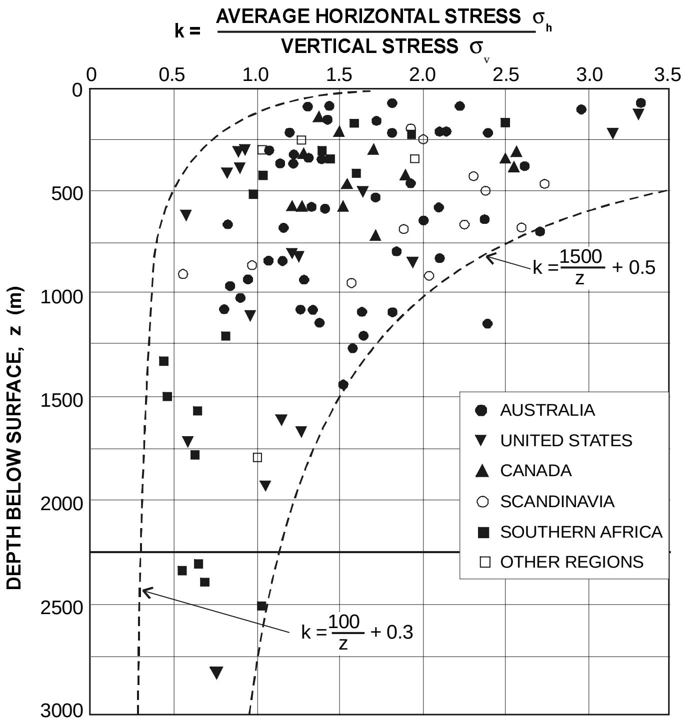


Figure 5.3 Variations of ratio of average horizontal stress to vertical stress with depth below surface (from Hoek and Brown, 1980).

It is important to be aware that the *k-value* discussed here, and shown in Figure 5.3, defines the ratio between stresses within the rock mass, and thus is not the same as the *K-value* used as boundary load in numerical models (see Section 12.3). As indicated by Figure 5.3, the k-value of the rock mass may well be higher than 3.

5.1.4 Residual stresses

Residual stresses, also referred to as *remnant stresses*, are stresses that have been locked into the rock material during earlier stages of its geological history. Stress caused by contraction during the cooling of a rock melt (magma) is a relevant example of this category. Vertical stresses that are abnormally high are often explained as being caused by residual stress.

5.2 Stresses surrounding rock excavations

Excavation disturbs the initial, virgin stress condition. The stresses set up around the opening depend upon the magnitudes and directions of the *principal stresses* and the geometry of the opening. They can be measured where a stress cell can be placed, or estimated from topography, overburden and knowledge of the general stress situation in the region.

When analysing the effect of rock stresses, the stress situation close to the contour of the excavation is of particular interest. In this connection the following important issues will be discussed here:

- Stresses surrounding circular openings.
- Stresses near corners.

5.2.1 Circular openings

5.2.1.1 Idealised conditions

The simplest case is represented by the following conditions:

- Homogeneous and isotropic, elastic material.
- *Isostatic* virgin stresses ($\sigma_1 = \sigma_2 = \sigma_3 = \sigma$).

The stresses around the opening with radius (r_i) will be the following, depending on the distance (r) from the circle centre:

radial stresses $$\sigma_r = \sigma\left(1 - \frac{r_i^2}{r^2}\right)$$

tangential stresses $$\sigma_\theta = \sigma\left(1 + \frac{r_i^2}{r^2}\right)$$

In Figure 5.4 these equations are shown graphically. Particularly important to notice is the rapid increase in tangential stress close to the contour. Generally in a case like this, a tangential stress with a magnitude of twice the magnitude of the isostatic stress will be induced all around the periphery.

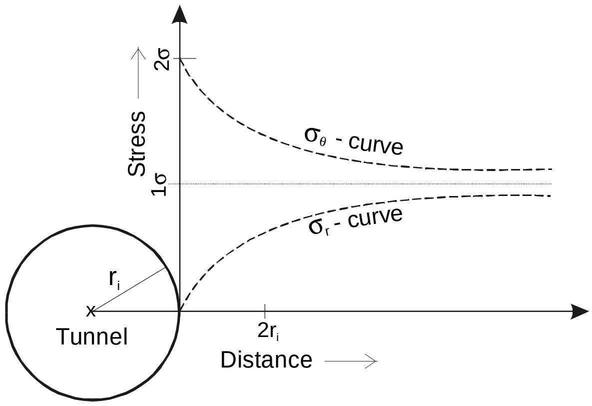


Figure 5.4. Tangential and radial stresses (σ_θ and σ_r, respectively) surrounding a circular opening in an isostatic stress field ($\sigma_1 = \sigma_2 = \sigma_3 = \sigma$).

5.2.1.2 Anisotropic conditions

However, as discussed in Section 5.1, the virgin stress situation is often highly anisotropic. Therefore the tangential stress will vary around the periphery of a circular opening.

For an anisotropic stress condition, the so-called *Kirsch's equations* may be used for evaluating the tangential stresses. According to Kirsch the tangential stress will reach its maximum value ($\sigma_{\theta(max.)}$) where the σ_1-direction is a tangent to the contour, and its minimum value ($\sigma_{\theta(min.)}$) where the σ_3-direction is a tangent. The actual values will be:

$$\sigma_{\theta(max)} = 3\sigma_1 - \sigma_3$$
$$\sigma_{\theta(min)} = 3\sigma_3 - \sigma_1$$

The distribution of tangential stress is strongly influenced by the degree of stress anisotropy. If the stresses are very anisotropic, the minimum tangential stress may even be negative, i.e., tensional. A practical method to estimate the tangential stresses is described in Section 5.2.4.

5.2.2 Non-circular openings/sharp corners

Non-symmetrical geometry, and sharp corners in particular, will strongly influence the magnitude of the tangential stress.

When the radius of curvature is being reduced, the magnitude of the tangential stress will increase. This means, for instance, that the sharper the corner is between the wall and the roof of a cavern, the higher the stress concentration in that corner will be. In extreme cases such stress concentration may reach magnitudes of more than 10 times the major principal stress value.

In cases with benches or protruding corners the stress situation will be the opposite. Here the stabilising stresses, or the confinement, will be reduced, and very often stability problems will be the result.

The magnitude of the maximum tangential stress depends in theory on the shape of the underground opening, and not on its size. The zone of influence will, however, increase when the size is increasing. In situ rock stress measurements indicate that the stresses stabilise at a constant level at a distance from the tunnel contour corresponding to approximately half the tunnel width. This constant level corresponds to the actual virgin stress.

5.2.3 *Influence of the rock mass character*

The distribution of the tangential stress will depend much on the deformation properties of the rock mass, and hence also on the way the excavation is carried out. In carefully blasted tunnels and TBM-bored tunnels in hard rocks the *stress peak* is steep, and a distinct maximum stress value is located at the tunnel contour as shown by the solid curve in Figure 5.5. In fractured rock, and also in soft rocks, the stress peak is relatively flat, and the maximum stress value is located at some distance from the tunnel contour, as shown by the stippled curve in the figure. As a result of blasting damage the situation in most drill and blast tunnels will tend to be like this.

5.2.4 *A practical method to estimate the magnitude of the tangential stresses*

A practical method to estimate the magnitude of tangential stresses around various types of underground openings in massive rock has been developed by Hoek and Brown (1980). Based on a large number of detailed boundary element stress analyses (see Chapter 12) they have established the following correlations.

Tangential stress in roof: $\sigma_{\theta r} = (A \times k - 1)\sigma_z$
Tangential stress in wall: $\sigma_{\theta w} = (B - k)\sigma_z$

where: A and B = roof and wall factors for the various excavation shapes in Table 5.1.
k = ratio horizontal/vertical stress
σ_z = vertical virgin stress

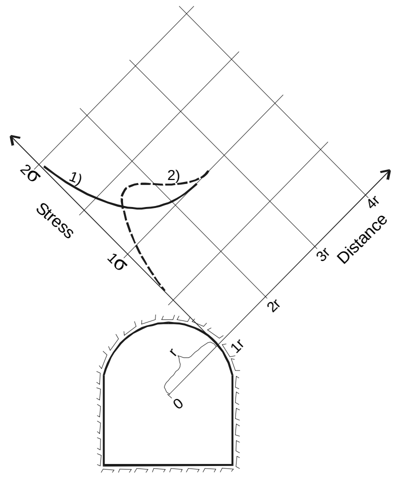


Figure 5.5. Principle sketch illustrating the concentration of tangential stress in a tunnel when 1) the contour rock is hard and undisturbed, and 2) it is fractured or soft. (The virgin stress is assumed to be isostatic, $\sigma_1 = \sigma_2 = \sigma_3 = \sigma$).

Table 5.1 Values of the factors A and B for various shapes of underground openings (from Hoek & Brown, 1980).

	Tunnel shape								
A	5.0	4.0	3.9	3.2	3.1	3.0	2.0	1.9	1.8
B	2.0	1.5	1.8	2.3	2.7	3.0	5.0	1.9	3.9

Applying these equations, approximate estimates of the stresses in the rock mass surrounding a tunnel can be found when the magnitudes of the vertical stresses and the ratio k are known. As an example, the tangential stresses around a horseshoe shaped tunnel can be considered. According to the first equation and Table 5.1, the tangential roof stress is:

$$\sigma_{\theta r} = \sigma_z (3.2k - 1)$$

In many cases, the vertical stress is identical with the magnitude of the gravitational vertical component, and the equation can be written as:

$$\sigma_{\theta r} = 0.027\ H\ (3.2k - 1)$$

where H = the overburden in metres

In Scandinavia, the value of k is often in the range 2 to 3. In such cases the tangential ***roof*** stress varies between:

$$\sigma_{\theta r} = 5.5\ \sigma_z \quad \text{and} \quad 8.6\ \sigma_z$$

or, if expressed as the depth below surface:

$$\sigma_{\theta r} = 0.14\ H \quad \text{and} \quad 0.224\ H$$

Correspondingly, the tangential ***wall*** stress can be expressed as:

$$\sigma_{\theta w} = \sigma_z(2.3 - k)$$

And, in the same ground conditions as defined above:

$$\sigma_{\theta w} = 0.008\ H \quad \text{and} \quad -0.018\ H$$

which indicates that tension stresses will occur in cases with high k-values.

5.3 Stress effects

Normally in the contour of an underground opening, there are two diametrically opposite areas of tangential stress concentration, and two areas of minimum tangential stress. When rock stresses are causing problems, the problems are normally connected to the areas of maximum tangential stress. However, if the minimum tangential stress is very low, this may also be a problem.

5.3.1 High compressive stress

In hard rocks, if the tangential stress exceeds the strength of the rock, fracturing parallel to the tunnel contour will be the result. The situation has a certain similarity to fracturing in point load testing, in which the fracture also is induced by a compressive stress in the direction of fracturing.

The fracturing process is often accompanied by strong noises from the rock, a phenomenon referred to as *heavy spalling* when the activity is violent. The term *rock burst* is reserved for major cave-ins due to high stress and with few exceptions is relevant only for deep mines. At moderate stress levels the fracturing will result in a loosening of thin rock slabs, referred to as *rock slabbing* or *spalling*. If the tangential stress is very high, spalling may be rather dramatic, and have the character of *popping* of large rock slabs with considerable force and speed.

When the stresses are very high, rock bursts may be a major threat to safety if the right type of rock support is not installed at the right moment. In such cases extensive rock support is necessary. A common result of rock spalling is an asymmetric tunnel profile, so-called "*keel-formed overbreak*". In such situations, the location of the overbreak indicates the direction of the major principal stress (as the tangent to the contour where the keel is formed).

Rock spalling is most intensive at the working face immediately after excavation. Experience shows that the most difficult area is the section 10-20 m closest to the working face. In water tunnels, spalling has been known to continue for up to 10-15 years. This long-term effect is probably caused by a combination of high stresses, a reduction of the rock strength due to water saturation, creep effects and hydraulic pressure variations.

Superficial spalling with violent ejection of fragments is referred to as *strainbursting* by Stacey (1994). The typical geometry and characteristics of a strainburst event are summarised in Figure 5.6. Strainbursts are probably the most common damage mechanism observed in civil engineering excavations and the following conclusions, according to Stacey, may be inferred from the reported strainbursting and induced fracturing around tunnels:

- Strainbursting is more likely to occur in more massive rock types than in fractured rock masses. Broch & Sørheim (1984) have shown that rockbursting activity increases with increasing strength of rock.
- Strainbursting, accordingly, is more likely to occur in a machine-excavated tunnel than in a drill and blasted tunnel.
- Strainbursting does not occur only in brittle rock types, but is likely to be more severe in brittle rock.
- High stresses need not necessarily be present for strainbursting to occur, field stress down to 15 % of the rock uniaxial strength may be sufficient.
- Spontaneous fracturing of rock in a machine-excavation environment can cause significant cutter problems.
- Strainbursting conditions may lead to a significant decrease in tunnelling progress rate.

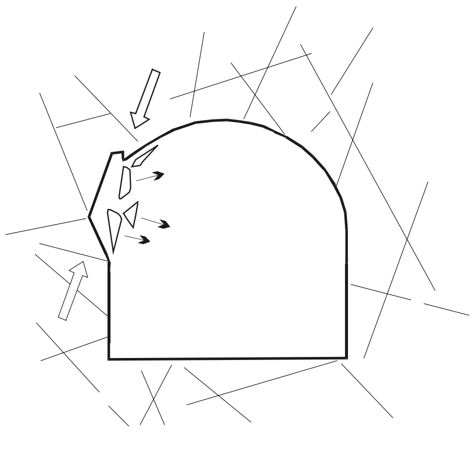


Figure 5.6 Strainburst: violent spalling resulting from localised superficial stress concentration (from Ortlepp and Stacey, 1994).

In soft rocks the stress problems will not be characterised by spalling. Because of the plastic nature of such rocks the potential problem here will be *squeezing*. In extreme cases reductions of the original tunnel diameter of several tens of centimetres due to squeezing may be experienced.

The potential effects of high compressive stress, including squeezing, are discussed in more detail in Sections 9.1 and 9.3.

5.3.2 Tensile stress

Due to its discontinuous character, a rock mass can resist very little tensile stress. Hence, even a very small tangential tensile stress may cause radial fracturing.

In most cases tensile fracturing will not have much influence on rock stability in the tunnel. For high-pressure hydropower tunnels it is, however, important to be aware that this secondary jointing and opening of existing joints may increase the possibilities of water leakage out of the tunnel.

5.3.3 The significance of geological factors

The orientation of the major principal stress relative to the directions of major joint sets and main structural features such as bedding and schistosity will have a major influence on the rock burst activity. Severe situations may occur if the schistosity runs parallel to the tunnel axis, and the major principal stress acts perpendicular to the tunnel axis and in the dip direction of the schistosity.

Along a tunnel with rock stress problems, there is always a certain variation in stresses, rock types and elastic properties, and hence also a variation in rock burst activity. Generally, there will be a concentration of stresses in stiff rocks, and a reduction in softer rocks. In gneisses, for instance, a common experience is that tunnel sections particularly rich in mica are often characterised by stress relief, while the rock burst is concentrated in more competent sections.

Major weakness zones may influence the rock stress situation considerably. As many such zones are able to transform shear stress to only a minor extent, the principal stresses will often be parallel and perpendicular to the zones. Hence, a tunnel through a major weakness zone may experience extensive rock spalling on one side of the zone, while the stresses are reduced to a moderate or low level on the other side.

5.3.4 Estimating rock load on tunnel support

Estimates of the levels of *support loads* are generally prone to uncertainty. Terzaghi (1946) presented an expression for the rock load, which in most cases has been found to give conservative results. Later, both the Q and the RMR system have presented estimates on the rock loads.

The Q-system gives some guidance concerning support pressure from the following expression:

$$p_{arch} \approx \frac{2.0}{10Jr} Q^{-1/3} \quad \text{in MPa}$$

An improved empirical fit from analyses of case records is obtained by a minor modification to the expression above, incorporating separate weighting for the number of joint sets (Jn)

$$p_{arch} \approx \frac{2Jn^{1/3} Q^{-1/3}}{30Jr} \quad \text{in MPa}$$

Support load can be determined from the RMR classification as:

$$p = \frac{100 - RMR}{100}\rho \times Dt = \rho \times h_t$$

The expression $h_t = \left[\frac{100 - RMR}{100}\right] Dt$ is the rock-load height in metres

where Dt = the tunnel width (span) in metres
ρ = the density of the rock in g/cm³

Support load as function of span according to the RMR method is presented in Figure 5.7.

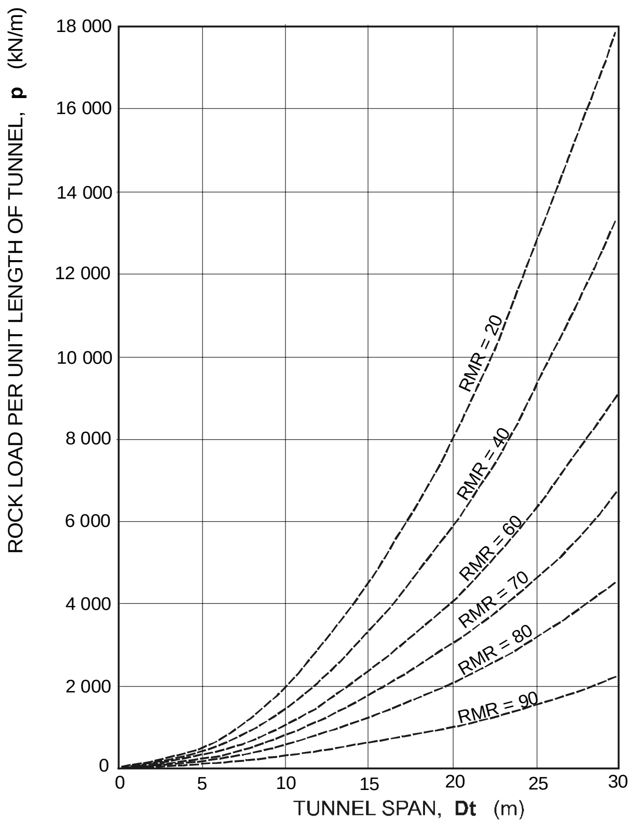


Figure 5.7 Rock load as a function of RMR-value and tunnel span (from Bieniawski, 1984).

5.4 Estimation of the effect of high stresses in brittle rocks

An old Norwegian rule of thumb used for hard rocks says that if side heights in a valley above the tunnel of 500 m or more are reached at an angle of 25° or steeper, one should always be prepared for stress induced stability problems. Although this rule represents a considerable simplification (for instance by not taking the important influence of tectonic stress into consideration), it still reflects experience from a large number of Norwegian projects located in valley sides.

A more recent study, representing a part of the Q classification system (Grimstad and Barton, 1993), predicts stress related problems according to the relationship between uniaxial compressive strength (σ_c) and major principal stress (σ_1) or, alternatively, between tangential stress (σ_θ) and σ_c, see Table 5.2. For more details, reference is made to Section 9.3.2.

Table 5.2 Prediction of stress related problems according to the Q-system (based on Grimstad & Barton, 1993).

	Consequence of stresses	σ_c / σ_1	σ_θ / σ_c
a)	In competent, massive, hard rock		
	Moderate slabbing after > 1 hr	5 – 3	0.5 - 0.65
	Slabbing and rock burst after a few minutes	3 – 2	0.65 - 1
	Heavy rock burst (strain burst) and immediate dynamic deformation	< 2	>1
b)	In squeezing rock		
	mild squeezing		1-5
	heavy squeezing		>5

Palmström (1995) has shown that rock stress problems occur where the competency factor

$$Cg = \frac{RMi}{\sigma_\theta} < 1 \quad \text{(see Section 9.3.3 and Figure 9.14).}$$

5.5 Regional stress distribution

Figure 5.8 shows a summary of horizontal stresses recorded during the more than 30 years of rock stress measurements in Norway, where Precambrian areas are particularly dominated by high tectonic horizontal stresses. This is a common trend in Scandinavia and also in many other parts of the world.

Figure 5.8 also illustrates that the rock stresses may vary considerably from place to place, and thus underlines the importance of carrying out stress measurements in each individual case.

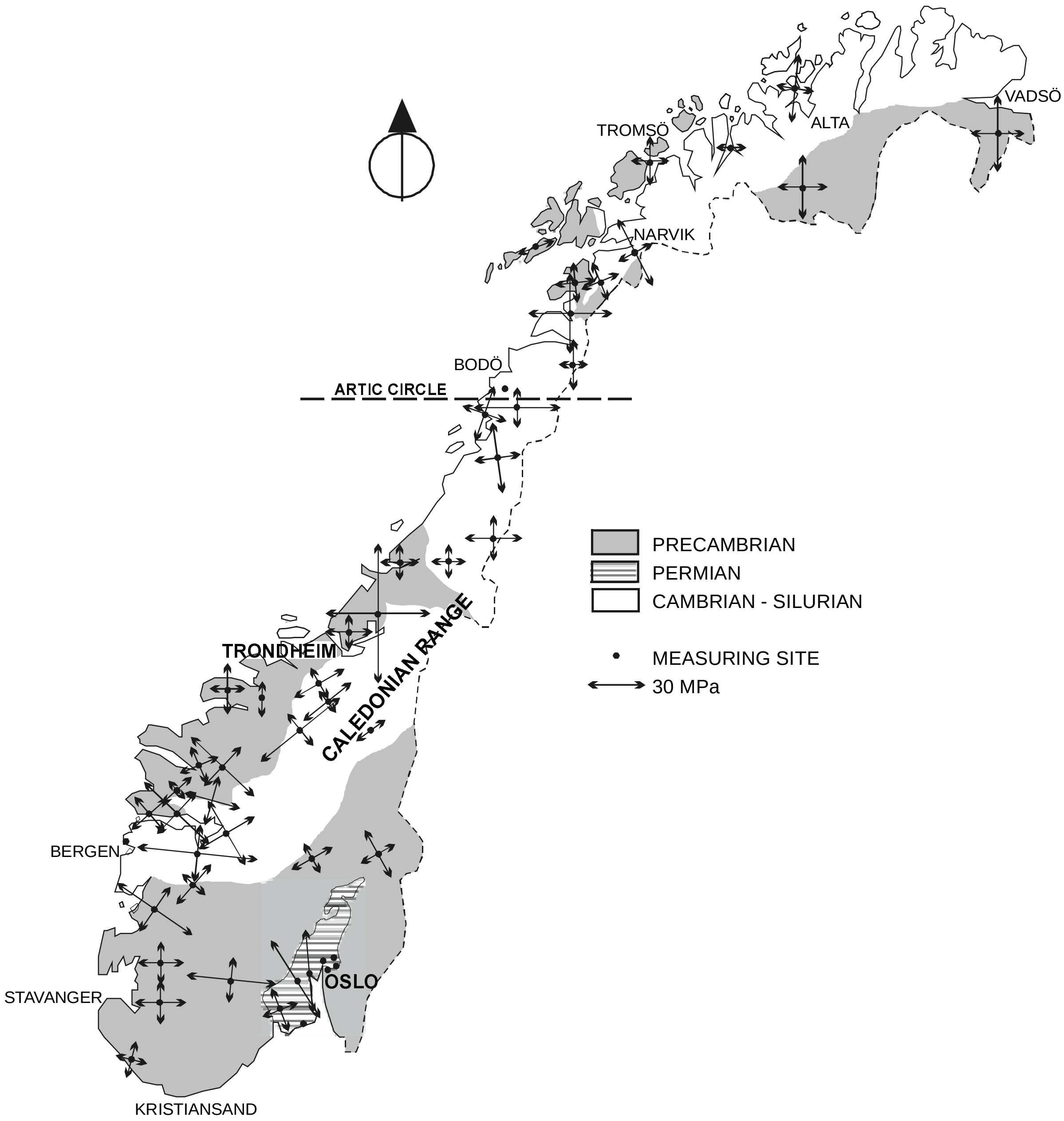


Figure 5.8 Directions and magnitudes of horizontal rock stresses in Norway (modified after Myrvang, 1993).

6 Groundwater and leakage

Groundwater, by definition, is the unconfined water that occurs below the groundwater table. The *groundwater table*, or *phreatic surface*, is the level below which the geologic formation is fully saturated. The groundwater normally represents the major part of subsurface water. There are, however, also several other ways in which water may occur in rock masses:

- *Chemically bonded* to the crystal structure (for instance in gypsum, $CaSO_4 \times 2H_2O$).
- *Absorbed*, by the crystal structure in some minerals (for instance smectite), to the surface in others.
- *Capillary*, in thin fissures and pore systems.

Some rocks, like young sandstones and certain limestones, may contain considerable volumes of capillary water. In the great majority of cases it is, however, the freely moveable groundwater that affects the excavation conditions and the long-term stability.

Water in rock masses is an integrated part of the *hydrologic cycle* as illustrated in Figure 6.1. The groundwater may travel considerable distances through a rock mass, and hence it is important to consider the regional geology and the overall groundwater pattern when potential water problems are to be analysed.

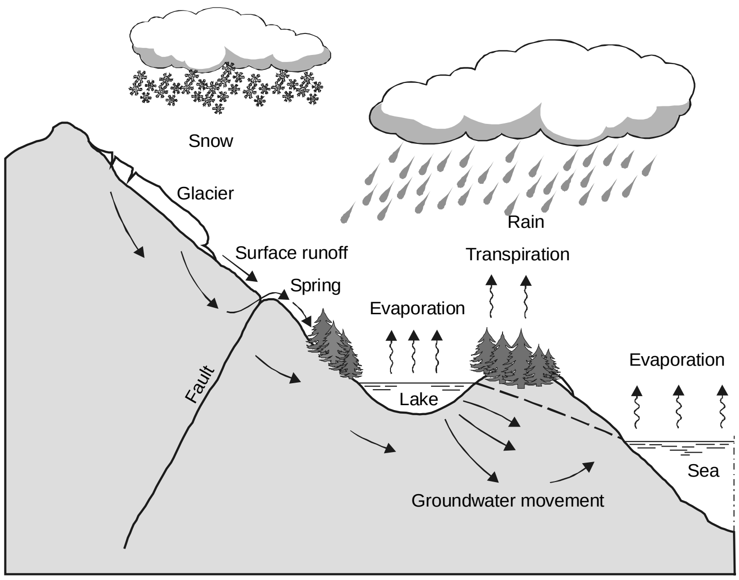


Figure 6.1 Simplified presentation of the hydrologic cycle showing some typical sources of groundwater.

6.1 Hydraulic conductivity and permeability

Most rocks have a rather small *effective porosity* (relationship between volume of saturable pores and total rock volume), see Table 6.1. The communication between individual pores is poor, and hence the permeability of intact rock is normally very low. In the great majority of cases, the rock mass permeability is governed by the permeability of joints and other discontinuities, although high porosity rocks such as young sandstones and certain volcanic rocks may in some cases represent exceptions from this general trend.

Table 6.1 Effective porosity of some Norwegian rocks (from Broch and Nilsen, 1996).

Rock	**Effective porosity** (%)
Basalt (Permian)	0.11
Diorite	0.84
Gneiss (Precambrian) - cores drilled parallel to foliation - cores drilled perpendicular to foliation	 0.75 0.69
Quartzite	1.09
Marble	0.48
Limestone	0.58
Sandstone (Devonian) - cores drilled parallel to foliation - cores drilled perpendicular to foliation	 0.22 0.27
Sandstone (Permian)	≈15
Sandstone (Jurassic)	≈30

Hydraulic conductivity, k (in m/s), also often referred to as the *coefficient of permeability*, is the parameter most commonly used for characterising hydrogeological conditions. The parameter represents the coefficient of proportionality of *Darcy's equation*:

$$v = \frac{Q}{A} = k \times i$$

where v = flow velocity (m/s)
Q = flow rate (m^3/s)
A = flow area (m^2)
i = hydraulic gradient

The unit of hydraulic conductivity is m/s, and its value depends on the nature of the rock mass as well as the nature of the fluid.

The *specific permeability*, K (in m^2) is often referred to simply as *permeability*. It depends on the nature of the rock mass only, and not on the nature of the fluid. The relationship between specific permeability and hydraulic conductivity is defined by the following equations:

$$K = k \times \frac{\mu}{\rho \times g} = k \times \frac{\nu}{g}$$

where μ = dynamic viscosity of the fluid [= 1.3 mPa × s (milliPascalseconds) = 1.3 cP [centipoise, or g/(cm×s)] for pure water at + 10°C]
ν = cinematic viscosity of the fluid (= 1.3 ×10-6 m^2/s for pure water at + 10°C)
ρ = density of the fluid
g = gravitational acceleration (9.81 m/s2)

An older, but still commonly used unit for permeability is *Darcy*. 1 Darcy is equivalent to the passage of one cm^3 of viscosity 1 cP fluid per second through a 1 cm^2 sample at a pressure of 1 atmosphere per cm of thickness:

$$1\,\text{Darcy} = \mu \times \frac{Q/A}{dp/dl} = 1.0 \times 10^{-12}\ m^2$$

where Q = flow rate (m^3/s)
A = flow area (m^2)
$\frac{dp}{dl} = \frac{\text{pressure drop}}{\text{flow distance}}$ (Pa/m)

To illustrate the orders of magnitude of the various parameters, typical values of fine sand and modestly jointed granite are indicated in Table 6.2.

Table 6.2 Examples of typical values of hydraulic conductivity (k, related to water flow) and permeability (K and d).

Material	**L** Lugeon	**k** m/s	**K** M^2	**d** Darcy
Sand, fine-grained	100	10^{-5}	10^{-12}	1
Granite, moderately jointed	0.1	10^{-8}	10^{-15}	10^{-3}

6.1.1 Conductivity of single joints

The width or aperture of joints has a major influence on the flow rate. For a planar array of parallel smooth joints the hydraulic conductivity is given by (Louis, 1969):

$$k = \frac{g \times e^3}{12\nu \times S}$$

where e = opening (aperture) of joint (m)
S = spacing between joints (m)
ν = cinematic viscosity (m^2/s)

For a doubling of the joint opening, the flow rate according to the *Louis-equation* is in theory increased by a factor of eight.

The Louis and the Darcy's equations are both based on *laminar flow* conditions, and the Louis-equation also assumes a joint geometry corresponding to a simple *parallel-plate model*. Due to the fact that most joints are rough and often partly filled, the in situ flow is unevenly distributed and in reality often follows irregular, narrow channels. In limestones and marbles the flow channels may reach considerable dimensions due to karst, the dissolution of calcite (see Section 6.3).

Due to the irregularity of the flow channels, the Louis and Darcy's equations will have limited validity for analysing single joints. Still, the equations are important for understanding the basic features of water flow through rock joints.

6.1.2 Conductivity of rock masses

The hydraulic conductivity of a rock mass is governed mainly by the degree of jointing and the character of the rock joints, and hence may vary within wide limits as shown in Figure 6.2.

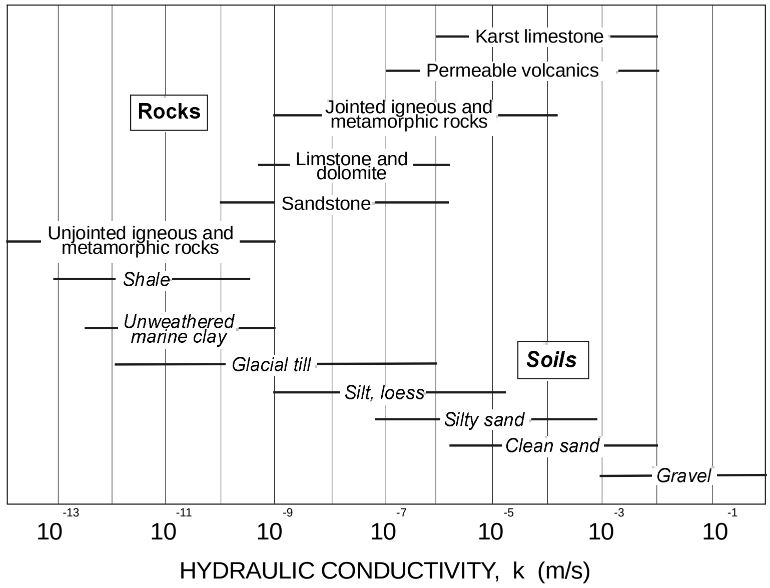


Figure 6.2 Typical hydraulic *conductivity* of rocks and soils (from Freeze and Cherry, 1979).

Jointed igneous and metamorphic rocks may have a hydraulic conductivity corresponding to that of sand, while unjointed rocks of the same categories may have a value lower than that of marine clay.

As result of jointing, the rock mass is inhomogeneous and anisotropic also in terms of conductivity. Thus, in a bedded rock mass the conductivity may be 1.5-2.0 times higher horizontally than vertically. Due to the general reduction of the aperture of joints with depth, and often the increase of spacing, that the conductivity decreases with depth below the surface is a typical trend as shown in Figure 6.3.

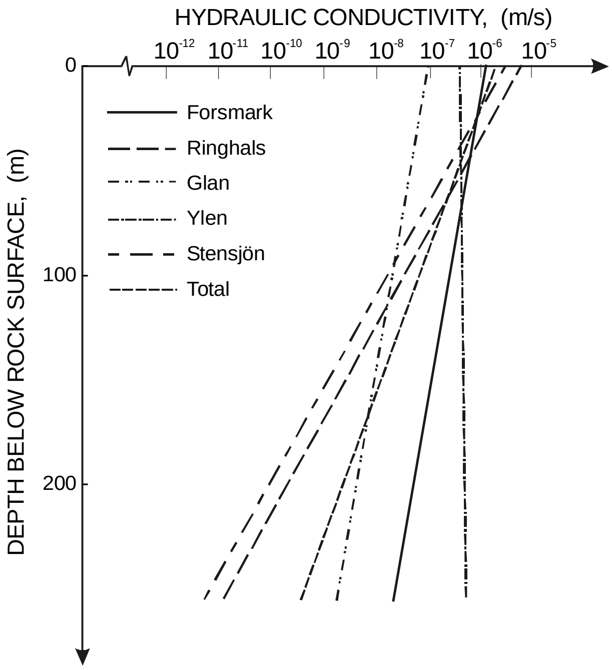


Figure 6.3 Hydraulic conductivity as a function of depth for Swedish test sites in Precambrian rocks (from Carlsson and Olsson, 1977).

6.2 Estimation of water leakage

It is generally very difficult to predict the locations and quantities of potential water leakage in a planned underground excavation. Permeability testing as discussed in Section 8.3 may give certain indications, and geoelectrical methods may also be used for evaluating the leakage potential. Scale effects, i.e., the conversion of small scale test results to large scale in situ conditions, often cause problems in such prediction.

In estimating *water leakage*, one of the following two approaches are normally used:

- Basic flow theory
- Numerical modelling

The methods may be applied for estimating the leakage into a tunnel or cavern during construction, as well as potential leakage out, for instance of a hydropower tunnel, during operation.

6.2.1 Basic flow theory

Throughout the years, various approaches to this problem have been suggested. A technique developed by Tokheim and Janbu (1984) was intended originally for evaluating potential air loss from unlined compressed air cushion surge chambers in jointed rock, but is also well suited for estimating water inflow.

According to Tokheim and Janbu, the water inflow into a tunnel or cavern is defined by the following equation:

$$Q_w = \frac{2\pi \times K \times L \times p}{\mu_w \times G}$$

where Qw = inflow rate (m^3/s)
K = specific permeability (m^2)
L = length of tunnel or cavern (m)
p = potential (active head) (Pa)
μw = dynamic viscosity of water (kg/m) = density $\times$ cinematic viscosity
G = geometry factor

The *geometry factor* (G) describes the flow pattern relatively to the geometry of the tunnel or cavern, and is given by:

$$G = \frac{\ln(2D - r)(L + 2r)}{r[L + 2(2D - r)]}$$

Where D = distance between the centre line of the excavation and the groundwater table.
r = "equivalent radius" of idealised geometry, i.e., the radius of a cylinder with a surface area equal to that of the actual excavation.

As indicated by the inflow equation, information about permeability is needed for Tokheim and Janbu's method to be applied. Hence, to obtain reliable input, permeability (water loss) testing of boreholes has to be carried out as described in Section 8.3.8.

The uncertainty when applying this method is the evaluation of the geometry factor (G). The equations were designed originally for a three-dimensional flow pattern. In rock mass, however, one single joint set often completely dominates the water flow. To compensate for the potential error due to anisotropy, $L >> r$ should always be used when evaluating the geometry factor G.

6.2.2 Numerical modelling

Numerical models as presented in Chapter 12 may also be used for analysing water flow. *Finite element models (FEM)* and *finite difference models (FDM)* are commonly used to do this. The latter are based on dividing the actual section into a grid to obtain approximate solutions at each node by iterations from given initial values, and involve relatively simple mathematics.

6.3 Problems caused by water

In terms of cost as well as time delay, water problems may completely overshadow rock stability problems. In the following, problems that may be caused by water will be discussed with an emphasis on inflowing water.

6.3.1 Chemical reactions

Some minerals react chemically with water, and such reactions may occasionally cause considerable problems. The classical example here is the *dissolution of calcite* ($CaCO_3$) due to acid water, which may be the cause of large water inflows through so-called *karst* channels:

$$CO_2 + H_2O + CaCO_3 \rightarrow Ca^{2+} + 2HCO_{3-}$$

The *oxidation of sulfides*, producing aggressive water, is a chemical reaction that should also be mentioned. Certain sulfides (as for instance monocline *pyrrhotite* ($Fe_{1-x}S$) react very quickly in contact with water and air, and one of the reaction products here is sulphuric acid (H_2SO_4), which may attack concrete and cause a considerable acceleration of corrosion:

$$Fe_{1-x}S + H_2O + O_2 \rightarrow FeSO_4 + H_2SO_4$$

6.3.2 Water inflow during excavation and operation of underground openings

Excessive groundwater pressure and/or flow may be encountered in practically any rock mass, but normally it will cause serious stability problems only in crushed or sand-like materials ("*running ground*"), or when associated with other forms of instability. The main effects of groundwater on stability are reduction of the strength of rock material and the shear strength of discontinuities. In swelling clay, water will significantly reduce friction and strength.

In extreme cases tunnels may be lost due to heavy water inflow. Close to the surface, the rock mass is generally more jointed and the joints more open than deeper down. Therefore, most of the leakage in a tunnel normally occurs in the shallowest part, but the most difficult leakage, due to the high pressure, is often experienced in the deeper parts. At great depths, inflow of hot water may cause particular problems. (The average *geothermal gradient* is approximately 30°C per kilometre, but in volcanic and hot spring areas, high temperature may also be a problem at depths of less than 100 metres).

Relatively few failures in tunnels are clearly related to *joint water pressure*. Groundwater pressure may, however, contribute to instability, particularly in weak ground. The impact of groundwater pressure should always be evaluated in cases where it is potentially significant.

There are also many other ways in which water inflow may cause problems. These are some of the most obvious:

- Drilling and charging may become difficult, and working conditions very poor in general.
- In declined tunnels, pumping may become a considerable cost.
- The roadway may be damaged, and in the worst case, washed away.
- Severe *ground settlement* may occur on the surface due to lowering of the groundwater table.
- In cold climates, freezing of leakage water may cause considerable problems, particularly in traffic tunnels.

6.3.3 Norwegian experience

Large water inflows are very difficult to predict, and hence it is a general experience that such incidents often come as unpleasant surprises during tunnelling. Throughout the years, some trends have, however, been revealed which seem to be fairly representative of large inflows.

6.3.3.1 Hydropower tunnels

For cases of large water inflows in hydropower tunnels in the crystalline rocks of Southern Norway, see Figures 6.4 and 6.5, the following common features have been reported by Selmer-Olsen (1981):

- The rock is strong and has a high E-modulus.
- The leakage joints are mainly steep (dip angle greater than 70°) and have a difference in strike direction of 45° ± 15° relatively to the strike of the major shear faults in the area, i.e., apparently they represent tensile, feather joints (cf. Figure 6.5).

The main leakage usually comes from only one single joint set, and is evenly distributed among the joints of that set.

The leakage is concentrated mainly within channels in the joints caused by the solution of calcite.

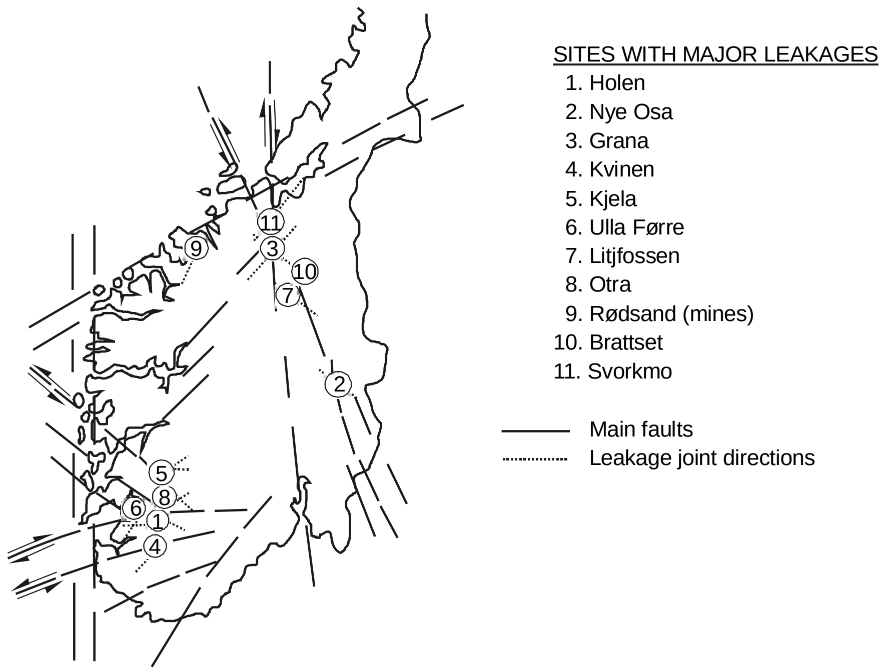


Figure 6.4 Locations of hydropower tunnels with large leakage in Southern Norway (from Selmer-Olsen, 1981).

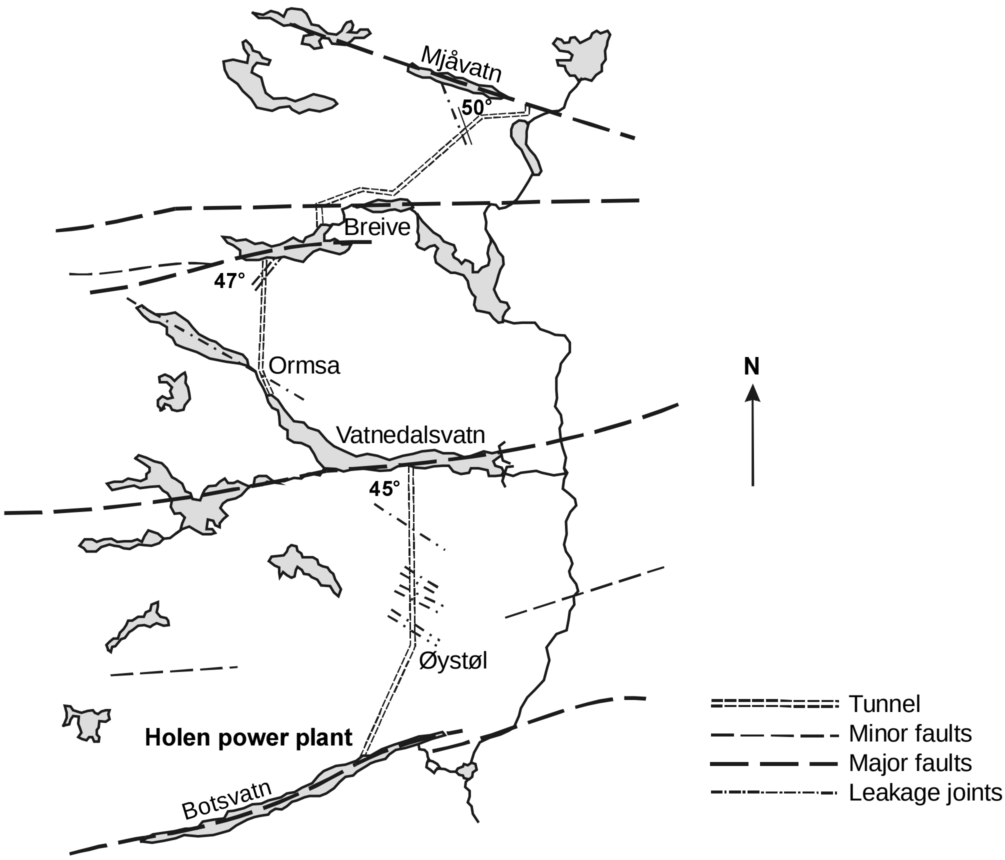


Figure 6.5 Major shear faults and leakage joints at the Otra Hydropower Project (site No. 8 in Figure 6.4), from Selmer-Olsen, (1981).

The sites in Figure 6.4 represent considerable variations in leakage volume as well as maximum pressure. Generally, however, the volumes are large with a concentrated leakage of 18,000 l/min at Kjela Hydropower Plant (No. 5 in Figure 6.4) representing the maximum value. In many cases the water head corresponds to the tunnel depth, with a pressure of 5 MPa (equal to 500 m of head) at Holen (No. 1 in Figure 6.4) representing the maximum value. (For smaller leakage it is a general experience that the water pressure is in most cases considerable lower than the theoretical maximum value corresponding to the tunnel depth. This is probably due to poor communication between permeable joints).

6.3.3.2 Sub-sea tunnels

In sub-sea tunnels, the downward inclination, the salinity of the leakage water and the unlimited reservoir add to the water problems already mentioned. Experience shows that in most cases, the leakage water becomes saline as soon as the coast line is passed (under steep valley sides, the hydraulic gradient may cause a slight delay). Water inflows in such tunnels are very unpredictable, but a general experience is that they are restricted to certain, limited sections along the tunnel. These sections normally do not coincide with major weakness zones, which are often rich in clay filling, but are most often characterised by distinct, single discontinuities (as is the case in the hydropower tunnels described above). Soil cover on the sea bottom also seems to play an important role for the extent of leakage.

In Table 6.3, figures for *water inflow* and *grouting* in some Norwegian sub-sea tunnels are summarised. All tunnels are in Palaeozoic and/or Precambrian rocks. Maximum water inflows are as measured in percussive probe drill holes (see Figure 6.6) and permanent leakage is the leakage after grouting. In subsea road tunnels, a water inflow of about 5 l/min per 25 m long probe drille hole represents a common threshold value for grouting and a permanent inflow per 1000 metres of tunnel of maximum 200-300 l/min is normally accepted.

Table 6.3 Water leakage and grouting in some Norwegian sub-sea tunnels (from Nilsen, 1993).

TUNNEL	MAX. WATER INFLOW ENCOUNTERED (l/m)	GROUTED LENGTH (in relative %)		GROUT CONSUMPTION (tons)	PERMANENT LEAKAGE (l/min/m)
		under sea	under land		
Vardö	65	12	0	83	0.38
Karmsund	300	5	11	64	0.08
Fördesfjord	< 300	13	0	35	0.09
Förlandsfjord	< 300		1	5	0.08
Hjartöy	≈ 200	13	5	35	0.09
Ellingsöy	≈ 400	34	15	345	0.28
Valderöy	≈ 100	12	0	44	0.29

6.4 Sealing of water leakage

Grouting for sealing of water leakage is much more likely to be successful when a sufficient counter-pressure can be established. Hence, whenever possible in underground excavation, grouting should be performed ahead of the tunnel face (*pre-grouting*), rather than behind the face (*post-grouting*). What is achieved by post-grouting in many cases is simply to move the actual water leakage to another position in the tunnel contour.

6.4.1 Practical procedure

The grouting procedure also includes initial drilling and testing of probe holes ahead of the tunnel face as discussed in Section 8.3.11. The final decisions on whether and how to grout are based on recordings from water loss testing and/or leakage measurements of these holes, or simply on observations and recordings from the drilling of blast holes. If the water leakage from a water-bearing zone is higher than a pre-set threshold value, pre-grouting is carried out as shown in principle in Figure 6.6.

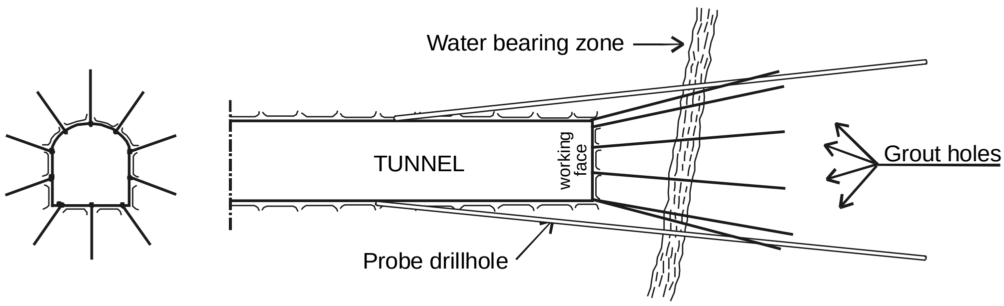


Figure 6.6 Principal sketch of drill hole pattern for pre-grouting of water-bearing zone.

Table 6.4 Suggested guidelines for selection of initial grout-hole spacing (from Boge & Johansen, 1995).

Jointing	**Sealing requirement**		
	Strict	**Intermediate**	**Low**
Weakly jointed, partly open discontinuities	3-5 m	4-8 m	8-10 m
Moderately jointed	2-4 m	3-6 m	6-8 m
Strongly jointed, discontinuities filled with silt or clay	1-3 m	3-4 m	5-6 m

In underground excavation, the tunnel jumbo is normally used for the drilling of grout holes with lengths of 25-30 metres and single packers are placed at the outer end of the holes. In very long grout holes, e.g., for dam foundations, packers moved in steps towards the outer hole end are used. It is important that enough holes are drilled to give a continuous grout barrier. Suggested guidelines for selection of primary hole spacings as a function of rock conditions and sealing requirements are given in Table 6.4. Adjustment of the initial spacing should be done, if necessary, as soon as experience from grouting is gained.

6.4.2 Grout materials

In most cases cement-based suspensions are used for grouting. When the leakage criterion is strict, chemical grouts such as silicates, acrylic products or polyurethane foams are used. Penetration into thin joints is far better for these products than for cement grout, which normally has a lower limit for penetration corresponding to a joint aperture of 0.1-0.2 mm. Because of the often toxic character of chemical grouts and their high price, there has been a tendency recently for micro-cements to gradually replace chemical grouts. A guideline for selection of grout material for various ground conditions is presented in Table 6.5.

Table 6.5 Recommended guidelines for selection of *grout material* (based on Boge and Johansen, 1995).

Type of discontinuity and aperture *(fill material)*	**Typical Lugeon Value**	**Recommended grout material**
Open channels/karst (stone/gravel)	≥ 50	Cement with sand/gravel and accelerator/ expanding admixture. Polyurethane for stopping major inflow.
Major discontinuities, aperture ≥ 1cm *(coarse gravel)*	10 - 50	Cement with bentonite or plasticizer/ expanding admixture. Polyurethane is useful for stopping flowing water
Intermediate discontinuities (joints), aperture 0.3 - 1 cm *(gravel)*	3 - 15	Cement with super-plasticizer (SP). Polyurethane is useful if there is flowing water.
Joints, aperture 0.01 - 0.1 cm *(coarse-intermediate sand)*	1 - 5	Micro-cements with SP. Polyurethane, silicates, acryles.
Small joints,aperture ≤ 0.01 cm *(fine-intermediate sand)*	≤ 1	Ultra fine-grained micro-cement with SP and/or silicates, acryles, epoxy, polyurethane.

6.4.3 Grout pressure/stop criteria

By utilising high-pressure pumps the grout is forced into the discontinuities which intersect the drill hole. The optimum grouting pressure depends much on the local rock conditions. For cement grouting in shallow-seated tunnels, pressures between 1 and 4 MPa (10-40 bars) are commonly used, or, as proposed by Boge and Johansen (1995) as an initial guideline, pressure in bar corresponding to 1-2 times the tunnel depth in metres. For cement grouting in deep-seated tunnels, grout pressures of more than 6-7 MPa have effectively been used. In cases with particularly difficult rock conditions it might be necessary to make a concrete plug at the tunnel face to be able to increase the grout pressure to a satisfactorily high level.

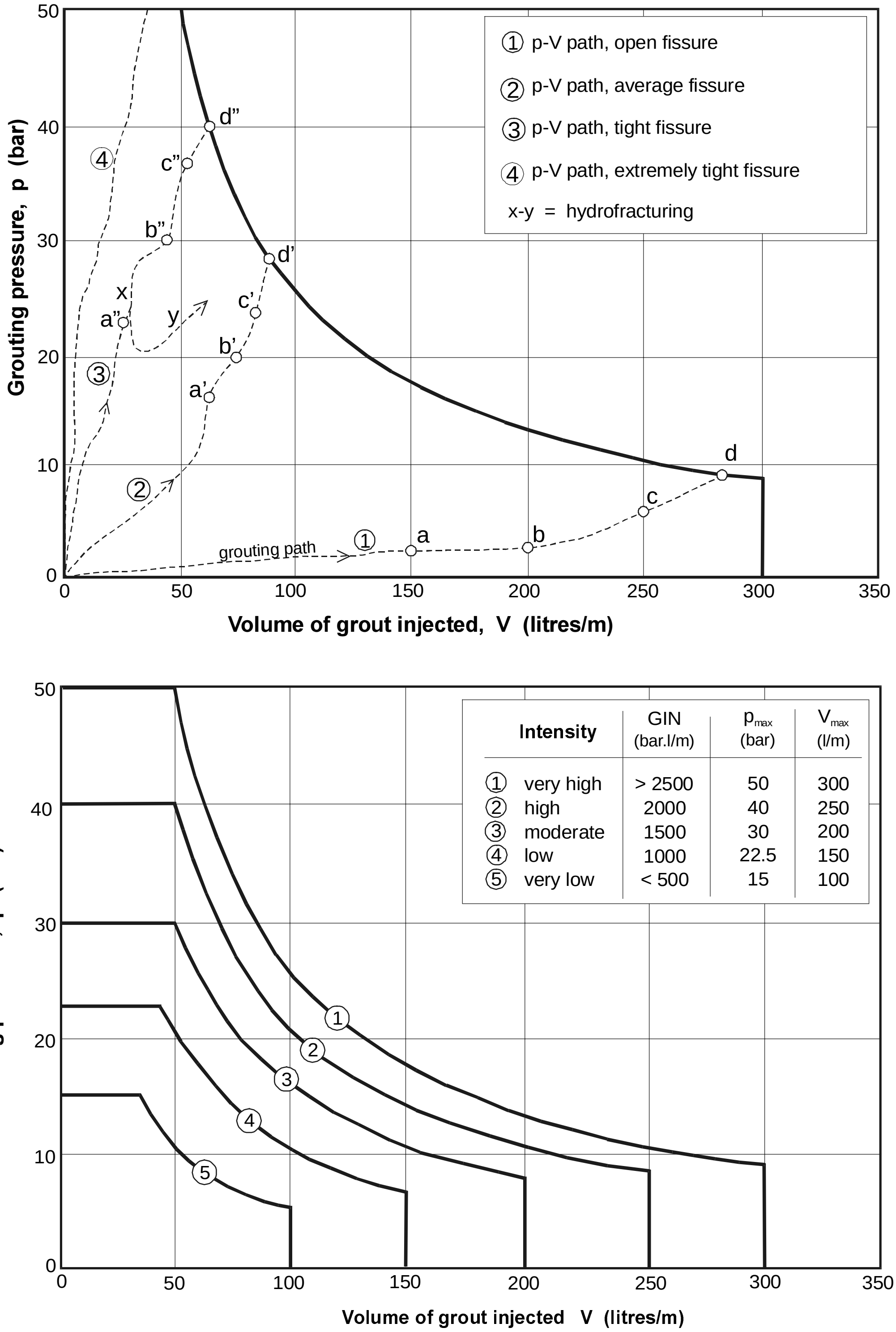


Figure 6.7 Examples of grout paths (upper diagram) and proposed p × V-limits (lower diagram) according to the GIN-principle (from Lombardi and Deere, 1993).

Normally, a maximum final grout pressure and/or a maximum grout take per packer position is defined before changing the grouting strategy (grout material, pressure, etc.) and in many cases such criteria work well. Internationally, the so-called *GIN-concept*, introduced by Lombardi (1993) has gained more and more acceptance, and is today very commonly used. *"GIN"* stands for *Grout Intensity Number* and is the product of pressure (in bar) and grout take (in litres per drillhole metre), see Figure 6.7.

To obtain a considerable reduction of the water leakage by pre-grouting is in many cases relatively simple. A completely impervious tunnel, however, is in most cases very difficult to achieve. It is often less expensive to stop the initial 95% of a water leakage than the remaining 5%.

7 Basis for planning and design

The efforts made in ground investigations, which represent the main input for planning and design of any rock excavation project, vary considerably from project to project and from country to country. Due to mainly favourable geological conditions, the extent of investigation in hard rock countries such as Norway has traditionally been relatively limited, typically representing 0.5-1% or less of the total construction cost. The recent trend towards more complex projects, often located in urban areas and with potentially more serious consequences from instability, have, however, escalated investigation costs up to around 5% in some cases.

In USA, the U.S. National Committee on Tunneling Technology (USNC/TT, 1984) recommends site exploration budgets averaging 3.0% of the estimated project cost.

The basic principle for defining the necessary extent of investigation and exploration always should be to relate it to the type and complexity of the project. Thus, much more comprehensive investigation and planning are required, of course, for a nuclear waste repository than for a minor water tunnel, for instance. In any case, the investigation and planning have to be adjusted to the actual geological conditions.

Guidelines for geotechnical planning and design have been defined in several countries. In the following some main aspects of the guidelines according to the Norwegian Council for Building Standardisation (NBR) and the European Committee for Standardisation (CEN) will be described.

7.1 Geotechnical classes and categories

7.1.1 NS 3480

The *Norwegian Standard NS 3480* "Geotechnical planning", covering rock as well as soil, gives guidelines for geotechnical and engineering geological investigation, planning, supervision and control. A basic principle is that project owner and designer jointly, based on evaluation of so-called *damage consequence class* and *degree of difficulty*, define the *geotechnical project class* as shown in Table 7.1.

Table 7.1 Definition of geotechnical project class according to Norwegian Standard (from NBR, 1988).

Damage consequence	**Degree of dificulty**		
	Low	Medium	High
Less serious	1	1	2
Serious	1	2	2
Very serious	2	2	3

Potential damage consequences to be evaluated according to NS 3480 are related to life as well as property, including long-term economical consequences. Degree of difficulty is to reflect uncertainty in planning and construction, and depends mainly on:

- The in situ engineering geological conditions.
- The extent to which the ground conditions will influence the planned project.
- Whether reliable methods exist for defining the ground conditions and the input parameters for analyses.
- Whether reliable methods exist for design of the project.
- Whether experience exists from similar projects.

Hydropower tunnels and low-traffic road tunnels in rural areas might be mentioned as examples of projects often belonging to geotechnical project class 1, while for instance sub-sea tunnels and large caverns in urban areas often belong to class 3.

The geotechnical project class, as described in more detail in NBR (1988), also defines the amount of effort to go into:

- Collection of information on ground conditions.
- Analyses and planning.
- Design supervision and control.
- Construction supervision and control.

7.1.2 Eurocode 7

Concerning European standardisation in this field, the relevant standard, *Eurocode 7* (EC 7), exists as a so-called European Prestandard , or “ENV”, (CEN, 1994). After having gained experience from practical application it will, if required, be modified and most likely converted to European Standard (EN) in 2000. It will then replace relevant national standards in European countries (in Norway: NS 3480).

The basic approach to geotechnical design of EC 7 is in principle the same as in NS 3480. However, while NS 3480 mainly provides the framework for geotechnical design, EC 7 gives more detailed rules.

Three so-called *Geotechnical Categories*, 1, 2 and 3 (corresponding to the Geotechnical Project Classes in NS 3480), are introduced in EC 7. Preliminary classification of a structure according to geotechnical category normally should be performed prior to geological investigations. The category has to be checked, and may be changed at each subsequent stage of the design and construction process.

The following factors, according to CEN (1994), shall be taken into consideration when determining the geotechnical design requirements:

- Nature and size of the structure and its elements, including any special requirements.
- Conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.).
- Ground conditions.
- Groundwater situation.
- Regional seismicity.

- Influence on the environment (hydrology, surface water, subsidence, seasonal changes in moisture).

The three geotechnical categories according to EC 7 are defined as follows:

1. ***Geotechnical category 1)***
   Small and relatively simple structures,
   - for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations,
   - with negligible risk for property and life.

2. ***Geotechnical category 2)***
   Conventional types of structures and foundations with no abnormal risks or unusual or exceptionally difficult ground or loading conditions. Quantitative geotechnical data and analyses are required to ensure that the fundamental requirements will be satisfied, but routine procedures for field and laboratory testing may be used.
   "Tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements", are listed among examples of projects belonging to this category.

3. ***Geotechnical category 3)***
   Very large and unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas.

The various design aspects may require treatment in different geotechnical categories. It is not necessary to treat the entire project according to the highest of these categories.

7.2 Design principles

The design, according to EC 7, may be based on:
1. Calculation.
2. Prescriptive measures.
3. Load tests and experimental models.
4. The observational method.

These four approaches may be used in combination. For detailed descriptions of the methods reference is made to CEN (1994). It is emphasised here, however, that according to EC 7 as well as NS 3480, all calculation is to be carried out according to the *partial factor principle*, i.e., by applying *partial factors* for action and materials, and not an overall safety factor.

In principle, calculation according to the partial factor principle is carried out as follows:

$$F_d = F_k \times \gamma_f$$

$$M_d = \frac{M_k}{\gamma_m}$$

where	F_k =	characteristic action	M_k =	characteristic strength
	F_d =	dimensioning action	M_d =	dimensioning strength
	γ_m =	material factor	γ_f =	partial factor for action

The design is satisfactory if $M_d > F_d$.

In the great majority of rock engineering cases, the ultimate limit state is dimensioning for stability and design, and the respective partial factors recommended by CEN (1994) and NBR (1997) are as shown in Table 7.2. The figures in brackets are indicative values, so-called “*boxed values*”, defined by CEN. The authorities in each respective member country are expected to assign definitive values. The values assigned by the Norwegian authorities (NBR) are shown in bold in Table 7.2 where different from the CEN indicative values.

Table 7.2 Partial factors, ultimate limit states in persistent and transient situations (based on CEN, 1994 and NBR, 1997).

	Actions (γ_f)			**Ground properties (γm)**			
Case	**Permanent**		**Variable**	**Tan φ**	**c’**	**c_u**	**σ_c**
	UNFAVOURABLE	**FavourABLE**	**UNFAVOURABLE**				
A	[1.00]	[0.95]	[1.50]	[1.1]	[1.3] **1.1**	[1.2]	[1.2]
B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
C	[1.00]	[1.00]	[1.30]	[1.25] **1.2**	[1.6] **1.2**	[1.4] **1.3**	[1.4]
φ = angle of shearing resistance; c’ = cohesion intercept in terms of effective stress c_u = undrained shear strength; σ_c = compressive strength of soil or rock							
Case A is only relevant to buoyancy problems, where hydrostatic forces comprise the main unfavourable action, and thus is not very relevant in rock engineering.							
Case B is often critical to the design of the strength of structural elements involved in foundations or retaining structures. Where there is no strength of structural elements involved, case B is irrelevant.							
Case C is generally critical in cases, such as slope stability problems, where there is no strength of structural elements involved. *It generally is the most relevant in rock engineering design.* Where there is no strength of ground involved in the verification, case C is irrelevant.							

The design, according to CEN (1994), shall be verified for each of the three cases A, B and C separately as relevant. The three classes have been introduced in order to ensure stability and adequate strength in the structure and in the ground in accordance with Eurocode 1: "Basis of design".

For more details concerning geotechnical design, reference is made to the respective standards: CEN (1994) and NBR (1997)/the National Application Document (NAD) of the relevant country.

7.3 Supervision of design

In *NS 3480*, and the Norwegian application document for EC 7 (NBR, 1977), the following guidelines for supervision of the design, depending on geotechnical category, are defined:

- For category 1, supervision can be carried out by the person who has made the design (*simple supervision*).
- For category 2, supervision shall be carried out by a person, who is appropriately qualified and experienced and who has not taken part in the design (*normal supervision*).
- For category 3, it is recommended that in addition to normal supervision, extra supervision is carried out by a person or organisation that is independent of the geotechnical designer (*extended supervision*).

This is also in accordance with the principles of Eurocode 7, although supervision of design is not expressly mentioned here.

7.3.1.1 Investigations and tests

Site investigations are the efforts that have to be made to characterise the rock mass conditions of the project area. *Testing* in this connection is carried out at an in situ level for rock mass tests and at a laboratory level for rock mechanics tests.

In contrast to most other materials used for construction purposes, the quality of the rock mass is evaluated more by observation than by testing. The large volume of material involved, its inherent properties and the limited possibility to actually observe the material represent great challenges in investigation and interpretation of results as well as characterisation of the complex material called rock mass.

The geological conditions of sites may vary within wide limits. Each site has its own characteristics, and there is no "standard investigation procedure" that will be the only right one in all cases. When it comes to engineering geological investigations, flexibility is a keyword, representing the potential of considerable cost saving advantages in geo-investigation practice.

7.4 Objective

The main goals of engineering geological investigations are generally to obtain:

- The input necessary for an evaluation of siting and design alternatives and for the overall planning of the project.
- A basis for stability analyses and the input parameters necessary for evaluation of excavation method, planning of rock support, use of excavated material, etc.
- A basis for cost evaluation and for preparation of tender documents.

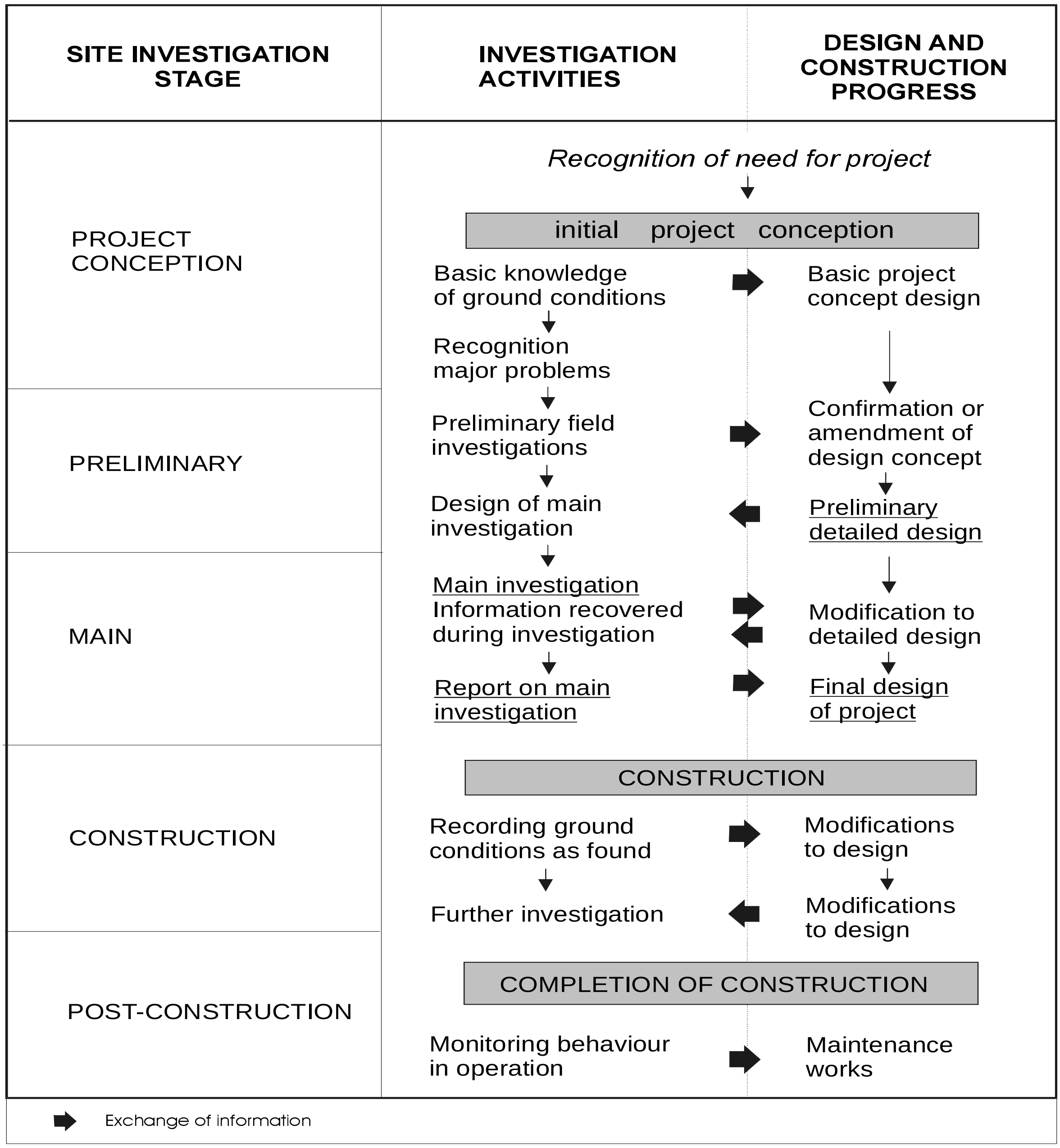
The facts that geological formations are three-dimensionally variable, and that only a limited number of measurements or observations can be made, have important consequences. The principal one is that the subsurface has to be described by a limited number of parameters, and that the values of these parameters are imprecisely known. It is important to accept the fact that a geotechnical parameter is expressed by a range of values, and that the actual range may be wider than actually observed. Thus, in most cases it is recommended not to make too large an effort to obtain accurate values of the various parameters. Often it is better to collect a wider statistical material.

The types of investigations and the investigation procedures will vary according to the nature of the project, the complexity of the geology, the background of the engineering company and the experience of the personal involved. It is generally recognised that investigations are best organised for maximum benefit if they are linked with the progress of engineering design and construction as indicated in Table 8.1

7.5 Investigation stages

Engineering geological investigations related to tunnels and underground openings can be divided into two main stages:

Table 8.1 Site investigations versus the progress of engineering design and construction as recommended by IAEG (1981).



1. *Pre-construction phase investigations*, or *pre-investigations*.
Underground excavation has not yet started and information has to be collected on or from the surface.

2. *Construction phase investigations* or *post-investigations*.
As tunnels are excavated, the rock masses become more accessible for inspection and sampling.

Each of the main stages again can be divided into two sub-stages (Broch, 1988):
- preliminary site exploration,
- detailed surface investigations,

and:
- detailed subsurface investigations,
- tunnel mapping.

In the following, the two investigation phases and their stages will be discussed in more detail.

7.5.1 Pre-construction phase

7.5.1.1 Primary site exploration

This initial stage, also referred to as *feasibility study exploration*, is often based on the designers' pre-feasibility study. The aim is either to study the feasibility of a planned tunnel, or, more often, to evaluate and reduce the number of alternatives based on geotechnical information. This is a highly challenging phase. Important decisions are made, often based on limited information. Experience from similar projects and sites is therefore of particular value.

At this early stage, *desk studies* of available geological information, such as reports, geological and topographical maps and aerial photos are made. This will give the first information on:
rock types and boundaries,
areas where the bedrock is covered with soils,
the locations and directions of the more important weakness zones, and
the stress situation in the area.

During the following *walk-over survey,* certain key points of the actual area are investigated. Under favourable conditions the necessary fieldwork can be reduced to sampling and control of important points in the project area. *Rock sampling* for simple classification tests are done, and the most important joint information is collected.

In the *feasibility report,* all collected information is presented and the different alternatives discussed. Plans and cost estimates for further investigations are presented, and any need for supplementary maps are made known. At this stage, an important decision has to be made as to whether or not to follow up with more expensive investigations.

7.5.1.2 Detailed surface investigations

Based on the feasibility study report, the client, in co-operation with their consultants, has to decide whether or not further planning should be carried out, and if so, what alternatives should be investigated. Additional air photos have to be taken if required, and better maps drawn. The engineering geologist normally needs air photos and maps that cover a larger area than is strictly necessary for the other planning operations of the project.

The air photos and maps for the detailed investigation should be on a scale that is relevant for the actual problem. Air photos to scale 1:15,000-1:20,000 and maps to scale 1:10,000 or 1:5,000 are recommended as a basis. For important areas such as tunnel entrances, cavern locations and dam sites, maps on a considerably larger scale are recommended (scale 1:1,000, occasionally even larger).

At this stage of the investigations, a detailed engineering geological *field mapping* is carried out. The goal of this mapping should be to collect information about all factors that may cause difficulties for the project.

The results of the detailed surface investigations are collected in a *detailed investigation report,* which may be used as a part of the tender documents. This report contains engineering geological descriptions, evaluations of construction and stability problems in the different parts of the project and an estimation of necessary rock support.

7.5.2 Construction phase investigations

7.5.2.1 Detailed sub-surface investigations

During the planning of underground projects, important decisions have to be made about what investigations should be carried out before the start of excavation, and what investigations might be postponed. When the construction work has started and the tunnel can be entered, the possibilities of obtaining better information on the ground conditions improve considerably.

A high degree of flexibility and simple pre-investigations are recommended when it is possible to start construction phase investigations early in the construction period. Expensive pre-investigations, e.g., deep core drillings, can often be replaced by much cheaper pilot borings from the tunnel face during construction. Rock stress measurements should preferably be made in underground openings and tunnels, and these are also good examples of detailed investigations that may be postponed until tunnelling has started.

Detailed sub-surface investigations should not only be delayed pre-investigations, but also a control of and supplement to the pre-investigations. For sub-surface works the pre-investigation report always has to be based on certain assumptions. The sooner these are verified, the better the work plans can be for the remaining part of the underground works.

7.5.2.2 Tunnel mapping

Many tunnels are difficult to inspect once they have been put to use. For the owner it is therefore useful to have detailed information of the ground conditions of the project. Such maps should contain all geological elements that influenced the stability and conditions of the tunnel, e.g., rock type and character, jointing, faults, water leakage and areas with rock burst problems, in addition to information about support work.

Tunnel mapping may be carried out in various ways, and there are several alternatives for presentation of the results. One alternative is shown in Figure 8.1. For additional documentation, photos can be used as a supplement to the form.

When the project is completed and all investigations carried out, a final report should be made containing all experience gained during the planning and construction period. Maps and drawings as earlier described should be included in this report.

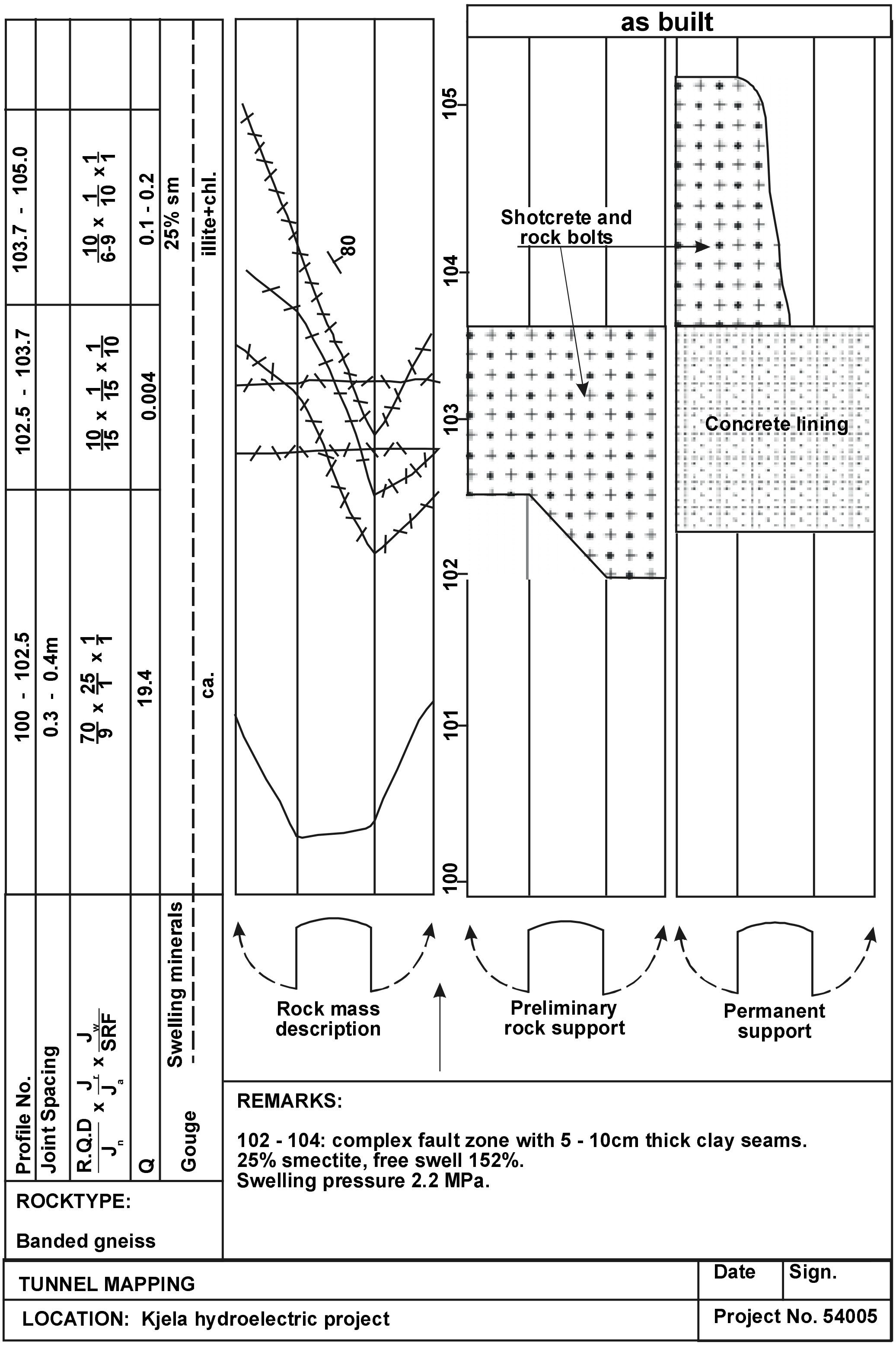


Figure 8.1 Example of detailed tunnel mapping (revised from NBG, 1985).

7.6 Field investigations and tests

The methods for the collection of geological data have not changed very much over the past 20 years. Due to the high cost of sub-surface exploration, the main investigation is often restricted to field observations. The site of a proposed underground excavation is therefore seldom investigated in as much detail as a design engineer could wish.

Since many field tests are high cost, their value should always be carefully weighed against their costs.

7.6.1 Approach

Investigation of engineering geological conditions is mainly based on mapping of:

- *Outcrops* or open cuts.
- *Pilot tunnels*, *adits* or *shafts* made prior to construction.
- Existing, nearby underground openings.

In addition, information may be collected by drilling and geophysical methods as shown in Table 8.2, and by a variety of more or less sophisticated field tests and experiments.

Table 8.2 Main possibilities for collecting information on characteristic rock mass parameters (from Palmström, 1995).

Rock mass parameter	**Data collected from**				
	Outcrops	Refraction seismic profiles	Drill cores	Adits	Underground openings
Rocks					
- distribution of rocks	x/(x)	-	x	x	x
- sample for strength tests	x/(x)	-	x	x	x
Joints and jointing					
- joint spacing	x	(x)	(x)	x	x
- joint length	x/(x)	-	-	(x)	(x)/x
- orientation	x	-	(x)	x	x
- waviness	x	-	-	x	x
- smoothness	x/(x)	-	(x)	x	x
- filling or coating	(x)/-	-	(x)/x	x	x
Faults and weakness zones					
- persistence	(x)	-	-	-	-
- orientation	(x)	-	-	(x)	x
- thickness of zone	(x)/-	(x)	(x)	(x)/x	x
- gouge material	-	-	(x)	x	x

x Parameter can be measured or task accomplished
(x) Parameter/task may partly or sometimes be measured/accomplished
- Not generally possible to measure/accomplish the parameter/task

7.6.2 Desk study

A lot of valuable information can be obtained during the desk study, and spending time at this early stage of investigation on collecting, systematising and studying relevant background material such as topographical and geological maps, aerial photos and geological reports is normally a very good investment.

Aerial photographs are a particularly useful tool since weakness zones are often easily detected on a stereo-pair of aerial photographs due to the exaggerated vertical scale. Because of the special features of aerial photographs, important information for the planning of underground excavations may thus be obtained early.

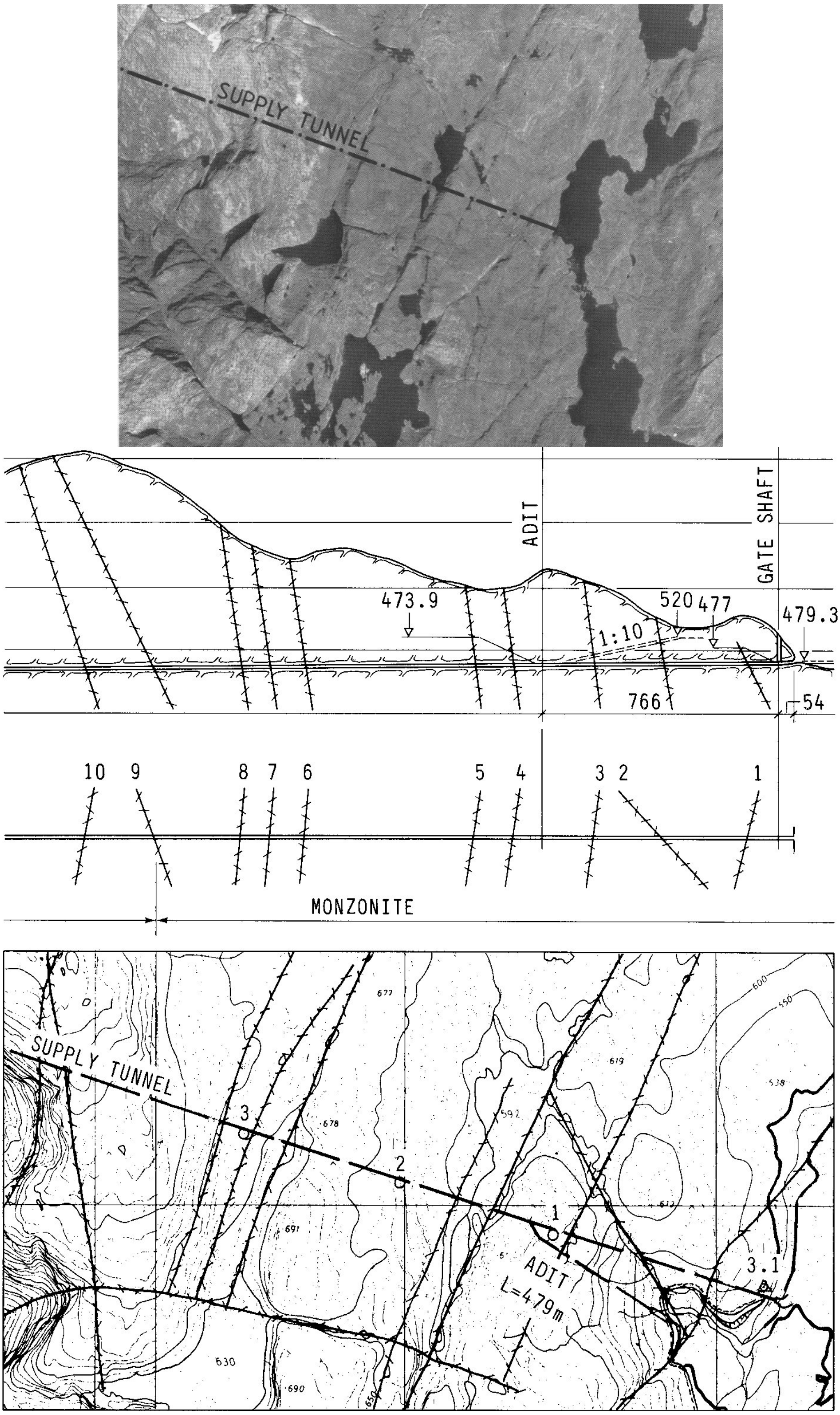


Figure 8.2 Aerial photograph in scale 1:15,000 covering the upper part of a water supply tunnel, and interpretations of locations and orientations of weakness zones based on the photo.

Jointing and foliation can often be seen on large-scale photographs. It is, however, necessary to measure their orientations more accurately by means of field observations. Flat-lying or less pronounced joints may often not be seen.

Weakness zones may be identified on aerial photographs as trenches, gullies and depressions in the terrain, due to the exaggerated vertical scale. It is especially for this reason that aerial photographs are so useful. The observations can be transferred to the map as shown in Figure 8.2. If the zones are planar, which they often are over limited lengths, their dip angle may also be calculated.

7.6.3 Field mapping

An engineering geologist's basic equipment for field mapping consists of a *hammer*, a *compass* with a *clinometer* and a *notebook*. In addition to this, maps, aerial photographs, a pocket *stereoscope*, a *knife*, *hydrochloride acid* (for identifying potential calcite), etc., are indispensable tools. In some cases, a *GPS-instrument* and an *altimeter* may also be useful. For the later stages of project planning, extensive *photographic documentation* is often a good investment.

For the engineering geologist it is more important to divide the types of rocks according to their mechanical properties than according to their mineralogical composition, see Section 2.2.4. The character of discontinuities is mapped according to the principles described in Chapter 3.

Sampling is an important part of the fieldwork. A logical first step often is to take small hand specimens to get an overview of the distribution of the different rocks and the variations within each type. After these hand specimens have been studied, a programme for the sampling of larger specimens for mechanical testing (often 15-20 kg or more) can be worked out. Great care must be taken that the specimens collected are representative. The fewer specimens that are analysed, the more important is the sampling. To avoid the effect of weathering, some blasting is often necessary. This may, however, induce new cracks in the specimens. Laboratory methods for the testing of physical and mechanical properties are described in Section 8.4.

7.6.3.1 Strike and dip measurements

The orientation of joint planes and other planar geological features is described by their *strike* and *dip*. A geologist's compass is needed to take the necessary readings. The strike of a plane is the trace of the intersection of that plane and a horizontal surface, and is measured with the compass held horizontally against the plane. The dip describes the plane's inclination to the horizontal, and is measured with the compass held in the vertical plane. The term *dip direction* is also used by many geologists. The relationship between the three terms is illustrated in Figure 8.3.

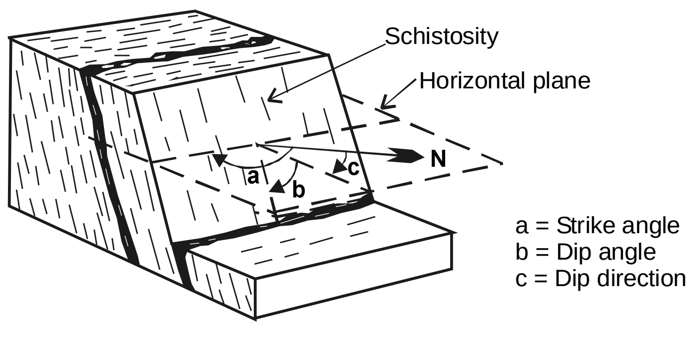


Figure 8.3 Definition of geometrical terms.

There are many ways to note the results of strike and dip measurements. The best method is always the one that the respective engineering geologist is most familiar with. If no particular preference exists, it is recommended to measure the strike angle in a clock-wise direction as indicated in Figure 8.3. To make the dip designation unambiguous, it is important to indicate also the direction towards which the plane dips. It is also important to indicate whether a 360° or 400^g compass is used.

For the situation in Figure 8.3, the recommended designation of strike and dip thus will be:

strike/dip = N130°E/60°NE.

The dip direction and dip are:

dip direction/dip = N40°E/60°

(generally dip direction is: strike ± 90°, in this case 130° - 90° = 40°).

The orientation of weakness zones are best evaluated and calculated from studies of aerial photographs and maps. Additional field observations are, however, advantageous, and often necessary. One problem often met in the field is to find representative planes on which the zone orientation measurements can be made, since close to a tectonic weakness zone there are always joints and fissures of different directions.

7.6.3.2 Presentation of strike and dip measurements

There are various graphical methods for presenting the results of a joint survey. One is the *joint rosette* or strike frequency diagram, of which two alternative versions are shown in Figure 8.4. Here, the direction of the strike line is directly plotted onto a simplified compass rosette. Joints with strikes within intervals of 5-10° are counted together, and the numbers are plotted along radial axes given by the concentric circles.

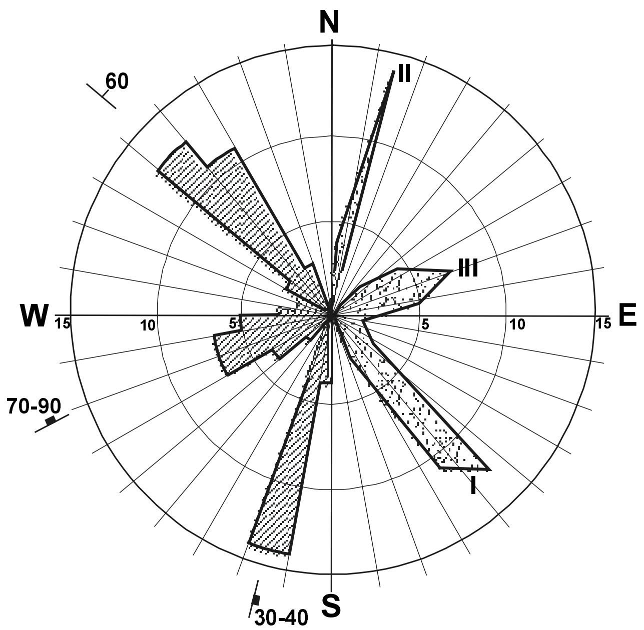


Figure 8.4 Two alternative ways of presenting the same data in a joint rosette (on left and right side of the N-S axis, respectively). From ISRM, 1981a.

For the main joint sets in the rosette, the dips are indicated as shown together with a brief description of joint character. Larger discontinuities such as faults may be designated separately, and technical data such as the orientations of underground excavations may be shown. The number of joints represented in a joint rosette should always be indicated, and it is generally required that the number of observations gives a statistical correctness.

The joint rosette can be a handy tool in evaluating the optimal orientation for an underground opening. However, when a large number of joint measurements are to be studied, *stereographic projection* is often more useful. This tool gives much more detailed information on variations in dip of each individual joint set than the other statistical methods, and is also a much better basis for identifying discontinuities that are of an unfavourable orientation for the actual tunnel or cavern.

The basis of stereographic projection and *stereonets* is the spherical projection. The so-called reference-sphere, as shown in Figure 8.5a, has a horizontal reference great-circle (the equator plane) and a fixed North-South axis. The geological plane in question is imagined placed through the centre of the sphere. The *great circle* of the plane in question is defined as its intersection with the sphere. The *pole* of the plane is defined as the intersection between the sphere and the normal of the plane through the centre of the sphere, as shown in Figure 8.5b. In principle, the upper and lower hemispheres provide the same information, and only one of them are therefore used. In engineering geology, only the lower is used.

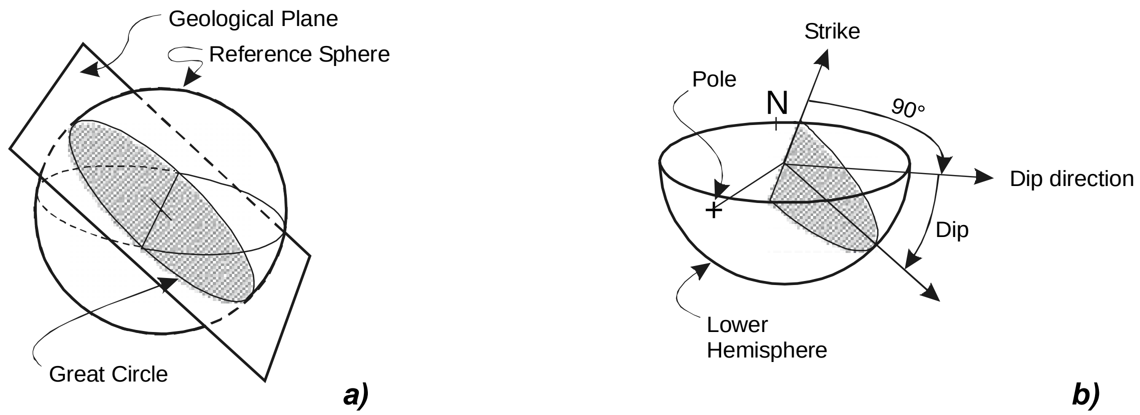


Figure 8.5 Definition of basic terms of the stereographic projection.

To present the great circle and pole two-dimensionally on a sheet of paper, *equal area projection* (also referred to as *Lambert's projection*) or *equal angle projection* is used, see Figure 8.6a. Polar and *equatorial projections* of the reference sphere are shown in Figure 8.6b. Based on whether the equal area or the equal angle projection has been used, the resulting stereonets are also referred to as either equal area (*Schmidt net*) or equal angle (*Wulff net*). Both alternatives may be used for basically the same purposes, but plots may not be transferred directly from one to the other. In engineering geology, the Schmidt net is most commonly used.

For presentation of results from the mapping of discontinuities, and as an alternative to the joint rosette, plotting of poles in *polar projection* is used. An example is shown in Figure 8.7.

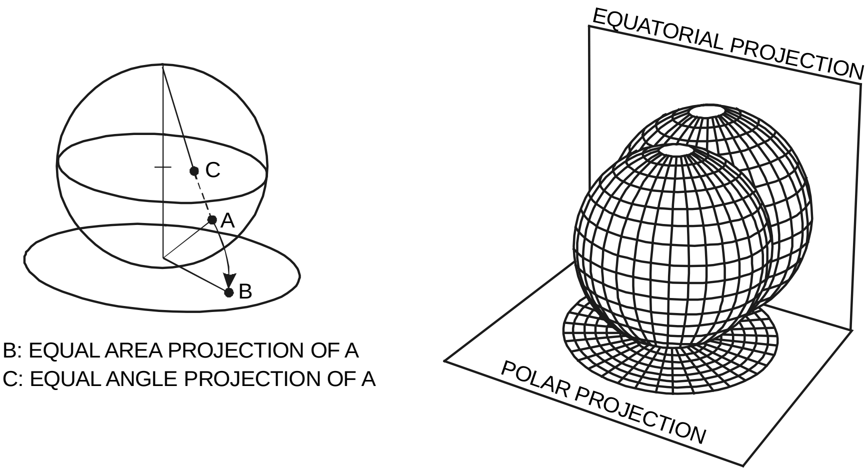


Figure 8.6 Main stereographic projections.

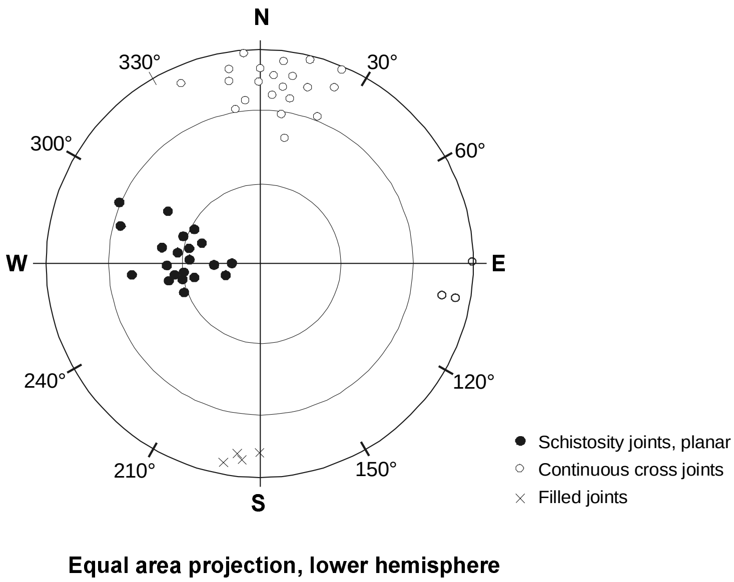


Figure 8.7 Presentation of joint data in a polar projection.

In the example, each point represents the pole of a joint plane, and thus the orientation of the plane. The direction from the pole towards the centre of the stereonet indicates the dip direction of the plane, and the grading along the centre line indicates the dip angle (dip angle = 90° for poles along the periphery, 0° for poles at the centre). In the stereonet, distinct joint sets may be identified as pole-concentrations. In cases with large amounts of data, *contouring* of the pole-plots is sometimes carried out. Pole-plots in the equal area polar projection may be transferred directly to an equal area equatorial stereonet (Schmidt net) of the same diameter.

The Schmidt net is commonly used for analysing structural geology, and may be used also for analysing rock slopes and underground excavations. The use of this tool for defining potential failure modes in slope stability analysis is discussed in Section 10.3.

7.6.3.3 Outcrop confidence

The reliability of the results of the field mapping depends upon the complexity of the geology, the experience of the engineering geologist and the possibility of observing representative rock masses in outcrops. Table 8.3 shows the amount of confidence that may be placed in the rock mass characteristics observed in various geological settings.

Table 8.3 Classification of *outcrop confidence* (from Kirkaldie, 1988).

Term	**Description**
High level	Massive homogeneous rock units with large vertical and lateral extent. History of low tectonic stress levels.
Intermediate level	Rock characteristics are generally predictable, but with expected lateral and vertical variability. Systematic tectonic stress features.
Low level	Extremely variable rock conditions due to depositional processes, structural complexity, mass movement or buried topography. Frequent lateral and vertical changes can be expected. Variable tectonic stress features.

7.6.4 Core drilling and logging

The core that has been recovered from *diamond drilling* or *core drilling* is used to obtain information about rock masses that cannot be observed on the surface. It is one of the most important methods of sub-surface exploration. The usual diameters for the holes and resulting core diameters are shown in Table 8.4. ISRM (1975a) recommends the NX diameter as frequently being the most adequate size for obtaining the quality and quantity of information desired.

Information from *core drilling* is always a valuable supplement to results from outcrop mapping. To further improve knowledge of the underground conditions, such drilling is often also combined with geophysical investigations as shown in Figure 8.8.

Table 8.4 Diameters used in diamond core drilling, based on ISRM, 1975 and Atlas Copco product catalogue 1992, part 2.

American Series (from ISRM)					**Wire Line Drilling** (from Atlas Copco)				
Reference	Hole		Core		Size	Hole		Core	
	mm	in	mm	in		mm	in	mm	in
EX	38	1 ½	22	7/8	-				
AX	49	1 15/16	29	1 1/8	A	48	1.89	26.9	1.06
BX	60	2 3/8	41	1 5/8	B	60	2.36	36.3	1.43
NX	76	3	54	2 1/8	N	75.6	2.98	47.6	1.88
2 3/4 × 3 7/8	98	3 7/8	68	2 11/16	H	96.1	3.78	63.5	2.50
-					P	122.7	4.83	85.0	3.34
4×5 1/2	140	5 ½	100	3 15/16	S	145.9	5.75	108.2	4.26
6×7 3/4	197	7 ¾	151	5 15/16	-				

Conventional drilling (from Atlas Copco)								
Metric sizes			**American sizes**					
Size	Hole	Core	Size	Hole		Core		
mm	mm	mm	mm	In	mm	in	mm	
36	36.3	21.7	-					
46	46.3	31.7 - 35.3	A	1.89	48	1.19 – 1.31	30.0– 33.2	
56	56.3	41.7 - 45.2	B	2.36	60	1.66 – 1.77	42.0 – 44.9	
66	66.3	51.7	-					
76	76.3	47.7 – 61.7	N	2.98	75.6	1.88 – 2.38	54.7 – 60.5	
86	86.3	57.7 – 71.7	-					
101	101.3	71.7 – 86.7	H	3.91	99.2	2.82 – 3.11	71.7 – 79.0	
116	116.3	85.7 – 101.7	-					
131	131.3	100.7 – 108.0	-					
146	146.3	115.7 – 131.7	S	5.76	146.3	4.56 – 4.84	115.7 – 123.0	

The purpose of core drilling in most cases is to:

- Verify the geological interpretation.
- Obtain more information on rock type boundaries and degree of weathering.
- Supplement the information on orientation and character of weakness zones.
- Study the groundwater conditions.
- Provide sample material for laboratory analyses.

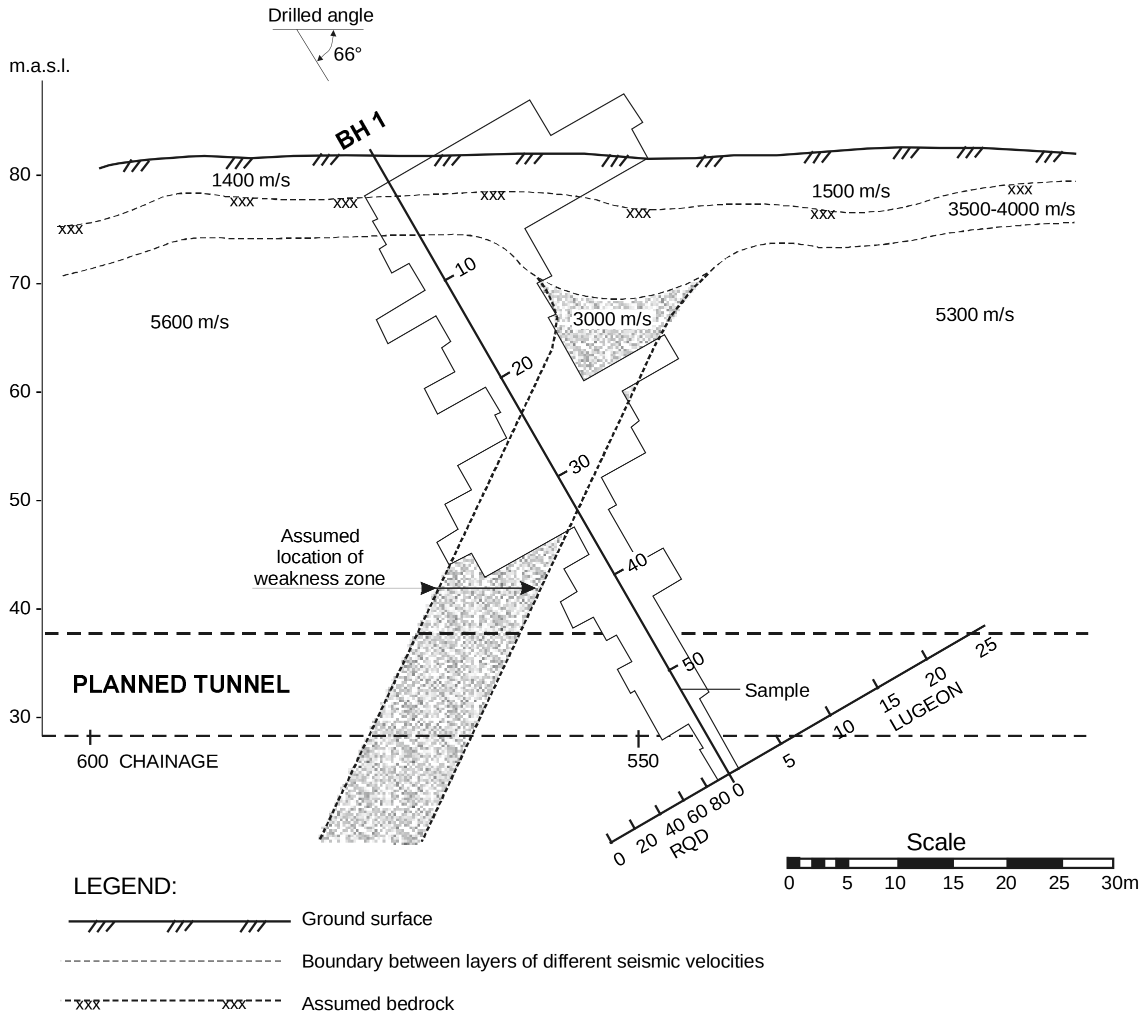


Figure 8.8 Presentation of results from core drilling and refraction seismic investigation. RQD-values along the drill hole and results from Lugeon testing (see Section 8.3.8) are shown (from NBG, 1985).

In hard rocks dominated by discontinuities, core drilling is often carried out as illustrated in Figure 8.8 with the primary purpose of investigating major faults or weakness zones assumed to be crucial for the stability and groundwater conditions of the opening. In such cases the drill holes will, however, also give valuable additional information where they penetrate the adjacent rock mass.

An example of detailed core logging is shown in Figure 8.9. Considering the high cost of good quality *core recovery*, it is in most cases well worth spending a little extra to provide for good routine core examination and carefully prepared reports with high quality photographs of the cores before they are placed in storage.

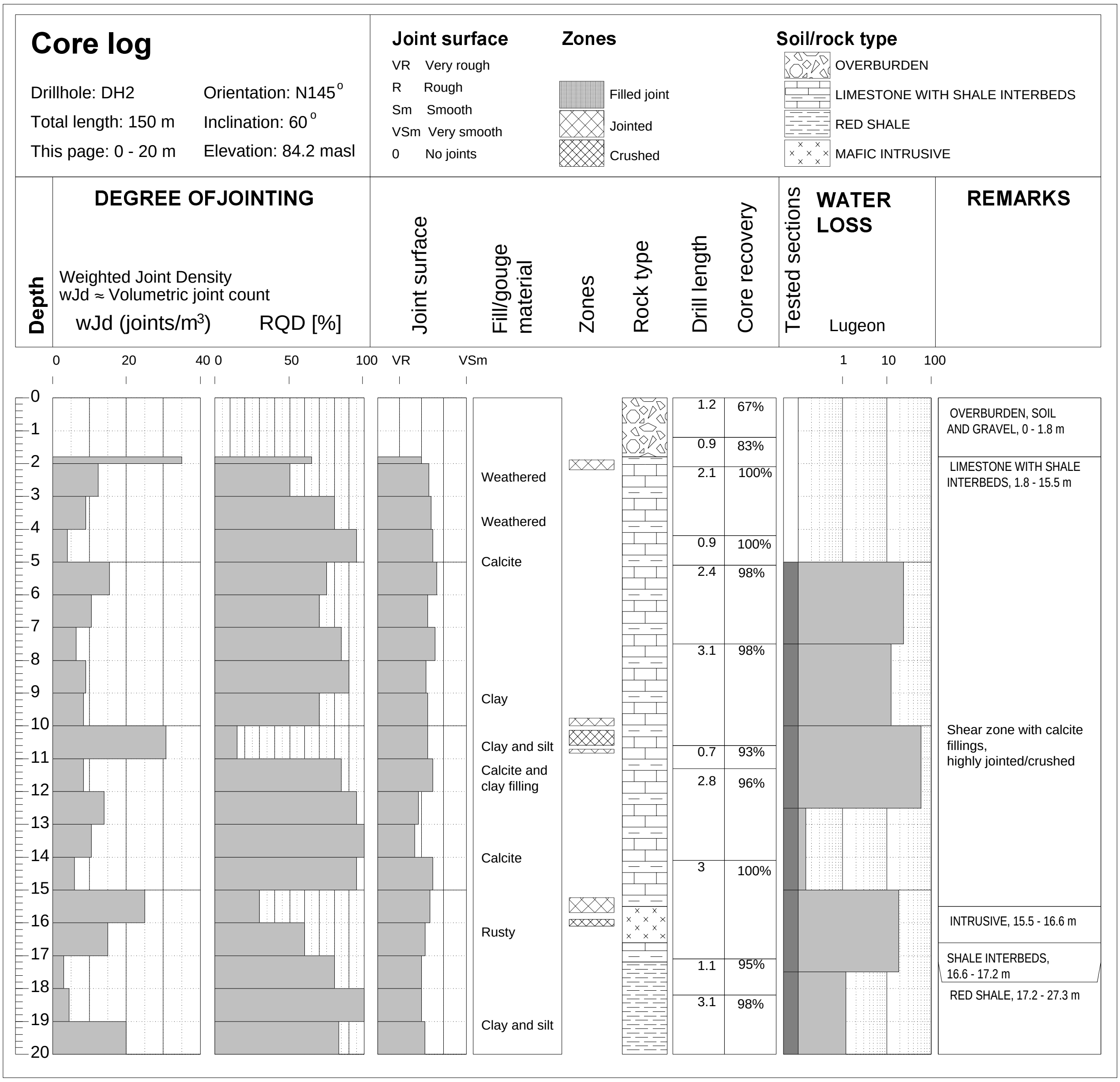


Figure 8.9 Example of core logging.

Groundwater testing in boreholes is described in Section 8.3.8.

7.6.5 Geophysical investigation

The main types of geophysical methods used in rock mass investigation are listed in Table 8.5.

As indicated in the table, the *refraction seismic* method is most commonly used. In most cases it is used for logging the thickness of soil cover or weathering and for evaluating rock mass quality as shown in Figure 8.8, but it may also provide valuable information on more specific issues, such as jointing density as described in Section 3.4.4.

Good quality rock masses below the water table have *seismic velocities* typically higher than 5,000 m/s, while the poor quality rock mass of weakness zones has velocities lower than 4,000 m/s. Typical ranges for seismic velocities in soils and massive rocks are shown in Figure 8.10.

Table 8.5 Summary of relevant geophysical methods.

Method	Main information	Main limitations	Application
Seismic refraction	Thickness of soil layers Location of groundwater table Location of rock surface Location of low velocity zones Quality of rock mass	"Blind zones" (if velocity does not increase with depth) Side reflection	Extensively used on land and sea bottom
Seismic reflection	Locations of different layers (soil, rock, sea bottom, etc.) Soil/rock structure	Penetration Side reflection Interpretation for great depths	Limited use (mainly used for sub-sea tunnels)
Crosshole tomography	Rock mass quality Karst caverns, etc.	Interpretation uncertainty	Increasing use
Electric resistivity	Location of groundwater table/rock surface Character of weakness zones	Interpretation Stray current/buried metal	General use
Electromagnetic (radar)	Location of groundwater table/-soil structure Openings	Restricted mainly to soft ground	Limited use
Magnetic	Structural geology	Interpretation	Minimal use
Gravitational	Structural geology	Interpretation	Minimal use

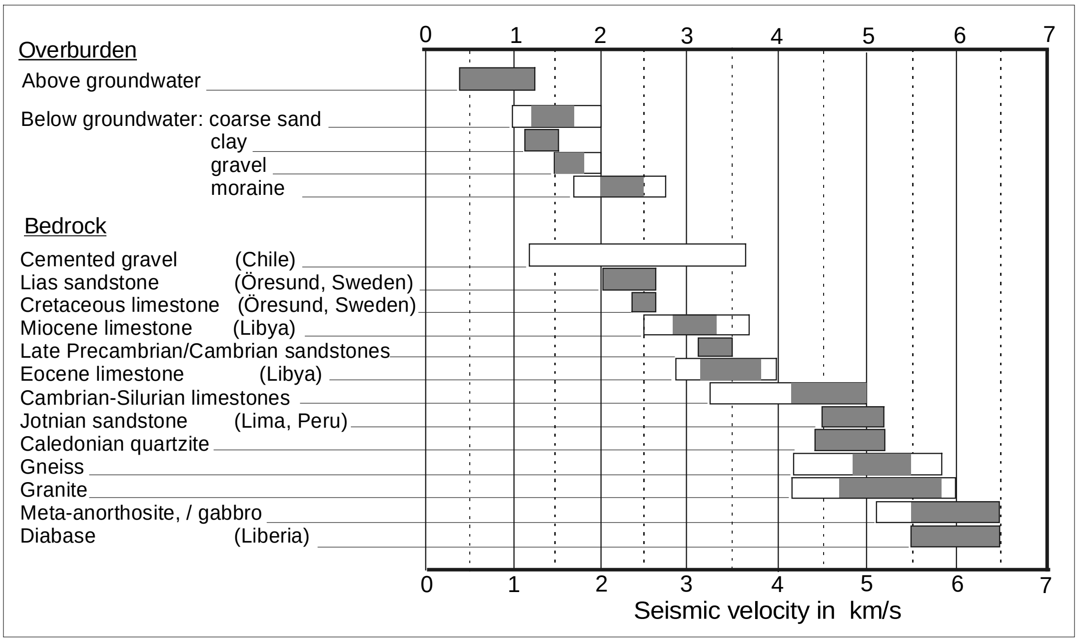


Figure 8.10 Characteristic P-wave sonic velocities (from Sjögren, 1984).

Compared to the values given above, the sonic velocity in air is about 330 m/s, and in water about 1,500 m/s (in ice, it is about 3,500 m/s).

In addition to the variation in rock, the in situ *seismic velocities* in rock masses depend on:

- The acting stresses. There is a general increase of seismic velocity with depth. This is mainly caused by the closing of open joints and cracks. Thus, direct comparisons of velocities at the surface and in the tunnel cannot be made. This feature reduces the ability of refraction seismic measurements to effectively characterise the degree of jointing in deep tunnels.
- The degree of jointing. This effect is an important feature of the interpretation of refraction seismic measurements to assess the block size.
- The presence of open joints or joints with fillings.
- The groundwater conditions.

Thus, seismic methods do not automatically give high quality results in all geological environments. Refraction seismic investigation has particular limitations across deep clefts due to side reflection. When a high degree of accuracy is needed, control boring should be carried out in such topography.

Geophysical methods are often used to supplement information from sub-surface exploration by core drilling and excavation of exploratory adits or shafts, as the generally high costs often limit such exploration.

7.6.6 Exploratory adits and shafts

Due to the general unreliability of projecting geological information obtained from surface mapping to depth, excavation of adits or shafts may be required as part of the site investigations. This is most relevant when very detailed information of the rock mass conditions is required. Rock mass characterisation is carried out by mapping the excavated adit or shaft, and special features or properties (for instance the shear strength of discontinuities) may be measured by large scale field tests.

7.6.7 Rock stress measurements

To analyse the effects of rock stresses, information about the magnitudes and directions of the principal stresses is necessary. Surface indicators such as exfoliation (see Section 3.2.1) and large scale shear failure, or *core discing* (intense parallel fracturing perpendicularly to the core axis) may give a warning of high rock stresses (Myrvang, 1999). However, full information on stress magnitudes and directions can only be obtained by performing rock stress measurements. The rock stresses may vary considerably from place to place, and there is little general correlation between rock cover and stress magnitudes. This emphasises the importance of carrying out stress measurements in each individual case.

Throughout the years a variety of different equipment for in situ rock stress measurement has been developed. Today, the following two methods are most commonly used:

- Triaxial stress measurements by drill hole overcoring.
- Hydraulic fracturing.

The flat jack method is sometimes used as well, although it is not as commonly used as the two other methods and in Norway, two-dimensional overcoring (by the so-called *door stopper method*, see Figure 8.11) is often carried out, particularly for measurements in pillars and in cases of heavy fracturing or core discing.

In the following, the basic principles of the two main methods are discussed. More detailed descriptions of rock stress determination are given in the ISRM Suggested Methods (ISRM, 1987).

7.6.7.1 The overcoring technique

The drill hole *overcoring technique* has the longest tradition, and there are several versions of this method. Figure 8.11 illustrates the principles of the most commonly used version. The method involves the following three steps:

1. A diamond drill hole is drilled to the desired depth. A concentric hole with a smaller diameter is drilled approximately 30 cm further.
2. A measuring cell containing three strain rosettes is inserted, and the rosettes are glued to the walls of the small hole.
3. The small hole is overcored by the larger diameter bit, thus relieving the core of stresses. The corresponding strains are recorded by the rosettes. When the elastic constants are known, the triaxial state of stress can be computed.

Thus, what is actually being recorded, are the strains. To be able to compute the stresses, laboratory analyses of the elastic properties have to be carried out.

The overcoring technique (and flatjack testing) is normally used in underground openings, although triaxial overcoring has in a few cases also been carried out from the surface in 20-30 m deep drillholes.

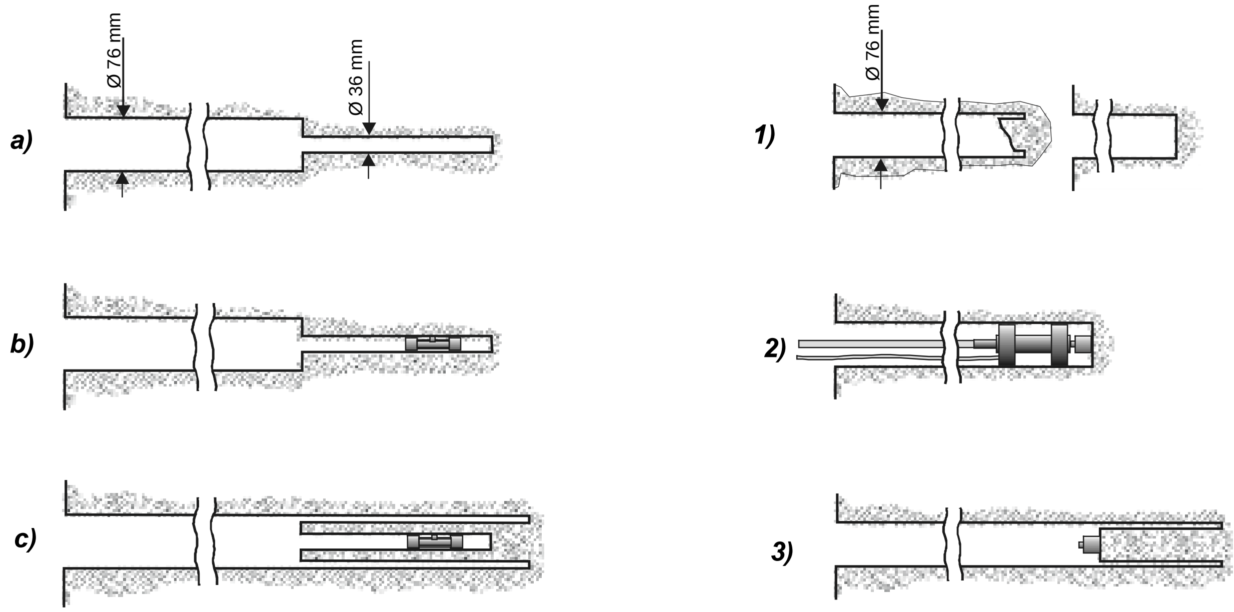


Figure 8.11 The principle of three-dimensional (to the left) and two dimensional (to the right) rock stress measurements by overcoring (after Myrvang, 1983).

7.6.7.2 Hydraulic fracturing and jacking

The basic principle of *hydraulic fracturing* is to isolate a section of a drill hole and, by gradually increasing the pressure of water which is pumped into the hole, to obtain fracturing of the surrounding rock. By recording water pressure and flow, the principal stress situation may be evaluated. Hydraulic fracturing is the only rock stress determination technique that is successfully applied to deep drill holes.

No standard for pressurisation rate or flow rate exists; however, a common range of pressurisation rates according to ISRM (1987) is about 0.1-2.0 MPa/s. The pressurisation rate is controlled by the constant flow rate selected.

An idealised hydraulic fracturing record is shown in Figure 8.12. The drill hole fluid pressure at the moment of rupture is termed the "*fracture initiation pressure*" (p_f) or *breakdown pressure*.

After injecting a volume sufficient to propagate a fracture length about three times the drill hole diameter, injection is stopped and the hydraulic system is sealed or "shut-in", yielding the "*instantaneous shut-in pressure*" (p_s). Additional re-pressurisation cycles are used to determine the "*fracture reopening pressure*" (p_r) and additional measurements of the shut-in pressure (p_s).

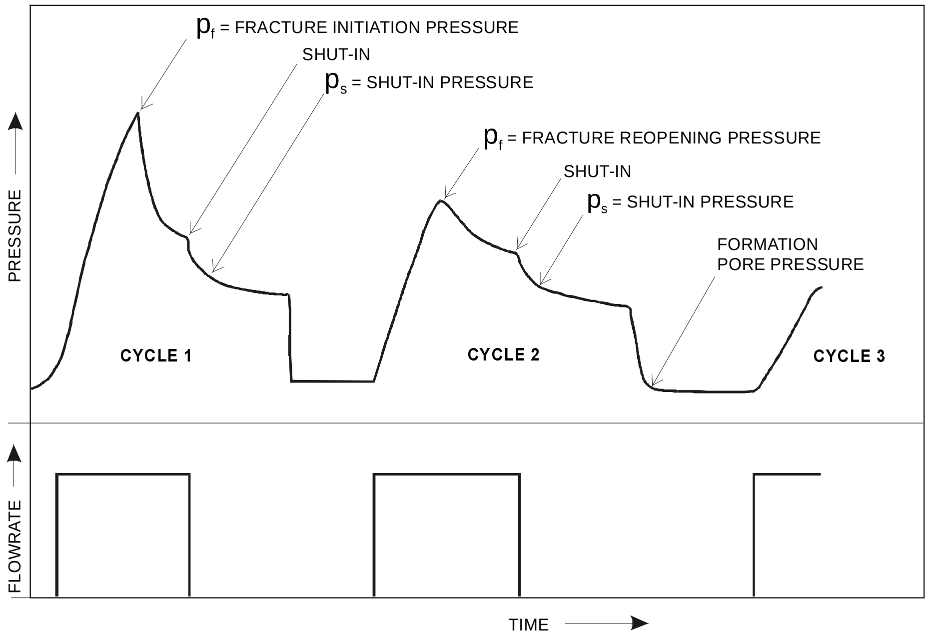


Figure 8.12 Idealised hydraulic fracturing pressure record (from ISRM, 1987).

To be able to calculate stresses, the drill hole direction has to be a principal stress direction. Usually, this assumption is considered valid for vertical holes drilled from a horizontal surface. In such cases, the vertical stress is calculated from the overburden weight, and when the plane of hydrofracturing is nearly parallel to the drill hole axis, the following expressions may be used to obtain the horizontal stresses (ISRM, 1987):

$$\sigma_{min} = p_s$$
$$\sigma_{max} = T + 3p_s - p_f - p_0 \qquad \text{(for initial pressurisation cycle)}$$
$$\sigma_{max} = 3p_s - p_r - p_0 \qquad \text{(for subsequent pressurisation cycles)}$$

where σ_{min} = minimum horizontal stress.
σ_{max} = maximum horizontal stress.
T = drill hole rupture strength of the rock.
p_0 = initial pore water pressure.

Often, this test is used mainly as a "pilot-test" for evaluating the tightness of the rock mass when exposed to high pressure water or gas, and the main emphasis in such cases is to apply hydraulic jacking to unfavourable joints. The main principle of such testing is to increase the water pressure gradually in the test section, and to record the water flow as a function of pressure as shown in Figure. 8.13

7.6.8 Groundwater measurements

The main reason for in situ testing in this connection is to obtain a better basis for evaluating potential water leakage. In some cases test results are needed as direct input in analyses. During excavation the main reason for testing is often to evaluate the need for grouting.

There are a great variety of techniques for testing *permeability* and water flow. Here, the discussion will be restricted to brief comments on the following main testing principles:

- The Lugeon test.
- Water pressure (piezometer) measurements.
- Water inflow registration (mainly carried out during tunnelling, and described in Section 8.3.11).

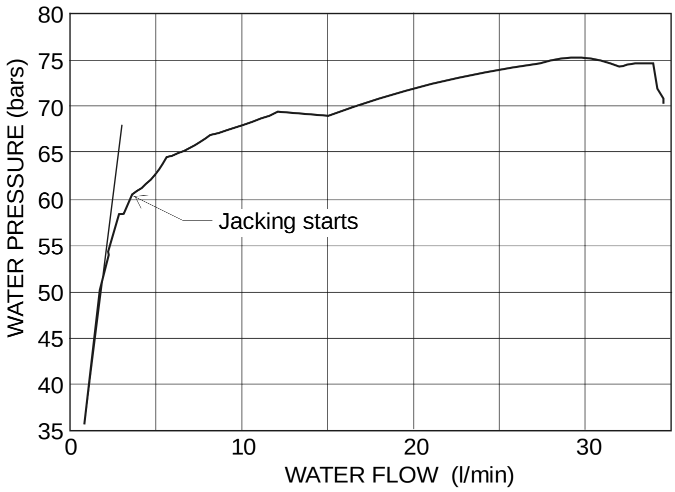


Figure 8.13 Result of hydraulic jacking test (from Nilsen & Thidemann, 1993).

In addition, the pumping test is used for measurement of groundwater flow and ground conductivity.

7.6.8.1 Lugeon tests

The unit 1 *Lugeon* (1 L) is defined as the loss of water in litres per minute and per metre borehole at an over-pressure of 1 MPa (= 10 bars).

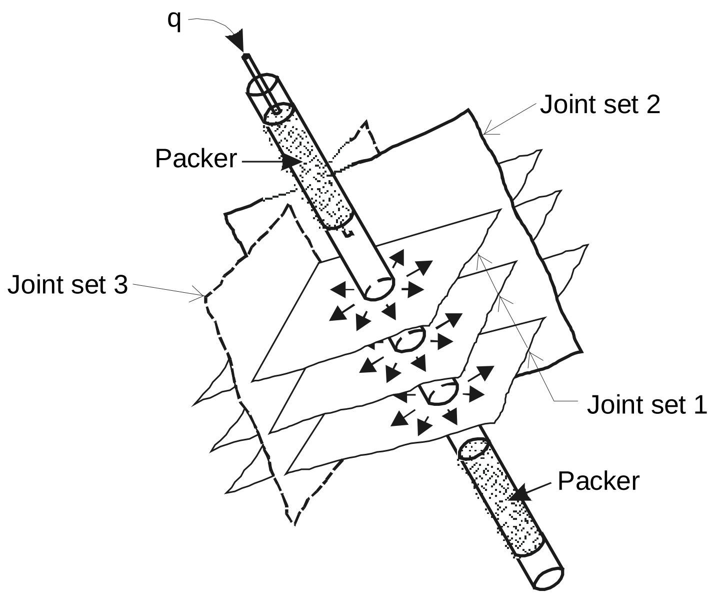


Figure 8.14 The principle of Lugeon pressure testing.

In the *Lugeon test* or *water loss test,* water is pumped into a section of a borehole, and the loss of water is measured, see Figure 8.14. Originally this test was introduced to obtain a basis for evaluating the need for grouting in rock masses under dams.

The actual borehole section should have a length of approximately 3 m and is often between two packers as shown in Figure 8.14, allowing a stepped testing of the entire hole length. By using a single packer the end part of the borehole can be tested. This is normally performed while the hole is drilled. This measurement is generally considered more accurate, as leakage in one packer is less probable than in two.

Each section is put under a constant overpressure of 1 MPa relative to the original groundwater pressure. The pressure is controlled by a gauge on the water hose at the opening of the hole, and the loss of water during a 5 minutes period is measured with a flow-meter. The (over) pressure in MPa (=10 bars) is found from:

p_w = 1/100 of vertical depth (m) to groundwater level from surface + 1/100 of height (m) of pressure gauge above ground + pressure measured (in MPa).

The water level in the borehole above the packer should be observed during each test, as a rising level may indicate that leakage is occurring around the upper packer.

Where a test has been conducted at pressure heads considerably less than the standard 10 bars, the Lugeon value may be somewhat overestimated by the above formula, due to possible differences in energy loss between laminar flow (at low pressure) and turbulent flow (at high pressure).

Another method is to run a *staged Lugeon test* at each location in the borehole, using different pressures. A five-stage test is desirable, with the pressure applied in three equal increments and then reduced with decrements of the same amount, see Figure 8.15. The data obtained from these measurements are particularly useful in assisting the interpretation of the behaviour of the rock under testing.

The varying values of pressure and flow recorded during the test may be plotted as shown in Figure 8.15. The interpreted Lugeon value is given by the formula (Geoguide 2, 1987):

$$L = \left(\frac{p_L}{L}\right)\left(\frac{q}{p_w}\right)$$

where p_L = the standard pressure (1 MPa) for Lugeon test
L = length of the test section
q = the flow rate (l/min)
p_w = overpressure (MPa) during the test
q/p_w = slope of the graph in Figure 8.15

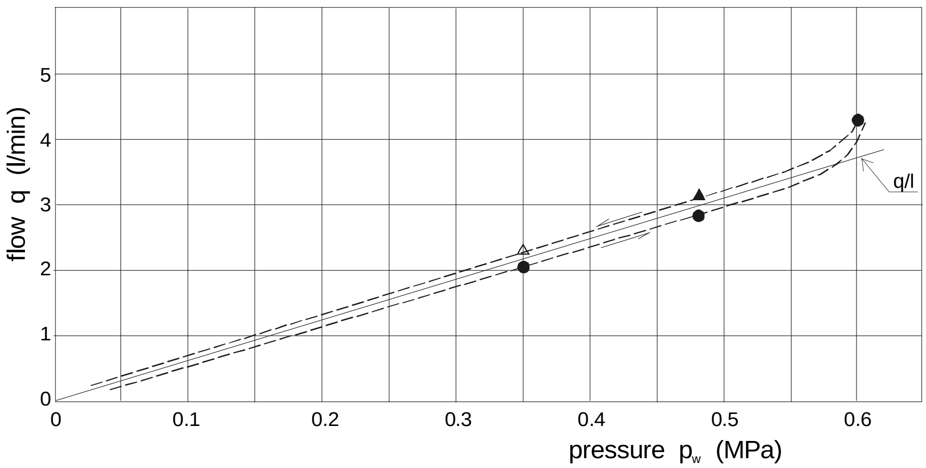


Figure 8.15 Example of plotting results from the staged Lugeon test (from Geoguide 2, 1987).

Based on a pumping test of one single borehole in isotropic conditions, the following approximate relationship between the Lugeon value and the hydraulic conductivity (k) has been presented by Hoek and Bray (1981):

$$k = \frac{1.4q_c}{L \times H_c}$$

where: q_c = pumping rate (l/min) necessary to maintain a constant over-pressure.
L = length (m) of the test section.
H_c = over-pressure (constant head, given in metre).

According to this equation a water loss of 1 L corresponds to a hydraulic conductivity of $k = 2.3 \times 10^{-7}$ m/s. This approximation is based, however, on highly idealised conditions. For complex conditions, and if more exact figures are needed, more advanced analyses should be carried out.

The Lugeon test results can be markedly influenced by single joints, and the results can, therefore, be misleading. There are also often uncertainties connected to the execution of the test, for instance leakage through the packer. This is especially true for double packers.

7.6.8.2 Water pressure measurement

Data on in situ *water pressure* are needed when performing the Lugeon test. Such data are also necessary in other tests as, for instance, the hydraulic fracturing test, and in some cases reliable data on in situ water pressure is of vital importance when carrying out rock stability analyses.

When the conductivity is high, the water pressure may be measured by simply closing a borehole section and attaching a manometer to the water hose as explained for the Lugeon test. In a low conductivity rock mass data may be obtained by measuring the pressure drop of a pressurised borehole against time as shown in Figure 8.16.

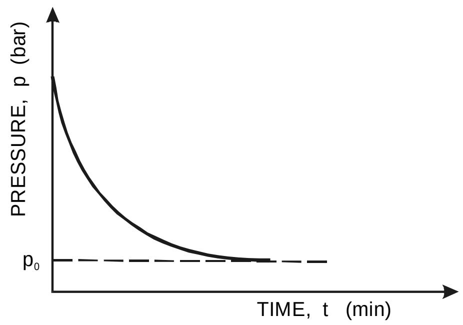


Figure 8.16 Joint water pressure p_o measured by the pressure drop method (from Nilsen and Thidemann, 1993).

7.6.9 Other tests and measurements

As part of the field investigations, a variety of other tests and measurements may be relevant depending on local conditions such as:

- In situ shear strength testing of joints.
- Convergence measurement in excavations (by tape extensometer or borehole extensometer).
- Measurement of in situ deformability of rock masses.
- Geophysical logging of boreholes.

Recommendations for performance of such tests and measurements are given by ISRM, see Chapter 18.

7.6.10 Special requirements for sub-sea tunnel investigations

Sub-sea tunnels require an expanded investigation programme because of their complexity and the obvious problems with surface mapping. The depth of the seabed, thickness of loose materials, and rock mass quality are significant factors affecting the design of sub-sea tunnels.

At the *main planning stage*, shallow *reflection seismics* (so-called *acoustic profiling*) together with echo sounding for sea floor mapping are carried out in order to map the seabed and the thickness of loose materials covering the bedrock. Normally, a coarse network covering the area that is of interest is investigated first. After an evaluation of the tunnel alignment, a finer network is investigated along the selected corridor. Based on information from these investigations, refraction seismic measurements are made in areas of possible uncertainties.

At the *detailed planning stage* it will, as a rule, be necessary to supplement the refraction seismic measurements with profiles along the tunnel line and cross profiles to identify weakness zones and to check the location of the rock surface. Core drilling should be carried out to map the joint density, possible leakage and the character of faults and weakness zones. The core drill hole can also be used to perform further geophysical measurements, e.g., seismic tomography, resistivity measurements, etc.

In special cases, where it is essential to know the location of the rock surface, drilling with sampling from a ship should be considered.

7.6.11 Investigations performed during tunnel excavation

Exploratory drilling is carried out to obtain information on the rock mass conditions ahead of the tunnel working face, such as leakage, faults and major weakness zones. It is important that such features are discovered sufficiently far from the face for appropriate measures to be made for a safe tunnelling through them. Sometimes, exploratory drilling is also used to check the rock cover.

Normally, the tunnel jumbo with percussive drill is used for the *exploratory* or *probe drilling*. Core drilling is used where particularly difficult rock conditions are expected. The extent of necessary probe drilling depends on the rock conditions expected, rock cover and experience gained so far on the job. The number of drill holes can be increased where zones of weakness or other poor rock mass conditions are expected, or where there is a risk of leakage or a need to check the rock cover.

In some cases, *Lugeon testing* is carried out in the probe drill holes and decisions concerning grouting are based on the results of this test. However, in most cases the decision is simply based on measuring the volume of leakage water from the holes, and grouting is carried out if the leakage exceeds a pre-defined limit. For sub-sea tunnels this limit is commonly 5 l/min for one hole of 20 to 25 m length.

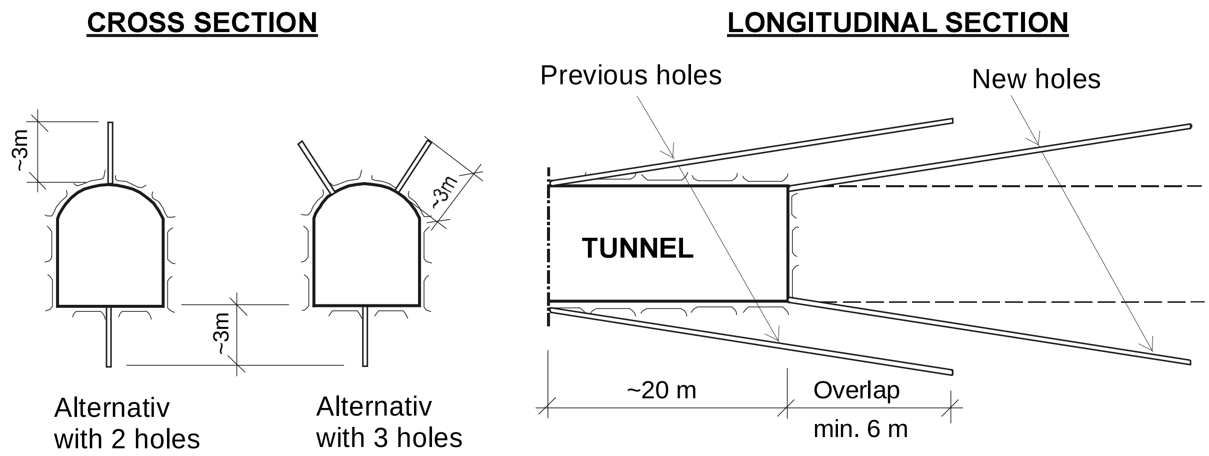


Figure 8.17 Examples of common procedures for percussive probe drilling.

7.7 Laboratory testing

Representative samples are tested as an important part of the investigation programme. The types of tests and extent of testing depends much on the character and complexity of the project. An overview of common tests is given in Table 8.6.

7.8 Uncertainties and errors

Due to the complex character of geological materials there is always an element of uncertainty in geological investigation and testing. In this connection the following terminology is often used:

- *Uncertainty* or lack of absolute sureness in geology means that observations, measurements, calculations and evaluations are not reliable. As a consequence, the use of geological data often may involve some form of guesswork.
- *Error* is defined as the difference between computed or estimated result and the true value.
- A *bias* is the difference between the estimated value and the true value based on statistically random sampling. For example, joints sub-parallel to an outcrop have less chance of being sampled than joints perpendicular to an outcrop. This is a bias in sampling for orientation.

Table 8.6 Common laboratory tests according to Norwegian practice.

Testing/ Investigation of	Method	Sample	Reference
Mineral composition	*Microscopy* *Differential-thermic analysis (DTA)* *X-ray diffraction analysis (XRD)*	Thin section Powder Powder	
Rock strength -compressive -compressive -tensile -brittleness	 *Uniaxial compressive strength test (UCS)* *Triaxial strength test* *Point load test* Brittleness test	 Drill cores (/cubes) Drill cores (rock), soil sample Drill cores(/irregular specimens) Aggregate (8-11.2 or 11.2-16 mm)	 ISRM 14/Section 2.3 ISRM 20/Section 2.3 ISRM 30/Section 2.3 Section 2.7
Rock elasticity -Young's modulus -Poisson's ratio -E_{dyn}	 Uniaxial compression Uniaxial compression Sonic velocity	 Drill cores Drill cores Drill cores	 Section 2.6 Section 2.6 ISRM 19
Discontinuity shear strength	*Tilt test*	Drill cores or blocks (intersected by discontinuity)	ISRM 15
Gouge material *-mineral composition* *-swelling*	DTA-analysis XRD-analysis Electron microscope Colour test See swellability	Powder Powder Intact material Intact/powder	
Drillability	*Brittleness test* *Siever's J-value* *Abrasiveness*	Aggregate (11.2-16 mm) Sawn specimen Powder (- 1 mm)	Section 2.7 Section 2.7 Section 2.7
Blastability	Sonic velocity Point load strength Density	Drill cores Drill cores Drill cores/aggregate	ISRM 19 ISRM 30 ISRM 12
Swelling potential	Odometer test Hygroscopic properties Free swell	Fraction < 20 µm Fine fraction Fraction < 20 µm	Section 2.7 Section 2.7
Grain size distribution	Sieving Settling	Coarse and intermediate grained material Fine grained	ISRM 13 ISRM 13
Applicability as road aggregate	Brittleness test Flakiness test Los Angeles test	Aggregate (8-11.2 mm) Aggregate (5.6-8 mm) Aggregate (20 mm)	Section 2.7
Applicability as concrete aggregate	Brittleness test Flakiness test	Aggregate (8-11.2 mm) Aggregate (5.6-8 mm)	Section 2.7

The main types of measurement errors are:

- *Gross errors*, attributable mainly to blunders on the part of the observer, are usually large in magnitude and irregular in occurrence.
- *Systematic errors* arise when measurements tend to be consistently either too large or too small. They may come from a mis-calibrated instrument.
- *Errors of method* may occur when inappropriate methods are applied in the quantification of geology.

The following factors probably represent the most common uncertainties and measurement errors in engineering geology:

- Rock property tests for which, apart from some simple physical property tests, few methods give quite reliable data. The main reasons for the inadequacy of test results – as is accepted in most rock engineering design - are the complexity and variability of rocks and rock masses.
- Strike and dip measurements, which are often connected with significant errors, particularly when the actual plane is close to horizontal.
- Poor characterisation of properties and/or poor interpretation.

- Joint mapping, for which some significant errors due to subjective selection of joints often are:
  - Small joints tend to be disregarded.
  - Very large fracture surfaces may be measured more than once.
  - Joints sub-parallel to the foliation or bedding may be overlooked.

In addition, there is always uncertainty due to *omissions.* Many of the major failures of constructed facilities have been attributed to this.

All these aspects have important consequences in rock engineering and construction design. The main conclusions of this chapter are therefore:

1. Although extensive field investigation and good quality descriptions will enable the engineering geologist to predict the behaviour of a tunnel more accurately, it cannot eliminate the risk of encountering unexpected features.

2. A good quality characterisation of the rock mass will, however, in all cases except for wrong or incorrect interpretations, improve the quality of the geological input data for evaluation and analyses, and hence lead to better design.

3. The methods, effort and costs of collecting geo-data should always be balanced against the probable uncertainties and errors.

4. As a consequence of conclusions 1-3, flexibility in tunnelling contracts by a risk-sharing system (see Section 9.4) has obvious advantages.

8 Stability and rock support of underground openings

Stability is a relative term, as it may be connected to the required level of safety, which will vary with the use of the excavation. The level of safety may also be different in the various countries according to regulations for working conditions and safety requirements, as well as the experience of the contractor. ***Failure*** is the result of ***instability***. Both failure and instability are used rather inconsistently in the literature as they often overlap. ***Competent ground*** is a rock mass or soil having higher strength than the stresses acting on it.

8.1 Instability and failure modes

The stability of rock excavations depends mainly on:

- Rock mass quality,
- Rock stresses,
- Groundwater,
- Size, geometry and orientation of the excavation.

Concerning the rock mass properties, stability is mainly influenced by:

- A combination of unfavourable joints (unstable blocks),
- Weakness zones and faults with crushed rock with or without clay, showing especially low strength,
- Swelling of clay materials in zones, resulting in additional reduction of the stand-up time,
- Slaking or deterioration of the rock material caused by water saturation or moisture.

Basically, the instability of the ground, i.e., the rock masses surrounding an underground opening, may be divided into two main groups (Hudson, 1989):

1. *Block failure*, where pre-existing blocks in the roof and sidewalls become free to move as a result of excavation work. These may be called "structurally controlled failures", involving a great variety of failure modes (such as loosening, ravelling, block falls, etc.), see Figure 9.1.
2. *Stress failure*, which is induced by *overstressing*, i.e., the stresses developed around the opening exceed the local strength of the rock masses, see Section 5.3. This may occur in two main forms:
   a) Overstressing of *massive* or intact rock which occurs as spalling, popping, rock burst, etc. in brittle, hard rocks and *squeezing* in ductile, deformable rocks.
   b) Overstressing of *particulate materials*, i.e., heavily jointed rocks where squeezing and creep may take place.

The same type of ground will behave differently for different sizes of openings compared to the size of the blocks. It is important to evaluate whether the ground will behave as a *continuous, bulk material*, or if the failure may be caused by the character of the individual blocks in *discontinuous ground*. To determine this, the continuity factor (see Section 4.2):

$$CF = \frac{\text{tunnel diameter}}{\text{block diameter}} = \frac{Dt}{Db}$$

can be used to select the method appropriate for estimating the amount of rock support.

Continuous ground occurs in *massive rock* where CF < approx. 5 and in *particulate materials* where CF > approx. 100. Where 5 < CF < 100 the ground is ***discontinuous*** (blocky).

The main types of instability are shown in Figure 9.1, and some important definitions are given in the following.

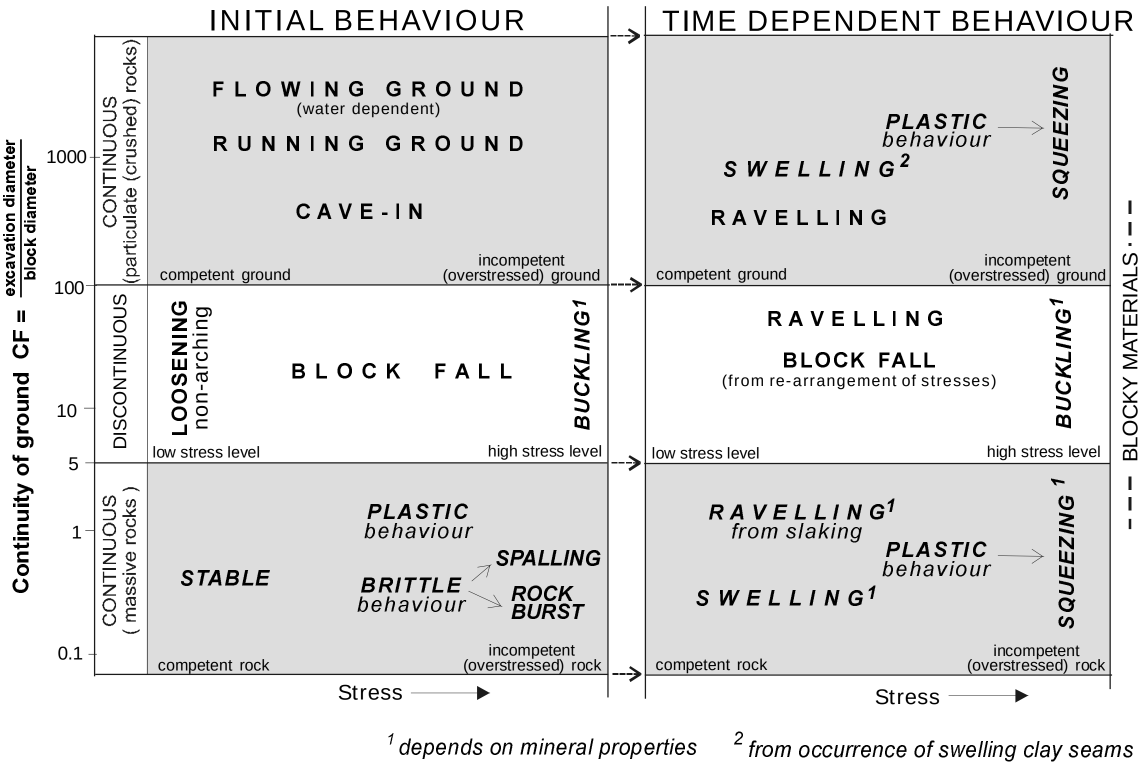


Figure 9.1 Modes of instability and rock mass behaviour (from Palmström, 2000).

Squeezing is time dependent deformation, essentially associated with creep caused by exceeding a limiting shear stress. Deformations may terminate during construction or continue over a long period (ISRM, 1998). Squeezing rock slowly advances without perceptible volume increase, and at almost constant water content.

Flowing ground is a mixture of water and solids invading the tunnel from all sides, including the bottom. It is encountered in tunnels below groundwater table in crushed or soil-like materials with little or no coherence.

Running ground occurs when a material invades the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero. Examples are clean, medium to coarse sands and gravels ***above*** groundwater level.

Ravelling ground indicates a material, which gradually breaks up into individual pieces, flakes, or fragments. The process is time-dependent and materials may be classified by the rate of disintegration as slowly or rapidly ravelling. For a material to be ravelling it must be moderately coherent and friable or discontinuous. Examples are jointed rocks, fine moist sands, sand-gravel with some binder and stiff fissured clays.

Block fall refers to individual blocks falling (< 10 m^3).

Cave-in refers to larger volumes (> 10 m^3) of rock debris falling in.

Disintegrate is to break or decompose into constituent elements, parts, or small particles.
Buckling is the breaking out of fragments along the surface of a column or tunnel wall under sufficiently high load due to deflection of the structure.
Slaking is breaking-up or disintegration of a rock or soil when exposed to moisture, saturated with or immersed in water.
Popping or *spalling* is the sudden, violent detachment of thin rock slabs from sides or roof caused by overstressing of hard, brittle rock.
Rock burst is caused by the same mechanism as popping/spalling, but is much more violent and involves considerably larger volumes (often registered as seismic events).
Swelling ground is ground advancing into the tunnel due to expansion caused by water adsorption. The ability to swell is limited to rocks containing clay minerals such as montmorillonite, and to rocks with anhydrite.

8.1.1 The time-dependent effects

The effect of time plays an important role when stability and rock support are evaluated, especially when poor stability (short stand-up time) conditions are encountered at the face.

When *time-dependent behaviour* of soil or rock around an underground opening is considered, there are two separate influences:

1. Short term, due to the variation of the stress field as the face advances away from the point concerned in the tunnel. Significant changes in tunnel stability may occur as a result of readjustment of stresses in the walls and roof of a tunnel as the face advances. The deformation and stress re-distribution after excavation requires time. The stability effect of this is represented by the *"stand-up time",* introduced by *Lauffer* (1958). Lauffer showed that the following features are related to each other:
   - The property of the rock mass,
   - The active span of the excavation, and
   - The time elapsed until unstable conditions occur.

   These relations, which are key-points in stability assessment, have been applied in both the *New Austrian Tunnelling Method* (NATM), and the *RMR* system.

2. Long term. The main geologic factors here are:
   - The *creep* factors, i.e., creep under constant shear stress. In highly stressed rocks the effect of long-time creep may change the strength of the material as described by Lama and Vutukuri (1978).
   - The influence from the environment.
   - The *durability* of the rock mass, for example the *slaking* of mudrocks.
   - The content of *swelling* minerals
   - The effect of groundwater.

The influence of possible alteration of rock, joint filling/gouge, or swelling, softening and weakening along discontinuities are factors that must be specially evaluated in each case. These effects are not necessarily obvious during construction, therefore, the long time stability may easily be underestimated during the construction period.

8.1.2 Effect of the shape of the opening and excavation method

It is commonly accepted that any method used to excavate a tunnel will cause some disturbance of the surrounding rock structure, which in turn will affect the stability. The various excavation techniques used may influence the tendency of blocks to loosen and fall out of the tunnel walls or roof differently. For example, mechanical tunnel excavation would tend to disturb the blocks much less than drill-and-blast excavation.

For excavations exposed to high rock stresses, Selmer-Olsen (1988) points out the importance of excavating an opening of a simple shape, without ledges and overhang, to reduce the amount of loosening and spalling. He shows that by shaping the tunnel with a reduced curve radius in the roof where the largest in situ tangential stress occurs, it is possible to reduce the area of over-stressing and hence the extent of instability.

Also, in jointed rocks without *overstressing*, the stability is improved where a simple shape of the opening is chosen omitting sharp corners, recesses, etc. The shape of the opening (horseshoe, circular, etc.) has a significant influence on the magnitude of the stresses set up in the rock mass surrounding the opening. Further, the amount of rock support depends significantly on the size of the opening.

In most cases, it is very difficult to distinguish between the “before” and “after” conditions and decide what impact excavation may have had on the amount of rock support necessary. Fracturing from blasting leads to reduced block size. Actual loosening of rock caused by vibration is, however, often described as “overbreak” rather than an additional support require-ment. The influence from blasting can be substantially reduced by controlled *perimeter blasting*.

8.1.3 Special behaviour related to faults and weakness zones

Special behaviour related to faults and weakness zones can have a major impact upon the stability of the opening as well as on the excavation process. Possible flowing and running ground, as well as high groundwater pressure and several other phenomena connected with such zones often make special investigations necessary. Such zones require special attention in underground works because their structure, composition and properties may be quite different from the surrounding rock masses.

An important inherent property influencing stability is the character of the *gouge* or *filling material*, as shown in Table 9.1.

Joint filling and fault gouge are normally impervious, with a major exception for sand-like gouge. Thus, high permeability may occur in the jointed rock masses adjacent to the fault zone. High water inflows encountered in underground openings when excavating in weak impervious gouge, is one of the most adverse conditions associated with faults.

8.2 Rock support

Rock support is provided to improve the stability in an underground opening. A main principle is to design the rock support in accordance with the actual ground conditions encountered in the tunnel. This requires flexible support methods, which can be quickly adjusted to meet the continuously changing quality of the rock masses. Such flexibility is achieved by the use of rock bolts, shotcrete, and cast-in-place concrete lining, either alone or as integral elements of the support.

Table 9.1 Various filling and gouge materials and their potential behaviour (modified after Brekke and Howard, 1972).

Dominant material in filling (or gouge)	**Characteristic behaviour**
Swelling clay	*Swelling, sloughing and squeeze. Stability problems with rock falls, slides, and in some cases, collapses have been caused by the swelling of clays in faults.* For swelling clays (smectite) the initial and later change of water content can be important for the mobilised swelling pressures. In addition, swelling clays have a low shear strength.
Inactive clay	*Slaking and sloughing caused by squeeze.*
Chlorite, talc, graphite, serpentine	*Ravelling.* The characteristic property of such gouge is low shear strength, in particular when wet.
Porous or flaky calcite, gypsum	*May dissolve.* Leads to reduced stability.
Quartz, epidote	*Durable, high strength, often with a "welding" result.* Improves stability.
Crushed rock fragments (gravel size) or sand-like filling	*Ravelling or running.* A significant problem may be caused by materials that have been *crushed* to an almost cohesionless (sand-like) material that may run or flow into the tunnel immediately after blasting.

The rock supporting works are normally carried out in two main stages:
1. The *initial support*, which is installed to secure safe working conditions for the tunnelling crew. The types and methods of support to be used should be decided in agreement between the engineer and the contractor. The contractor is responsible for the initial support: in practice the working crew decides the amount of rock support necessary for their own safety. The initial support should be designed to constitute a part of the permanent rock support.
2. The *permanent support*, which is installed to meet the requirements for a satisfactory function of the project during its life. The owner determines the final rock support. Usually, he and his consultant decide both the methods and amounts of support after excavation.

The active use of engineering geologists in tunnelling is important, not only to solve rock stability problems during construction and recommend rock support, but also to predict the probable rock conditions to be encountered and to map the rock mass conditions along the tunnel or cavern.

Some of the main methods of *rock support* are described in the following. Emphasis is placed on methods commonly used in the Nordic countries. Other methods, which may be commonly used outside of the Nordic countries are not included in this text (steel-ribs, wooden lagging, etc.).

8.2.1 Scaling

Scaling of loose rock at the contour of the underground opening is carried out after each blast round and also periodically at later stages. Manual scaling (with a crowbar) is still a common method, but is often combined with mechanised scaling. Mechanised scaling makes the work safer and more efficient, especially for larger cross sections.

Manual scaling is carried out directly from the muck pile, and in larger tunnels, often from a wheel loader. For small and medium scale tunnels, the average scaling time is normally 15-30 minutes. Thorough scaling is important for the safety of the crew during the drill and blast process.

Scaling is regarded as part of the rock support since blocks can be scaled down instead of being supported by rock bolts or shotcrete. This is the case where so-called *"extra scaling"* (more than for example 1.5 man-hours scaling per round) is necessary, for example in highly jointed and crushed rock and where rock bursting occurs.

8.2.2 Rock bolts

Rock bolting is a flexible method very commonly used for rock support. In Norway, approximately 500,000 bolts are installed annually. Rock bolts are frequently used as initial support at the tunnel face to obtain safe working conditions for the crew and they also form part of the final support. For systematic bolting, automatic, high-capacity rockbolting jumbos are sometimes used. Usually however, the tunnel jumbo is employed to drill bolt holes. As a measure of capacity, 50-150 bolts can usually be installed per shift (7.5 hours) by a crew of 1 or 2 men.

Rock bolts are installed in tunnels and caverns according to one of the two main principles shown in Figure 9.2: systematic or spot bolting.

Local rock bolts are installed individually to fix single, loose blocks

Systematic rock bolts are installed in a certain pattern as a more general support in unstable areas.

Figure 9.2. The two main applications of rock bolting.

Except for situations where rock burst occurs, expansion and resin anchored bolts are normally tensioned to 25-50% of their yield strength. In situations where rock burst occurs, minimum tensioning is applied and large steel plates are used to avoid crushing the surrounding rock. When the rock stresses are very high it is important to apply rock bolts with a high yield capacity.

For rock bolting behind the working face, untensioned, grouted bolts (dowels) are commonly used as shown at the bottom part of Figure 9.3. These are completely surrounded by grout, and hence well protected against corrosion. As an alternative to the expansion shell, resin anchoring is more commonly used in many countries today.

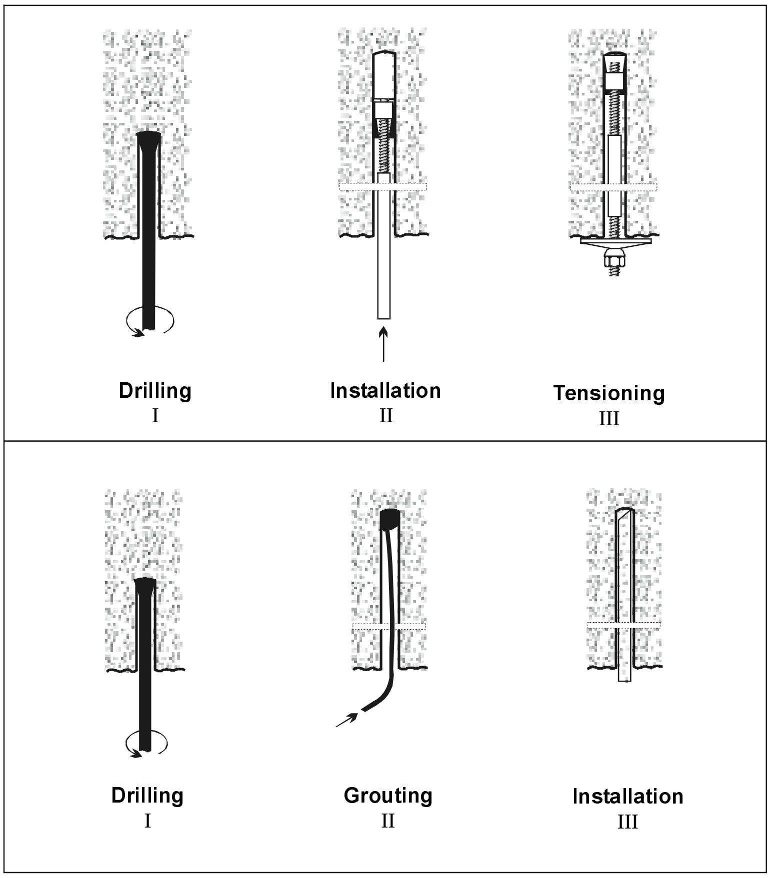


Figure 9.3 Principles for installing the two main types of rock bolts.
Upper part. The *point anchored bolt* (anchored here by expansion shell)
Lower part. The *grouted bolt*.

Friction bolts are rock bolts with the support capacity represented by the friction between the drill hole wall and the rock bolt along their entire length. Examples of such rock bolts are the Swellex-bolt and the Split-set bolt.

In combination with rock bolting, *wire mesh* and *steel straps* are often used, particularly in closely jointed rock and under rock burst conditions. However, there has been a gradual trend for shotcrete to replace mesh and straps.

In poor rock conditions and/or sections of insufficient rock cover (for instance entrance areas), so-called *spiling* is often used. The purpose of the spiling bolts is to maintain the correct profile and to create a "bridge" for the assumed unstable rock mass. Normally, spiling is carried out as temporary support, and supplemented later with permanent support such as shotcrete and radial bolts.

The spiling bolts are installed ahead of the tunnel face in a fan shaped pattern oriented 10-25^o relatively to the tunnel axis, see Figure 9.3. Most commonly, fully grouted bolts with a diameter of 25-32 mm and a length of about 6 m (bolt length = length of the blasting round + 2 to 4 m) are used. Typical bolt spacings are between 0.3 and 0.8 m (Statens Vegvesen, 1994).

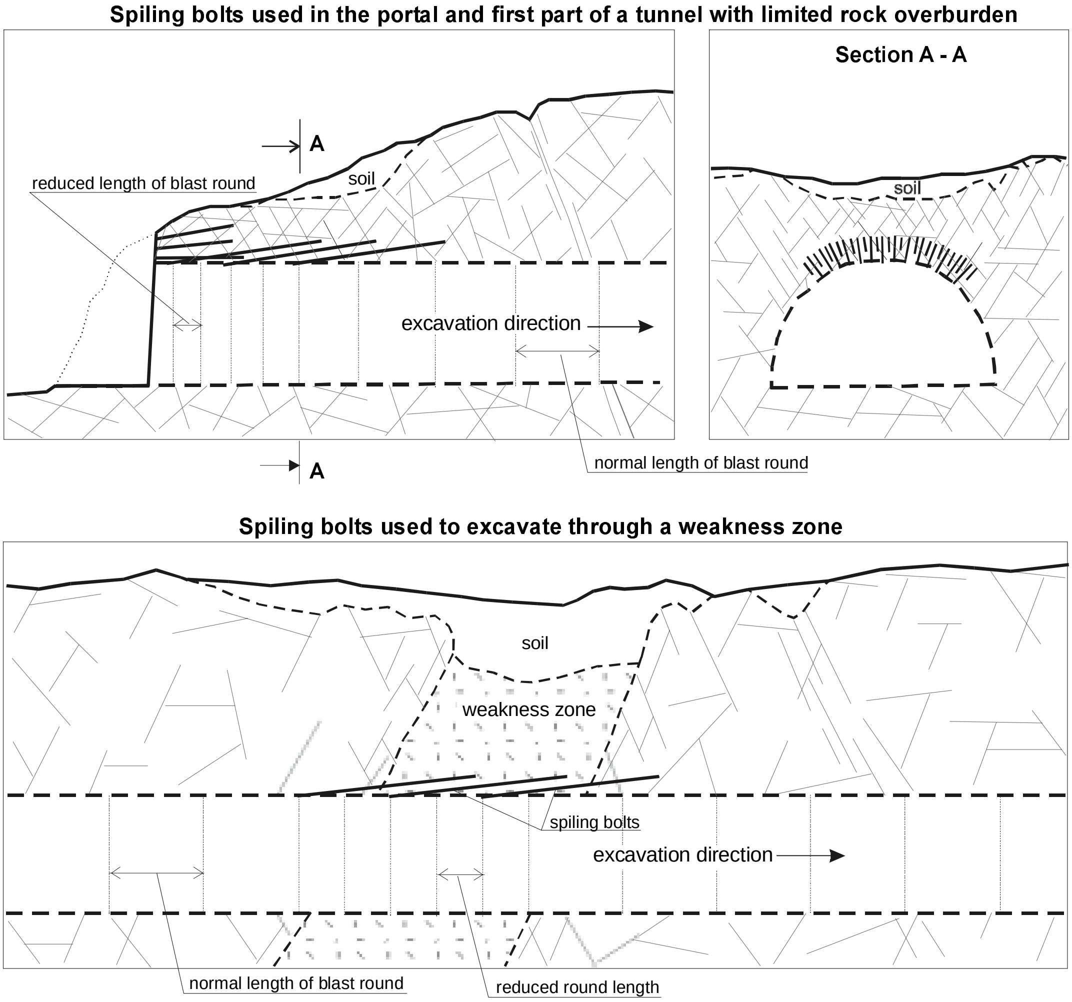


Figure 9.4 Typical applications of spiling (modified from Statens Vegvesen, 1994).

Typical lengths of rock bolts in tunnels are 2-4 m and typical diameters, 20-25 mm. In caverns, *bolt lengths* of 6 m and diameters of 25 mm and 32 mm are often used. In Norwegian tunnels the following expression has often been used to find the bolt length (in metres):

$$Lb = 1.4 + 0.184\ Dt$$

where Dt = the diameter or span of the tunnel (in metres)

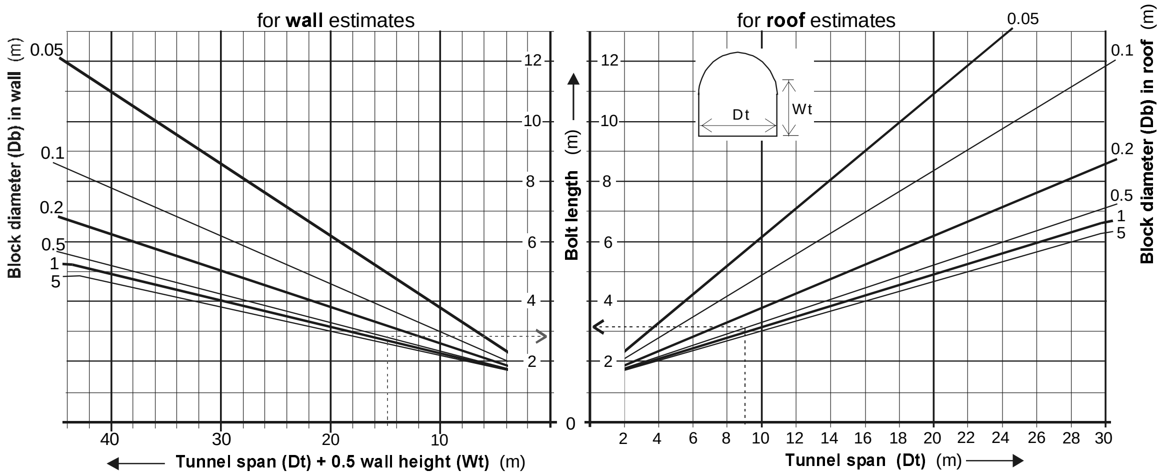
Ideally, the length of the bolts should be designed according to the rock mass conditions, especially regarding their block size. The following expressions have been suggested by Palmström (2000):

$$\mathrm{Lb_{roof}} = 1.4 + 0.16\mathrm{Dt}\left(1 + \frac{0.1}{\mathrm{Db}}\right)$$

$$Lb_{wall} = 1.4 + 0.08\left(Dt + 0.5Wt\right)\left(1 + \frac{0.1}{Db}\right)$$

where Db = the block diameter (in metres)
Wt = the tunnel wall height (in metres)

These equations are graphically solved in Figure 9.5, giving bolt lengths for roof and for walls. Note: The block size to be used is the average block size at the actual location.



Example:
For tunnel span 9 m, wall height 12 m and block diameter Db = 0.5 m:Lb = 3.1 m in roof, Lb = 2.8 m in wall

Figure 9.5 Bolt length in roof and walls (from Palmström, 2000).

Some key figures for commonly used rock bolts are shown in Table 9.2.

Table 9.2 Dimensions and capacities of some rock bolts

Type of rock bolt	Diameter (mm)	Yield strength (kN)	Failure load (kN)	Elongation of failure	Elogation for 3 m bolt (mm)	Standard bolt length (m)
Round steel bar	20	60-70	100	8%	240	0.8-6.0
Deformed bar	20	120	150	3%	90	0.8-6.0
	25	220	250	1%	30	0.8-6.0
Ørsta rock bolt	20	120	150	3%	90	1.5-6.0
CT-bolt	20	120	150	3%	90	1.5-6.0
	22	200	250	2%	60	1.5-6.0
Hollow bolt	27	100	130	8%	240	2.0-6.0

8.2.3 Shotcrete

This type of rock support is obtained by spraying concrete on the rock surfaces. *Shotcrete* for rock support has been used for several decades (in Norway since 1950), and has become increasingly popular because of its favourable properties together with high capacity and flexibility.

Today, three different shotcrete methods are in use:

1. Ordinary shotcrete sprayed in layers of up to about 100 mm thickness.
2. *Mesh reinforced shotcrete*. This is produced by first spraying a layer of concrete before installing the mesh with typically 5–6 mm diameter steel bars. Then a second and sometimes more layers are applied to cover the mesh entirely.
3. Fiber reinforced shotcrete *(Fibrecrete)*. This is a type of shotcrete where thin needles or fibres, 2-5 cm long, of steel or other materials are mixed into the (wet) concrete. In Norway, this method is the most used, and has almost completely replaced the mesh reinforced shotcrete.

In the wet-mix method, the shotcrete is mixed as a low slump concrete that is pumped to the nozzle. Compared to the dry-mix method, the wet process has the following main advantages:

- Higher capacity.
- Reduced rebound.
- Improved working conditions.
- Steel fibres easier to use than mesh as reinforcement.

As a result of the last 20 years development in shotcrete technology, robot-operated wet process shotcreting is normally used instead of the hand-operated dry process, and in addition, the use of steel fibres and micro-silica in the wet-mix have become standard practice. Table 9.3 shows typical bond strengths of the shotcrete, and Table 9.4 shows an example of a typical mix-design for *steel-fibre reinforced shotcrete* (*fibrecrete*). The strength reduction caused by the use of accelerator, temperature, etc., necessitates a 20-30% higher minimum quality of the concrete delivered at the site, as shown in Table 9.5.

Table 9.3 Bond between shotcrete and different types of rocks (from NB, 1993).

Type of rock	**Bond** (MPa)		**Type of rock**	**Bond** (MPa)	
	rough-ground rock surface	*rough rock surface*		*rough-ground rock surface*	*Rough rock surface*
Shale	0.24 ± 0.18	0.28 ± 0.11	Granite / diorite	0.34 ± 0.12	1.12 ± 0.2
Mica schist	0.58 ± 0.19	0.85 ± 0.35	Granite / diorite	1.04 ± 0.32	1.40 ± 0.25
Gneiss // foliation	0.19 ± 0.05	0.51 ± 0.11	Granite / diorite	1.48 ± 0.45	1.71 ± 0.14
Gneiss ⊥ foliation	1.53 ± 0.28	1.8			
Sandstone, porous	(1.1)	(1.1)	Gabbro, diabase	1.56 ± 0.25	1.7
Sandstone	> 1.8	> 1.8			
Marl	1.49	1.84	Shotcrete surface	1.7	1.7
Limestone	1.58 ± 0.12	1.54 ± 0.30	Wooden plate	1.7	1.7
Marble	1.38 ± 0.30	1.52 ± 0.28			
( ) indicates rupture in rock					

Table 9.4 Typical mix for C40 wet shotcrete.

Portland cement	470 kg
Micro-silica	8%
Aggregate 0-8 mm	1670 kg
Superplasticizer (BNS)	5 kg
Plasticizer (lignosulphonate)	3.5 kg
Steel fibres	50 kg
Accelerator (modified silicate)	5%

Table 9.5 Minimum quality requirement for concrete delivered at a shotcreting site (from NB, 1993).

Specified shotcrete quality	C30	C35	C40	C45	C50	C55
Minimum quality of concrete	C38	C43	C48	C54	C60	C65

The use of steel fibres has the favourable effect of increasing the strength and the energy before failure. It has been shown that by adding only 1 vol. % of steel fibres, the load capacity of a 50 MPa shotcrete slab may increase by 85%, and ductility by as much as 20 times the original value (Kompen & Opsahl, 1986). The general trend has been to make use of increasingly longer fibres. Today, 25 and 32 mm long fibres are most frequently used.

Micro-silica in the mix improves the strength properties of the shotcrete and makes it easier to distribute the steel fibres in the fresh concrete. Other important effects are that the micro-silica reduces permeability and improves frost resistance.

For immediate support, shotcrete is primarily used in heavily jointed rock masses. A thickness of 5-10 cm is normally applied in such cases, and thus has the favourable effect of sealing locking joints and reducing deformations. In many cases the shotcreting is combined with rock bolting.

Another important field for immediate support are cases of heavy rock spalling, where steel fibre reinforced shotcrete has in many cases in Norway reduced problems considerably, and in some cases even eliminated them completely. In such cases the shotcreting is always supplemented by rock bolting.

A combination of steel fibre reinforced shotcreting and systematic rock bolting may often be used where cast-in-place concrete was previously the only alternative, see Figure 9.6.

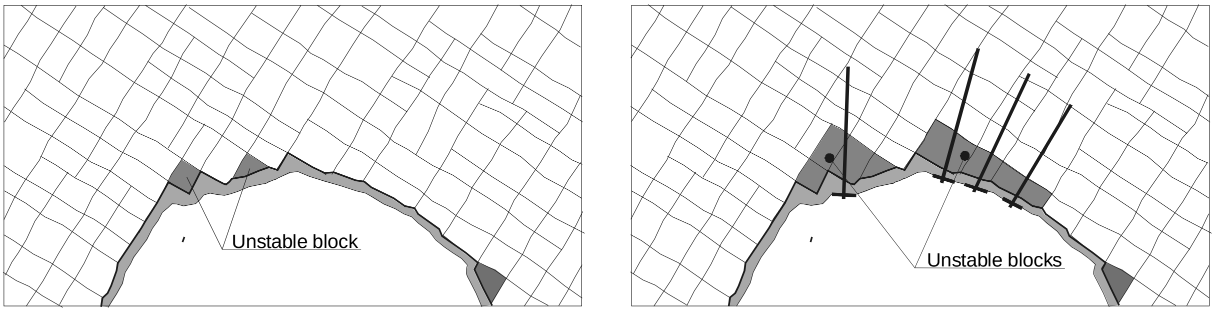


Figure 9.6 Left: Shotcrete applied for reinforcing unstable fragments and small blocks.
Right: Combination of shotcrete/fibrecrete and rock bolts used as alternative to in situ concrete lining.

In very poor rock conditions (including swelling clay), rebar reinforced sprayed ribs may represent a very favourable alternative to more expensive and time consuming solutions such as concrete lining, see Figure 9.7.

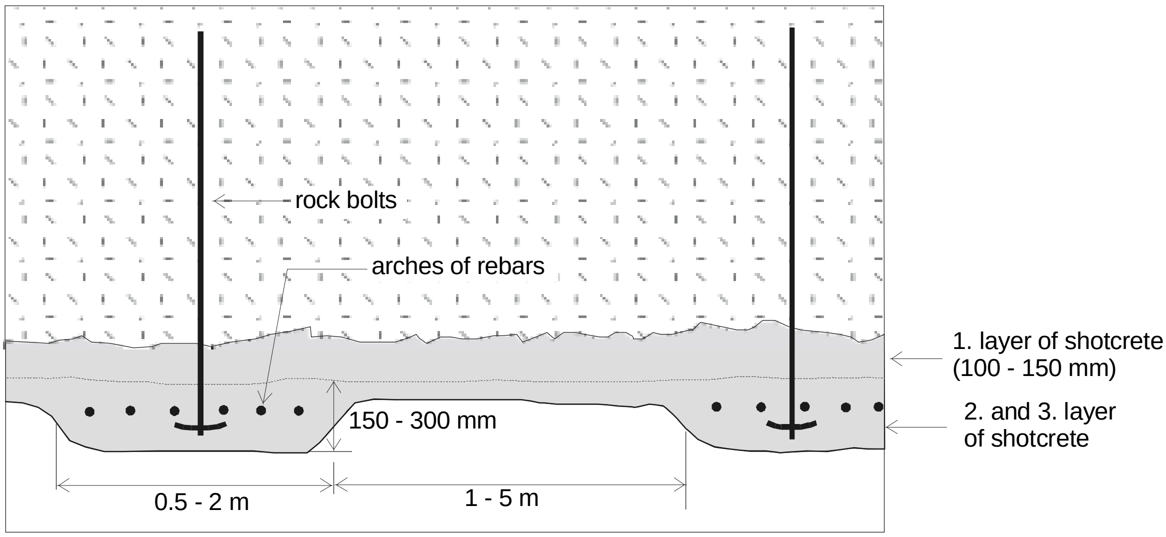


Figure 9.7. Typical design of shotcrete ribs (from NB, 1993).

The rebar steel has a typical dimension of 20 mm diameter. The width and thickness of the ribs and the spacing between them have to be adjusted to the local conditions. The thickness of the first layer of shotcrete (often steel-fibre reinforced) is most commonly 100-150 mm. This layer smooths the original rock surface. Shotcrete applied after installation of the rebar steel should be unreinforced. The ribs, as shown in Figure 9.7, are anchored to the rock mass with radial bolts.

Spraying capacities of 50-100 m^3 shotcrete per shift (7.5 hours) have often been achieved in underground excavations. Norwegian contractors have been in the forefront during the development of shotcrete and fibrecrete.

Control of shotcrete compressive strength is based on uniaxial compressive tests on drilled (cylindrical) cores. According to Norwegian recommendations (NB, 1993), the cores should have a minimum H/D ratio of 1.0, preferably 2.0, with a core diameter of at least 60 mm. Each test series should consist of 3 samples, all drilled from the same location. The minimum strength should be according to Table 9.6.

Table 9.6 The conversion of strength of cube samples to in situ drilled sample (from NB, 1993)

Example for C40: 40 MPa × 0.8 × 0.8 = 25.8 MPa

- 25.8 MPa: Required strength of tested in situ cylindrical sample
- 0.8: Conversion factor for in situ sampling
- 0.8: Conversion factor for cubic vs. cylinder specimens
- 40 MPa: Requirement for cast cube specimens

Flexural tests can be performed on beams or plates.

The main advantages of shotcrete and fibrecrete are:
- It can be ready for use in the excavation at short notice
- No formwork is needed
- It is independent of the shape of the excavation
- It has high placing capacity
- It is easily combined with other supporting methods
- Flexible deformation properties

The disadvantages are:
- Collapses of shotcrete have occurred where it has been applied on swelling rocks.
- Shotcrete may have limited effect in clay containing rocks or joints with lack of adhesion.

8.2.4 Cast-in-place concrete lining

Since this alternative can take considerable loads due to its arching effect, the cast-in-place *concrete lining* is preferably applied where long sections of poor rock mass conditions occur. A pre-constructed steel formwork is used for this work. Where the concrete lining is placed close to the tunnel face, the formwork has to be constructed to withstand blast damage.

Large faults or weakness zones with highly unstable rock masses are often supported using cast-in-place concrete lining. This is performed using a full lining shield designed to fit the actual cross section of the tunnel, see Figure 9.8. When properly planned, it may take as little as 2-4 hours from decision has been taken to use this method till the concrete casting can start.

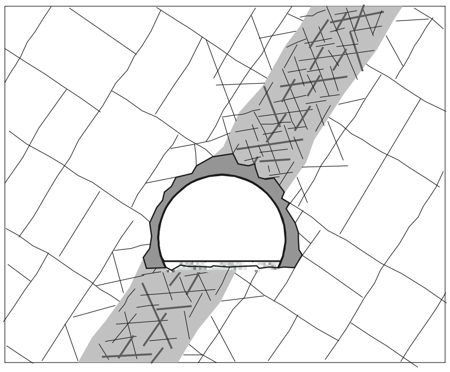


Figure 9.8 Cast-in-place concrete lining.

Compared to most other methods, concrete lining is, however, a costly and time-consuming rock support method. In many cases steel arches with or without shotcrete and later concrete lining may be an attractive alternative. Concreting of the tunnel invert is often necessary in such poor quality rock masses.

Very few incidents of collapse of concrete lining have been reported from Norwegian tunnels where a minimum thickness of 0.3 m is used. The efficiency in installing concrete lining has increased considerably during the last 15 years as result of improved, prefabricated steel formwork. Capacities of 1-2 m lining per shift during excavation and 50% more after break-through of the tunnel are normal.

In adverse rock mass conditions showing short stand-up times, shotcrete is often applied immediately after blasting, and then concrete lining before the next blast.

8.2.5 Other methods

As described in Section 6.4 the purpose of *grouting* is usually to reduce the groundwater inflow. Grouting is also occassionally carried out to improve the rock mass stability. In such cases it is performed as pre-grouting (ahead of the advancing tunnel face).

Occasionally in tunnelling, special rock support methods are used, not as a part of the permanent support, but rather as a tool for making further tunnelling possible in difficult rock conditions. One such method is *ground freezing*. This is a ground-strengthening technique which is used, for instance, if the tunnel must cross weakness zones or soil-filled depressions of exceptionally low stability. The freezing is achieved by installing special freezing pipes in 100-200 mm holes drilled in a fan shape around the tunnel. After the ground is frozen, excavation through the zone is performed using short blast rounds and successive support, for example by shotcrete and concrete lining.

8.2.6 Special rock support solutions for TBM tunnels

In TBM and other mechanically excavated tunnels with regular profiles, rock bolting is often combined with various types of mesh and ring beams. This type of support can rapidly be installed immediately behind the cutter head.

In minor weakness zones, a quick and simple alternative for immediate support is to use corrugated steel plates or liner plates. Such plates can be bolted together to form a continuous ring along the tunnel periphery and are quickly installed behind the cutter head.

Because of the more gentle rock excavation and the stable, circular profile, a TBM-tunnel normally needs considerably less rock support than the drill and blast alternative. In good rock conditions, according to Norwegian experience, the support cost of the TBM-alternative is in many cases as low as only one to two thirds of the support cost for a drill and blast tunnel of the same cross sectional area. It is, however, important to be aware that due to lesser flexibility of the

TBM-alternative, stability problems may also cause long delays and additional costs if the TBM-operation is not thoroughly planned.

In rock burst areas TBM tunnels are generally more susceptible to spalling than drill and blast tunnels. This is due to fissures produced in blasting that allow an easier redistribution of the stresses in the surrounding ground.

8.2.7 Costs and capacities

The costs of rock support will vary considerably from project to project depending on such factors as excavation type and size. It is therefore difficult to present general figures on costs and capacities. An indication of some typical, relative costs may, however, still be given as shown in Figure 9.9.

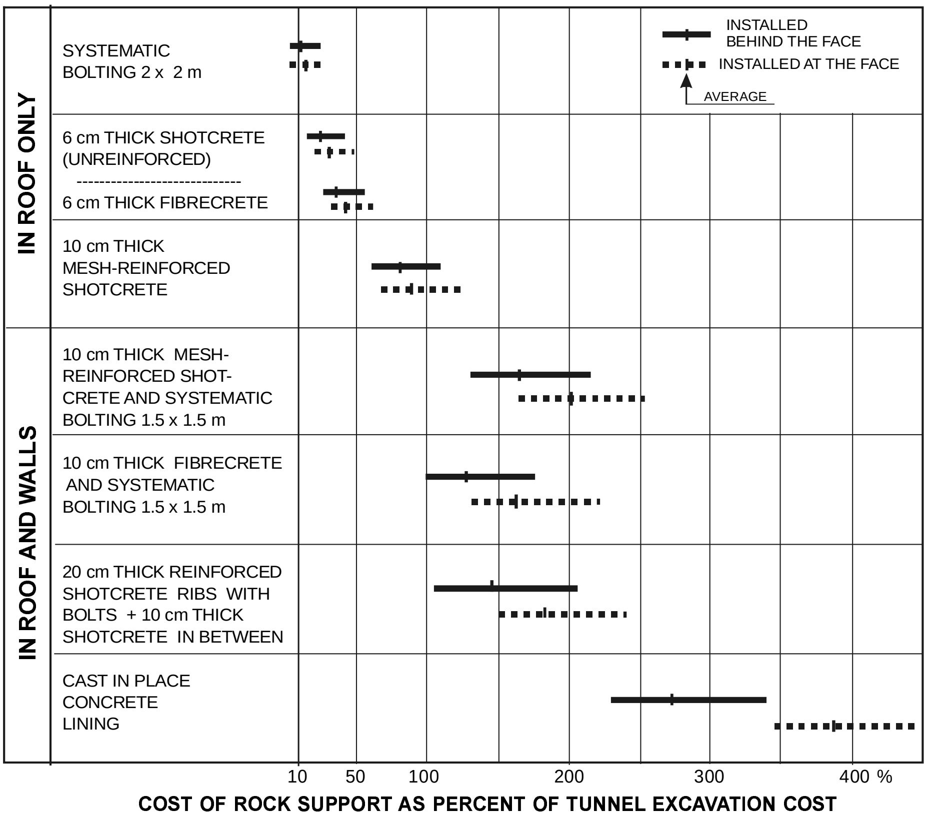


Figure 9.9 An indication of relative costs of rock support and lining for drill & blast tunnels.

Of the rock support alternatives included in the figure, in situ concrete lining, is obviously the most expensive. One metre of concrete lining normally represents 2-4 times the cost of drill and blast excavation including mucking and hauling. Shotcreting is also relatively expensive. Normally, a 10 cm thick layer of fibrecrete in the roof and walls of the tunnel costs, for instance, around 50% of the costs of excavation if applied behind the face, and about 80% if applied at the face.

Rock support at the working face, as shown in Figure 9.9, is generally much more expensive than support behind the face. This is because support work close to the tunnel face interferes with tunnelling operations and generally slows down the excavation advance. A good alternative therefore, is to perform only preliminary support on the working face and to install the permanent support later when it can be done without interfering with the tunnelling operations. For instance, immediate support consisting of a 50 mm shotcrete layer and concrete lining behind the working face is normally less expensive than concrete lining on the working face. It should be noted here that during the last few years there has been an apparent trend towards a reduction of the differences in cost for support at and behind the face.

Grouting is not included in Figure 9.9 because the costs may vary considerably. Complex support alternatives such as ground freezing are always very expensive. In situations with very difficult ground conditions such methods may, however, represent the only alternative for further excavation.

The delays caused by installing rock support are considerable. Table 9.7 gives *capacities* of some rock supporting works in Norwegian tunnelling contracts (1998). These capacities are lower than the real capacities.

Table 9.7 Normal capacities for various rock excavation and support works used in contracts by the Norwegian Road Authorities (1998)

Type of support	**Capacity**	
	At tunnel face	*Behind tunnel face*
Rock bolts	12 bolts/hour	15 bolts/hour
Steel straps	25 m/hour	40 m/hour
Wire mesh	10 m^2 /hour	15 m^2 /hour
Wet shotcrete	6 m^3 /hour	8 m^3 /hour
Concrete lining, 1/1 round length	0.2 m/hour	0.2 m/hour
Extra concrete (> 0.4 m average thickness per section casted)	10 m^3 /hour	10 m^3 /hour

8.3 Methods to estimate rock support requirements

The rock engineer is generally faced with the need to arrive at a number of design decisions in which judgement and practical experience have to play important roles. Estimation of support requirements also is based largely on observation, experience and the personal judgement of those involved. Often, the estimate is, however, backed by one or more of the following three methods:

- *Analytical methods,* based on analyses of stresses and deformations, and including techniques such as closed form solutions, numerical methods (finite elements, finite difference, boundary elements), analogue simulations (electrical and photoelastic) and physical modelling.
- *Observational methods,* relying on actual monitoring and ground movement during excavation to detect measurable instability, and on the analysis of ground-support interaction. Includes methods such as the *New Austrian Tunnelling Method (NATM)* and the Convergence-Confinement method. Although considered as separate methods, the observational approachs are the only way the results and predictions of the other methods may be checked.
- *Empirical methods,* assessing the stability of mines and tunnels based on statistical analyses. Engineering rock mass classifications are the best known empirical approach, and in many tunnelling projects are applied as the only practical basis for rock design.

All methods require geological input and consideration of safety requirements.

As the underground conditions are never known before being revealed during excavation, the final decisions concerning the amount and methods for rock support should not be made before observations can be made in the tunnel.

8.3.1 The RMR (Geomechanics) system

This engineering classification system, developed by Bieniawski in 1973, utilises the following six rock mass parameters:

1. Uniaxial compressive strength of intact rock material.
2. Rock quality designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

All of these are measurable in the field and can also be obtained from borehole data.

To apply the *RMR classification*, the rock mass along a tunnel route is divided into a number of structural regions, i.e., zones in which certain geological features are more or less uniform, see Section 4.5. The above six classification parameters are determined for each structural region from measurements in the field. Once the classification parameters are determined, the ratings are assigned to each parameter according to Table 9.8. In this respect, the typical, rather than the worst, conditions are evaluated. Furthermore, it should be noted that the ratings, which are 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.

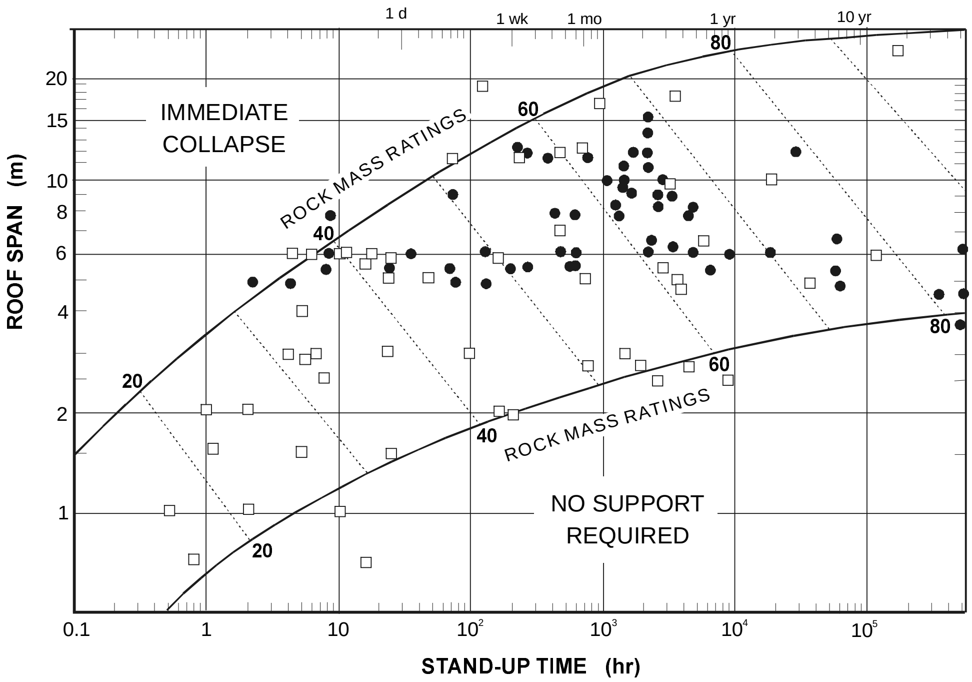


Figure 9.10 RMR classification of rock masses. (Contour lines indicate limits of applicability.) (Bieniawski, 1989)

Stand-up time as function of unsupported span and RMR-values are shown in Figure 9.10, and an example of recommended rock support according to the RMR system is shown in Table 9.9 (reflecting so-called ”South-African”, “European” and “American” practice, respectively).

Table 9.8 RMR classification of rock masses (Bieniawski, 1989).

A. Classification parameters and their ratings

PARAMETER			Range of values // **RATINGS**						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range uniaxial compr. strength is preferred		
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
		RATING	**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	Drill core quality RQD		90 - 100%	75 – 90%	50 - 75%	25 - 50%	< 25%		
	RATING		**20**	**17**	**13**	**8**	**5**		
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	Length, persistence	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m		
		Rating	**6**	**4**	**2**	**1**	**0**		
		Separation	none	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm		
		Rating	**6**	**5**	**4**	**1**	**0**		
		Roughness	very rough	rough	slightly rough	smooth	slickensided		
		Rating	**6**	**5**	**3**	**1**	**0**		
		Infilling (gouge)	none	Hard filling		Soft filling			
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm		
		Rating	**6**	**4**	**2**	**2**	**0**		
		Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed		
		Rating	**6**	**5**	**3**	**1**	**0**		
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min		
		$p_w / \sigma 1$	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	completely dry	damp	wet	dripping	flowing		
		RATING	**15**	**10**	**7**	**4**	**0**		

p_w = joint water pressure; $\sigma 1$ = major principal stress

B. Rating adjustment for discontinuity orientations

		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	**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	GOOD	FAIR	POOR	VERY POOR

D. Meaning of rock mass classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 – 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45^o	35 – 45^o	25 - 35^o	15 - 25^o	< 15^o

Table 9.9 RMR classification guide for excavation and support in rock tunnels (Bieniawski, 1989).

Shape: horseshoe; Width: 10 m; Vertical stress: below 25 MPa; Excavation by drill & blast				
Rock	**Excavation**	**Support**		
Mass class		*Rock bolts* (20 mm diam., fully bonded)	*Shotcrete*	*Steel sets*
1. Very good rock RMR: 81-100	*Full face:* 3 m advance	Generally no support required except for occasional spot bolting		
2. Good rock 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
3. Fair rock RMR: 41-60	*Top heading and bench:* 1.5-3 m advance in top heading; Commence support after each blast; Commence 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
4. Poor rock 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 walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light ribs spaced 1.5 m where required
5. Very poor rock RMR < 21	*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 forepoling if required. Close invert

8.3.2 The Q-system

The *Q-system* for rock mass classification, developed at the Norwegian Geotechnical Institute (NGI) in 1974, originally included a little more than 200 tunnel case histories, mainly from Scandinavia (Barton et al., 1974). In 1993 the system was updated to include more than 1000 cases (Grimstad and Barton, 1993). It is a quantitative classification system for estimates of tunnel support, based on a numerical assessment of the rock mass quality using the following six parameters:

- Rock quality designation (RQD).
- Number of joint sets (J_n).
- Roughness of the most unfavourable joint or discontinuity (J_r).
- Degree of alteration or filling along the weakest joint (J_a).
- Water inflow (J_w).
- Stress condition given as the stress reduction factor (SRF); composed of
  - Loosening load in the case of shear zones and clay bearing rock,
  - Rock stress in competent rock, and
  - Squeezing and swelling loads in plastic, incompetent rock.

The above six parameters are grouped into three quotients to give the overall rock mass quality:

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

- The first two parameters represent the overall structure of the rock mass, and their quotient is a relative measure of the block size.
- The second quotient is described as an indicator of the inter-block shear strength.
- The third quotient is described as the “active stresses”.

The ratings of the various input parameters to the Q-value are given in Table 9.10.

Table 9.10 Description and ratings for the input parameters of the Q-system (simplified from Grimstad and Barton, 1993).

***RQD** (Rock Quality Designation)*

Very poor	RQD = 0 - 25%
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

Notes:
(i) Where RQD is reported or measured as < 10 (including 0), a nominal value of 10 is used to evaluate Q
(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate

***Jn** (joint set number)*

Massive, no or few joints	Jn = 0.5 - 1
One joint set	2
One joint set plus random joints	3
Two joint sets	4
Two joint sets plus random joints	6
Three joint sets	9
Three joint sets plus random joints	12
Four or more joint sets, heavily jointed, "sugar-cube", etc.	15
Crushed rock, earthlike	20

***Notes:** (i) For tunnel intersections, use (3.0 x Jn); (ii) For portals, use (2.0 x Jn)*

***Jr** (joint roughness number)*

a) Rock-wall contact, b) rock-wall contact before 10 cm shear	
Discontinuous joints	Jr = 4
Rough or irregular, undulating	3
Smooth, undulating	2
Slickensided, undulating	1.5
Rough or irregular, planar	1.5
Smooth, planar	1.0
Slickensided, planar	0.5

***Note**: i) Descriptions 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	Jr = 1.0
Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Notes:
i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
ii) Jr = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength

***Ja** (joint alteration number)*

	JOINT WALL CHARACTER		Condition	***Wall contact***
Contact between joint walls	CLEAN JOINTS	Healed or welded joints:	filling of quartz, epidote, etc.	Ja = 0.75
		Fresh joint walls:	no coating or filling, except from staining (rust)	1
		Slightly altered joint walls:	non-softening mineral coatings, clay-free particles, etc.	2
	COATING OR THIN FILLING	Friction materials:	sand, silt, calcite, etc. (non-softening)	3
		Cohesive materials:	clay, chlorite, talc, etc. (softening)	4

	FILLING OF:	Type	***Some wall contact*** *Thin filling (< 5 mm)*	***No wall contact*** *Thick filling*
Some or no wall contact	Friction materials	sand, silt calcite, etc. (non-softening)	Ja = 4	Ja = 8
	Hard cohesive materials	compacted filling of clay, chlorite, talc, etc.	6	5 - 10
	Soft cohesive materials	medium to low overconsolidated clay, chlorite, talc,	8	12
	Swelling clay materials	filling material exhibits swelling properties	8 - 12	13 - 20

***Jw** (joint water reduction factor)*

Dry excavations or minor inflow, i.e. < 5 l/min locally	p_w < 1 kg/cm^2	Jw = 1
Medium inflow or pressure, occasional outwash of joint fillings	1 - 2.5	0.66
Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.3
Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 - 0.1
Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1 - 0.05

***Note:** (i) The last four factors are crude estimates. Increase Jw if drainage measures are installed*
(ii) Special problems caused by ice formation are not considered

***SRF** (Stress Reduction Factor)*

Weakness zones intersecting excavation	Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth)	SRF = 10
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m)	5
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5
	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m)	5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m)	2.5
	Loose, open joints, heavily jointed or "sugar-cube", etc. (any depth)	5

***Note:** (i) Reduce these SRF values by 25 - 50% if the relevant shear zones only influence, but do not intersect the excavation.*

		σ_c / σ_1	σ_θ / σ_c	SRF
Competent rock, rock stress problems	Low stress, near surface, open joints	> 200	< 0.01	2.5
	Medium stress, favourable stress condition	200 - 10	0.01 - 0.3	1
	High stress, very tight structure. Usually favourable to stability, may be except for walls	10 - 5	0.3 - 0.4	0.5 - 2
	Moderate slabbing after > 1 hour in massive rock	5 - 3	0.5 - 0.65	5 - 50
	Slabbing and rock burst after a few minutes in massive rock	3 - 2	0.65 - 1	50 - 200
	Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock	< 2	> 1	200 - 400

***Notes:** (ii) For strongly anisotropic stress field (if measured): when 5 < σ_1/σ_3 <10, reduce σ_c to 0.75 σ_c. When $\sigma 1/\sigma 3$ > 10, reduce σ_c to 0.5σ_c*
(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 low stress cases

			σ_θ / σ_c	SRF
Squeezing rock	*Plastic flow of incompetent rock under the influence of high pressure*	Mild squeezing rock pressure	1 - 5	5 - 10
		Heavy squeezing rock pressure	> 5	10 - 20
Swelling rock	*Chemical swelling activity depending on presence of water*	Mild swelling rock pressure		5 - 10
		Heavy swelling rock pressure		10 - 15

The Q-value is related to tunnel support requirement by defining the equivalent dimensions of the underground opening. This equivalent dimension, which is a function of the size and type of the excavation, is obtained by dividing the span, diameter or wall height of the excavation (Dt) by a quantity called the *excavation support ratio (ESR)*, given as:

$$De = \frac{Dt}{ESR}$$

Ratings of ESR are shown in Table 9.11

Table 9.11 Ratings of the excavation support ratio (ESR) (from Barton et. al., 1974).

Type or use of underground opening	**ESR**
Temporary mine openings	3.5
Vertical shafts, rectangular and circular respectively	2.0 - 2.5
Water tunnels, permanent mine openings, adits, drifts	1.6
Storage caverns, road tunnels with little traffic, access tunnels, etc.	1.3
Power stations, road and railway tunnels with heavy traffic, civil defence shelters, etc.	1.0
Nuclear power plants, railroad stations, sport arenas, etc.	0.8

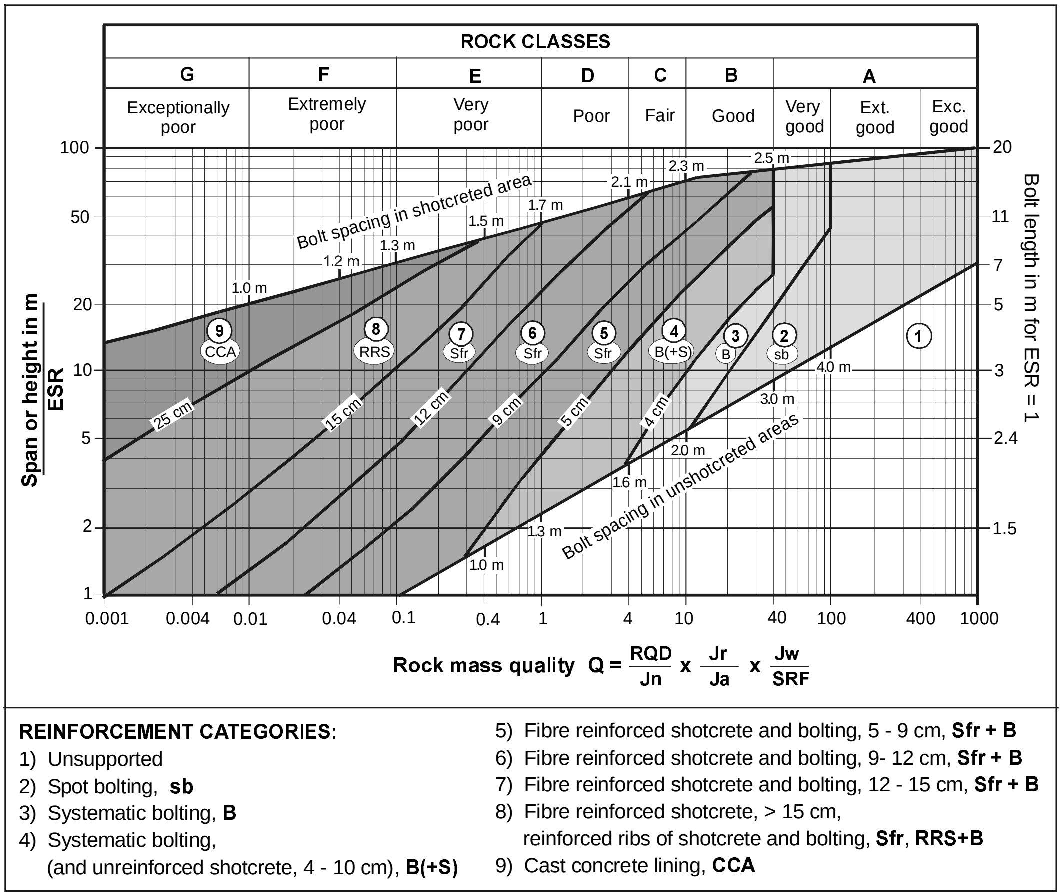


Figure 9.11 The Q system developed by the Norwegian Geotechnical Institute (NGI), is in worldwide use for estimates of rock support (from Grimstad and Barton, 1993).

The Q-value in Figure 9.11 is related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases. Wall support can also be found using the same figure by applying the wall height and the following adjustments to Q:

For $Q > 10$	use $Q_{wall} = 5Q$
For $0.1 < Q < 10$	use $Q_{wall} = 2.5Q$
For $Q < 0.1$	use $Q_{wall} = Q$

8.3.3 The RMi support method

The *RMi method* for rock support involves input as shown in Figure 9.12 (the RMi and its input parameters are described in Section 4.4.1).

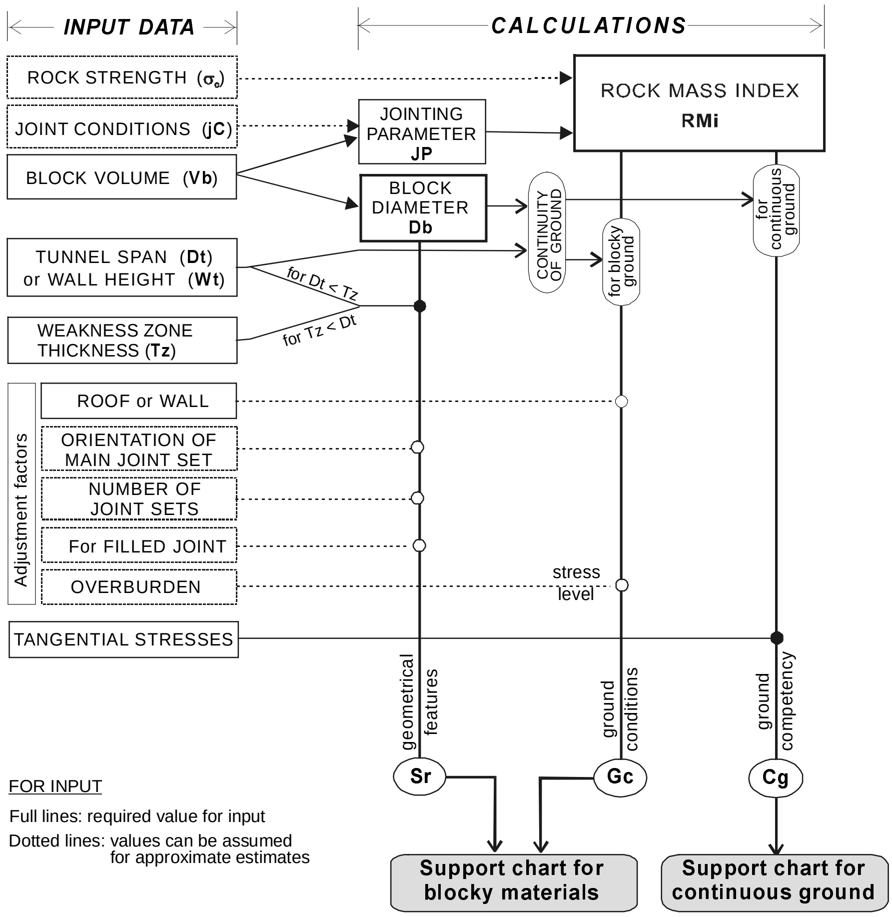


Figure 9.12 The input data and their use in the RMi rock support estimate method (from Palmström, 2000).

8.3.3.1 The rock support parameters

The amount of blocks in the periphery of the underground opening will largely determine whether the ground will behave as a *continuous*, bulk mass, or as a *discontinuous*, jointed or blocky mass dominated by the individual blocks and the character of the joints. The behaviour can be assessed by the continuity factor CF = Dt/Db = tunnel diameter/block diameter (see Section 9.1).

The RMi support method applies different calculations and support charts for these two types of ground since their behaviour is markedly different (see Figures 9.1 and 9.12).

8.3.3.2 Continuous ground

Continuous ground occurs when CF < approx. 5 (*massive rock*), in which the properties of intact rock dominate, and when CF > approx. 100 (*particulate or highly jointed rock*), where the ground behaves as a bulk material. In this type of ground the main influence on the rock mass behaviour in an underground opening is from the stresses. Therefore, a competency factor Cg = strength of the rock mass/stresses acting, which is expressed:

- In massive ground, as $Cg = \dfrac{RMi}{\sigma_\theta} = f_\sigma \times \dfrac{\sigma_c}{\sigma_\theta} \approx 0.5\dfrac{\sigma_c}{\sigma_\theta}$
- In particulate ground, as $Cg = \dfrac{RMi}{\sigma_\theta} = \sigma_c \times \dfrac{JP}{\sigma_\theta}$

Here σ_θ = the tangential stress in the rock masses around the opening. A method to estimate σ_θ in the roof and walls of a tunnel in massive rock is shown in Section 5.2.4.

Competent ground occurs where Cg > 1; else the ground is overstressed (incompetent). Cg is applied in the ground the support chart (Figure 9.14).

Massive, competent ground is generally stable, see Figure 9.1, and while some scaling work may be needed for drill & blast tunnels, support is generally not needed. Massive, *incompetent* (overstressed) ground, however, requires support because the following types of deformation and/or failures may take place:

- *squeezing* in overstressed ductile rocks (such as schists);
- *spalling* or *rock burst* in overstressed brittle, hard rocks (such as granite and gneiss).

Particulate materials (highly jointed rocks) generally require immediate support. Their initial behaviour is often similar to that of blocky ground and the Figure 9.13 support chart can be used. Time dependent squeezing will take place, in addition to the initial instability, in overstressed *(incompetent)* ground. The Figure 9.14 support chart will have to be updated when more experience with this type of ground is available.

Example

Estimate of rock support in continuous ground

A tunnel in massive granite with a compressive strength (σ_c) of 130 MPa has z = 1000 m overburden.

Evaluations: Massive rock means Vb = 8 m³ or larger. The tangential stress is estimated using the method described in Section 5.2.4:

Theoretical, vertical stress $p_v = 0.027 \times z = 27$ MPa; tangential stress in the roof of a horse-shoe shaped tunnel is $\sigma_\theta = p_v (A \times k - 1) = 102.6$ MPa (assuming k = 1.5).

In massive rock RMi ≈ 0.5 σ_c = 65, hence the competency factor is: $Cg = \dfrac{RMi}{\sigma_\theta} = \dfrac{65}{102.6} = 0.63$.

According to the Figure 9.14 support chart, brittle granite under these conditions will give mild rock burst, which will require support consisting of rock bolts spaced 1.5-3 m.

8.3.3.3 Blocky ground

Stability in blocky (jointed) ground is mainly influenced by the block size and shape, the shear strength of the joints delineating the blocks and the orientation of the same joints relative to the opening. The following two support parameters, which include all these features, are used in the Figure 9.13 support chart:

- The ground quality, given as the ground condition factor
  $Gc = RMi\ (SL \times C) = \sigma_c \times JP(SL \times C)$
- The scale factor, expressed as the size ratio
  $Sr = CF \times (Co / Nj) = (Dt / Db)(Co / Nj)$

The adjustment factors SL, C and Co, Nj have unit values for the most common or typical conditions, see Table 9.12. They are applied for more accurate evaluations where the values of the factors involved are known.

In cases where a *seam* or *filled joint* (with thickness Ts < 1 m) occurs at the location, the following adjustment to the size ratio may be made:

$$Sr_s = Sr\ (1 + Ts)\ Co_s$$

The symbols used in the above equations are defined below:

Dt	=	The diameter or span of the tunnel or cavern, in metres. (For walls, the height Wt is used instead of Dt)
Db	=	The equivalent block diameter in metre, $Db = \sqrt[3]{Vb}$
C	=	A gravity adjustment factor, whether the support is in the roof or in the walls. Its ratings depend on the inclination of the walls and roof, and can be found from Table 9.12 or from the expression $C = 5 - 4 \cos \delta$ where δ = the inclination of the roof or wall.
SL	=	A stress level adjustment, see Table 9.12
Co, Co_s	=	An adjustment factor for the main joint set, or seam, respectively, see Table 9.12
Nj	=	An adjustment factor for the number of joint sets; and hence the freedom for the blocks to fall. Its ratings can be found from Table 9.12, or from $Nj = 3/n_j$ where n_j = the number of joint sets. ($n_j = 1$ for one set; $n_j = 1.5$ for one set plus random joints; $n_j = 2$ for two sets, nj = 2.5 for two sets plus random joints; etc.)

Table 9.12 Ratings of the adjustment factors (from Palmström, 2000).

STRESS LEVEL		
Very low (in portals, etc.)	(overburden < 10 m)	SL = 0.1
Low	(overburden 10 - 35 m)	0.5
Moderate	(overburden 35 - 350 m)	1
High	(overburden > 350 m)	1.5 *

* For stability in high *walls* a high stress level may be unfavourable. Possible rating SL = 0.5 - 0.75

NUMBER OF JOINT SETS *		NUMBER OF JOINT SETS *	
One set	Nj = 3	Three sets	Nj = 1
One set + random	2	Three sets + random	0.85
Two sets	1.5	Four sets	0.75
Two sets + random	1.2	Four sets + random	0.65

* Means the number of joint sets at the actual location

ORIENTATION OF JOINTS (related to the axis of the tunnel)				
IN WALL		IN ROOF	TERM	
for strike > 30^o	for strike < 30^o	for all strikes		
dip < 30^o	dip < 20^o	dip > 60^o	favourable	Co = 1
dip = 30 - 60^o	dip = 20 – 45^o	dip = 45 - 60^o	fair	1.5
dip > 60^o	dip = 45 - 60^o	dip = 20 - 45^o	unfavourable	2
-	dip > 60^o	dip < 20^o	very unfavourable	3

INCLINATION OF ACTUAL TUNNEL SURFACE (ROOF or WALL)	
Horizontal (roof)	C = 1
30^o inclination (roof in shaft)	1.5
45^o inclination (roof in shaft)	2.2
60^o inclination (roof in shaft)	3
Vertical (walls)	5

Example

Estimate of rock support in a 10 m wide tunnel in blocky ground:

Input data: Vb = 0.04 m^3; jC = 0.2; SL = 0.5; Nj =1.5; Co = 2; Dt = 10 m; C_{roof} = 1; and σ_c = 50 MPa.

Using the lower diagram in Figure 9.13: JP = 0.017 and Db = 0.34

Calculations: From the values found $Gc = \sigma_c\ JP \times SL \times C_{roof} = 0.43$; $Sr = (Dt / Db)\ (Co / Nj) = 39$

The following rock support is estimated using the upper diagram in Figure 9.13: Rock bolts spaced 1.2 m plus 150 mm thick shotcrete (fibre reinforced).

8.3.3.4 Weakness zones

Weakness zones should in many cases be treated individually without the use of support or classification systems. Support assessments for crushed zones with blocky materials (where CF = approx. 1 to 600) may, however, be carried out using the support chart for blocky ground and input parameters as for blocky ground. In small and medium sized zones (thickness between 1 and approximately 20 m) stability is influenced by the interplay between the zone and the adjacent rock masses. Therefore, the stresses in such zones are generally lower than in the adjacent ground, which will reduce the effect of squeezing. The size ratio for weakness zones is

- for Tz < Db: Sr = (Tz/Db)A2 = (Tz / Db)(Co / Nj)
- for Tz > Db: Sr = (Dt/Db)A2 = (Dt / Db)(Co / Nj)

For zones with CF > 600 special rock support evaluations should generally be made. Large zones (thickness Tz > approx. 20 m) will often behave similar to continuous ground described earlier as there will be little or no arching effect.

For crushes zones some typical RMi values for the most common conditions have been given in Table 9.13. They may be used for estimates at an early stage of a project or for cases where the composition of the zone is not known. The approximate RMi_z values are based on assumed representative block volumes for the various types of zones.

Table 9.13 Typical values of the rock mass index (RMi_z) used for various types of crushed zones

Crushed zones	Average uniaxial compressive strength	Average joint condition factor	Approximate block size		Approximate typical value
			volume	diameter	
	σ_c (MPa)	**jC**	**Vb** (m^3)	**Db** (m)	**RMi_z**
	of rock blocks				
Coarse fragmented zones	100	0.5	0.01	0.2	2
Small fragmented zones	100	0.5	0.0001	0.05	0.3
Clay-rich (simple) zones	80	0.1	0.01	0.2	0.3
Clay-rich (complex) zones	40	0.1	0.001	0.1	0.03
Clay zones*	0.1 (of clay)	-	1 cm^3 (nom.)	0.01	0.05

* For zones with mainly clay, approx. support estimates may be carried out using a nominal minimum block volume of Vb = 1 cm^3

Example

Estimate of rock support for a 6 m wide tunnel in a Tz = 3 m thick weakness zone (where the composition of the zone has not been observed).

The weakness zone is assumed to be of a “small-fragmented crushed zone” type. From Table 9.13 the following values are found: RMi_z = 0.3 and Db = 0.05 m. These values as used to find the ground condition factor Gc_{roof} = $RMi_z \times SL$ = 0.3; and, as Tz < Dt, the size ratio is Sr_{roof} = (Tz / Db)(Co / Nj) = 60. (The adjustment factors are assumed to be SL = Co = Nj = 1.)

The roof support estimated using the Figure 9.13 support chart is: Rock bolts spaced 1.1 m and 200 mm thick shotcrete (fibre reinforced).

RMi values and the support parameters may be calculated much more rapidly and easily by using a spreadsheet where the RMi and support equations are combined.

8.3.3.5

8.3.3.6 Support charts

The support charts in Figures 9.13 and 9.14 are based on experience from several drill & blast tunnels and caverns in Scandinavia. As shown in Section 4.4.1, the RMi can be estimated from the input of block volume and rock strength only by assuming normal or common conditions for the joint characteristics (jC = 1.75). This is included in Figure 9.13 for blocky materials.

In the case of swelling and slaking rock, stability may be strongly influenced by local conditions. The rock support should therefore be evaluated separately for each of these cases. Other situations that should be assessed separately have to do with the local safety requirements, i.e., the required lifetime of the opening, and other influence from earthquake vibrations, vibrations from nearby blasting or any further possible impact from the activity of man.

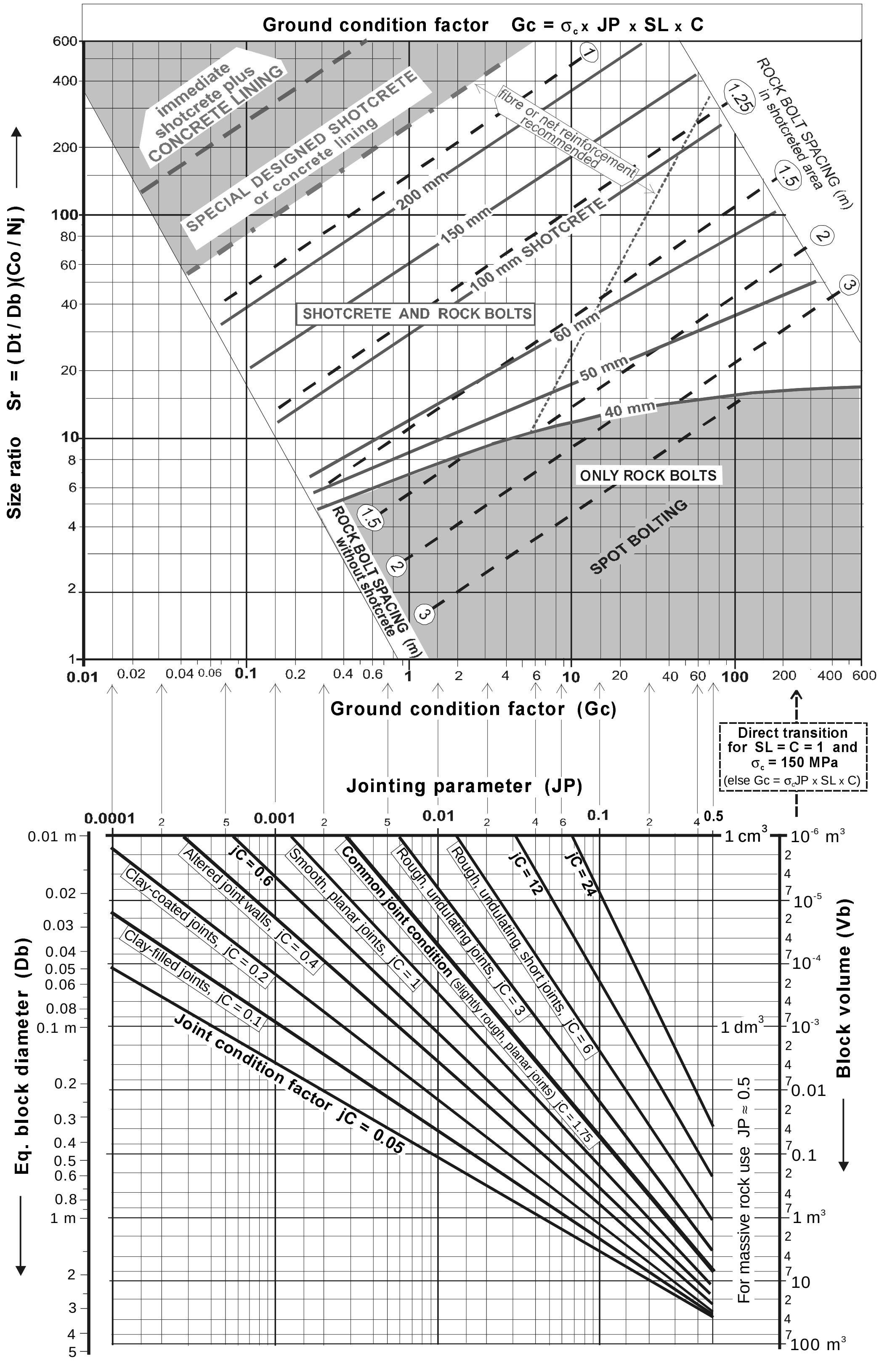


Figure 9.13 Chart to find the jointing parameter (JP) and to estimate rock support for jointed ground (from Palmström, 2000).

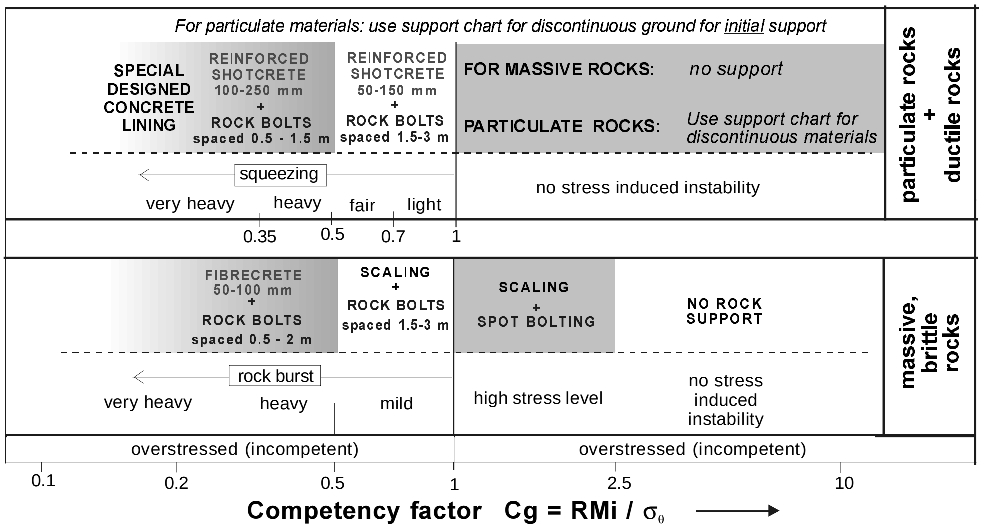


Figure 9.14 Chart for estimating support in continuous ground. Note that the support indicated for squeezing in particulate materials is approximate, as only a few cases have been used (from Palmström, 2000).

In massive ground without stress problems there is generally no need for support except for some scaling, see Figure 9.1. Massive, *incompetent* (overstressed) ground, however, requires support because of instability from *squeezing* in ductile rocks (such as schists) or ***spalling*** or ***rock burst*** in brittle, hard rocks (such as granite and gneiss).

In particulate (highly jointed) ground the initial instability and rock support will often be similar to that estimated using the principles in blocky ground, i.e., the Figure 9.13 support chart can be used. ***In incompetent (overstressed) ground*** squeezing may take place; therefore the Figure 9.14 support chart should be applied for estimates of permanent support. For incompetent, particulate ground the support indicated is based on data from only a few cases; therefore the support estimate should be controlled by other methods.

8.3.4 The New Austrian Tunnelling Method (NATM)

It is important to note that the *NATM* was developed for tunnelling in weak or squeezing ground. Such ground requires the use of structural supports, either to re-establish equilibrium or to limit displacements around the tunnel. The rock or soil material itself may be soft or hard.

The NATM has sometimes been assumed to be synonymous with the use of shotcrete during tunnel construction, probably because shotcrete is often applied in connection with the NATM. This is wrong; in practice the NATM involves the whole sequence of weak rock tunnelling aspects from investigation during design, engineering and contracting to construction and monitoring. Consequently, the NATM includes most factors involved in the execution of a tunnelling project.

A basic principle in the NATM is to take advantage of the load-bearing capacity of weak rocks. This is achieved by utilising the rock mass properties of dilating or bulking when yielding. During this process the high ground stresses close to the tunnel dissipate and the surrounding rock mass is transformed from a loading body into a load-carrying element. Only a reduced support is therefore needed to confine the unstable ground close to the tunnel.

This principle is practically achieved by allowing the rock masses around the underground opening to deform in a controlled way. The rock support therefore mainly has a confining function to stabilise the rock masses that deform. As a consequence, the support must have suitable load-deformation characteristics and be installed at the right time. The use of this requires knowledge of the inter-relationships between ground deformation and load as well as between support deformation and load.

This generally requires a support system consisting of systematic rock bolting and shotcrete. Whatever support system is used, it is essential that it is placed and remains in intimate contact with the surrounding ground and deforms with it.

The *timing of rock support* installation is another important factor for a favourable mobilisation of the inherent strength of the rock mass:

- If the rock support is installed too early, a heavier support is required to carry the resulting rock mobilised.
- An installation made too late may cause deformations of the rock mass surrounding the tunnel, resulting in loosening and failure.

It is, however, difficult to predict the time factor and its variations during tunnelling even for experienced rock mechanics and tunnelling engineers, see the ground reaction curve, Section 4.4.4. In order to investigate the behaviour of the ground during and after excavation, correct application of NATM is based on systematic in situ measurements primarily of deformations and stresses. From the progress of the deformations, it is possible to recognise early enough if an unacceptable trend appears and to act accordingly. Thus, monitoring of tunnelling in weak ground is not research, but a simple, essential means of finding whether the tunnel construction is proceeding satisfactorily.

Table 9.14 The NATM classification applied in the *Austrian ÖNORM* B 2203(1993).

CLASS and TERM	ROCK MASS CONDITIONS	ROCK SUPPORT FUNCTION AND/OR EXCAVATION MEASURE REQUIREMENTS
A1 Stable	Elastic behaviour. Small, quick, declining deformations. No relief features after scaling. The rock masses are long-term stable.	No need for rock support after scaling. Not necessary to reduce length of rounds, except for technical reasons.
A2 Slightly ravelling	Elastic behaviour, with small deformations which quickly decline. Some few small structural relief surfaces from gravity occur in the roof.	Occasional rock support in roof and upper part of walls necessary to fasten loose blocks. The length of rounds might only be limited for constructional reasons.
B1 Ravelling	Far-reaching elastic behaviour. Small deformations that quickly decrease. Jointing causes reduced rock mass strength, as well as limited stand-up time and active span*. This results in relief and loosening along joints and weakness planes, mainly in the roof and upper part of walls.	Systematic rock support required, but only in moderate amount. The length of rounds is determined from the stand-up time and the time required to install initial support.
B2 Strongly ravelling	Deep, non-elastic zone of rock mass. The deformations will be small and quickly reduced when the rock support is quickly installed. Low strength of rock mass results in possible loosening effects to considerable depth followed by gravity loads. Stand-up time and active span are small, with increasing danger for quick and deep loosing from roof and working face.	Systematic rock support required in roof and walls, and often also of the working face. The cross section of the heading depends on the size of the tunnel, i.e., the face can contribute to stability. The length of the rounds must be reduced accordingly, respectively, systematic use of support measures such as spiling bolts ahead of the face.
C1 Squeezing or swelling	"Plastic" zone of considerable size with detrimental structural defects such as joints, seams, shears. Plastic squeezing as well as rock spalling (rock burst) phenomena. Moderate, but clear time-dependent squeezing with only slow reduction of deformations (except for rock burst). The total and rate of displacements of the opening surface is moderate. The rock support can sometimes be overloaded.	Rock support of the whole tunnel surface is required, often also of the working face. The size of the heading should be chosen to effectively utilise stabilising effect of the face. The effect of the rock support is mainly to limit the breaking up and to maintain the 3-dimensional stress state. The length of the round must be adjusted according to the support measures ahead of the working face.
C2 Strongly squeezing or swelling	Development of a deep squeezing zone with severe inwards movement and slow decrease of the large deformations. Rock support can often be overloaded.	Comprehensive rock supporting works required in all excavated rock surfaces. The size of the unsupported surface after excavation is to be limited according to support measures performed ahead of the face. The large deformations require use of special support designs, for example deformation slots or other flexible support layouts. The support should be installed to maintain the 3-dimensional state of stress in the rock masses.

* Active span is the width of the tunnel or the distance from support to face in case this is less than the width of the tunnel.

The instruments are installed in sections along the tunnel when the initial support is placed. In addition to hazard control, the information from the measurements is related to the characteristics of the ground and the size of the opening. When interpreted in an appropriate way, it is possible to adapt the type and dimensions as well as the timing of rock support to the actual ground conditions encountered during the excavation.

In Austrian tunnelling practice, the ground is described behaviourally and allocated a ground class in the field, based on field observations, see Table 9.14. Construction and support can be estimated from this classification, as shown in Table 9.15 The qualitative ground description used is associated, rather inconsistently, with excavation techniques together with principles and timing of standard support requirements. The rather unsystematic use of geo-data is a drawback, which limits communication between people involved in tunnelling as well as a further development of the NATM.

The NATM has successfully been applied in a large number of tunnels in many parts of the world, some of which were constructed in poor and difficult ground conditions. Considerable cost savings have often been gained as well as reduced construction time compared to traditional tunnelling. NATM has, however, also experienced many unpleasant downfalls and some tunnel collapses.

Table 9.15 The construction and support principles in the NATM (from Bieniawski, 1989)

Class	Construction Procedure	Principle	Support procedure			
			Crown	*Springline*	*Invert*	*Face*
A1	Check crown for loose rock. When popping rock is present: placement of support after each round	Support against dropping rock blocks	Shotcrete: t = 0-5 cm Bolts: locally as needed; C = 15 t; L = 2-4 m	Bolts: locally; C = 15 t; L = 2-4 m	No	No
A2	Crown has to be supported after each round. Bolted arch in crown	Shotcrete support in crown Bolts: C = 15 t; L = 2-4 m; D = 4-6 /m²	Shotcrete: t = 5-10 cm with wire fabric (3.12 kg/m²) Bolts: locally; L = 2-4 m	Shotcrete: t = 0-5 cm	Bolts: if necessary; L = 3.5 m	No
B1	Shotcrete after each round: Other support can be placed in stages	Combined shotcrete and bolted round in crown and at springline	Shotcrete: t = 5-15 cm with wire fabric (3.12 kg/m²) Bolts: C = 15-25 t; L = 3-5 m	Shotcrete: t = 5-15 cm Bolts: C = 5-15 t; L = 3-5 m; D = 3-5 /m²	Adapt invert support to local conditions	Adapt face support to local conditions
B2	Shotcrete after each round Bolts in the heading have to be placed at least after each second round	Combined shotcrete and bolted arch in crown and springline; if necessary closed invert	Shotcrete: t = 10-15 cm with wire fabric (3.12 kg/m²) Bolts: fully grouted; C = 25 t; L = 4-6 m; D = 2-4 /m²	Same as crown	Slab: 20-30 cm	Adapt face support to local conditions
C1	All opened sections have to be supported at least immediately after opening. All support placed after each round	Support ring of shotcrete with bolted arch and steel sets	Liner plates: locally. Shotcrete: t = 15-20 cm with wire fabric (3.12 kg/m²) Steel sets: TH21 spaced 0.8-2.0 m Bolts: fully grouted; C = 25 t; L = 5-7 m; D = 1-3 /m²	Same as crown, but no linerplates necessary	Invert arch > 40 cm or bolts L = 5-7 m, if necessary	Shotcrete: t = 10 cm in heading (if necessary) t = 3-7 cm in bench
C2	As Class C1	Support ring of shotcrete with steel sets, including invert arch and densely bolted arch	Linerplates: where necessary. Shotcrete: t = 20 - 25 cm with wire fabric Steel sets: TH21 spaced 0.5-1.5 m Bolts: C = 25 t; L = 6-9 m; D = 0.5-2.5 /m²	Same as crown	Invert: > 50 cm Bolts: if necessary; L = 6-9 m	Shotcrete: t = 10 cm and additional face breasting

C = bolt capacity; L = bolt length; D = density of bolts (bolts/m²); t = thickness of shotcrete

8.3.5 Some comments on classification systems for rock support estimates

The main classification systems and methods for rock support assessment all apply input of important parameters such as rock mass features, stresses and ground water. The parameters are, however, used differently as shown in Table 9.16.

Table 9.16 The various parameters applied in the Q, RMi and RMR rock support systems (from Palmström, 2000).

	PARAMETER	APPLICATION				
		in the Q system		*in the RMi method*		*in the RMR system*
Rock	Rock strength	-		σ_c	uniaxial compressive strength	uniaxial compressive or point load strength
Jointing	Degree of jointing	RQD	rock quality designation	Vb	block volume	rock quality designation (RQD) joint spacing
	Joint sets (pattern)	J_n	joint set number	Nj	joint set factor	-
	Joint character	J_r	joint roughness number	jR	joint smoothness and waviness factor	joint roughness
	Joint coating or infilling	J_a	joint alteration number	jA	joint coating, filling and weathering factor	joint infilling, gouge joint weathering
	Joint size	-		jL	joint length and continuity	joint length, persistence
	Joint aperture	-		-		joint separation
	Joint orientation	-		Co	joint orientation factor	orientation of joints
Water	Ground water	J_w	joint water reduction factor	-		leakage condition
Stress	Rock stresses	SRF	stress reduction factor	SL	stress level factor	-
Tunnel	Tunnel dimensions	Dt Wt ESR	span wall height excavation support ratio	Dt Wt	span or diameter wall height	-
Ground	Rock mass strength	-		$RMi = 0.2\sqrt{jR \times \frac{jL}{jA}} \times Vb^D$		-
	Ground quality (in roof)	$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$		$Gc = RMi \times SL$		RMR = sum of the ratings for each factor above
	Scale factor (in roof)	$De = \frac{Dt}{ESR}$		$Sr = \left(\frac{Dt}{\sqrt[3]{Vb}}\right)\left(\frac{Co}{Nj}\right)$		-

The support charts used in the various classification systems or methods to determine rock support are based on experience from numerous underground projects. Being statistically based, a support chart can never replace or accurately represent the ground conditions at a specific site. The main reason for this is that all the actual geometrical features of discontinuities cannot be included in a support chart. An understanding of the geological conditions at a site is therefore a prerequisite for a good characterisation of the rock mass and the ground conditions.

For swelling and slaking rock the stability may be strongly influenced by local conditions. In such cases rock support should be evaluated separately for each case. The influence of single seams (filled joints), shears and similar discontinuities to a great extent will often depend on site-specific geometrical and structural features, and hence also here the use of support charts may give inaccurate results.

Other factors that should be assessed separately have to do with local safety requirements, the required lifetime of the opening, any possible influence from vibrations due to earthquakes or nearby blasting or any other impact from human activity.

The steadily increasing trend in the fields of engineering geology and rock mechanics to substitute mathematical idealizations for geological reality often leads to a reduced interest in uncertainty and the quality of input parameters. Especially therefore, experienced people with a practical background from underground construction should be more widely used in the collection of data for use in the mathematical models.

Correlations between the various classification systems for rock support estimates are often used. Some of them are:

$RMR = 9 \ln Q + 44$ or $Q = e^{(RMR - 44)/9}$ (Bieniawski, 1984)

$RMi \approx 0.01Q \times \sigma_c$ or $Q \approx RMi/0.01\sigma_c$ (for $Q > 1$) (Palmström, 1995)

$RMi \approx 10^{(RMR - 40)/15}$ or $RMR \approx 15 \log RMi + 40$ (best for $RMR < 70$) (Palmström, 1995)

As shown in Table 9.16 the classification systems for rock support only have some parameters in common. Erroneous results may be introduced when the transitions above are used.

8.4 The Norwegian method of tunnelling (NMT)

The term *Norwegian method of tunnelling* (*NMT*) was introduced a few years ago to describe principles, design features and construction practice for tunnelling mainly in *hard, jointed rock*.

The main point of NMT is *fast tunnel excavation at low cost*, especially for tunnels in massive and jointed rock masses under low as well as high stress regimes. NMT has been developed mainly from experience gained in the excavation of approximately 5000 km of tunnels in Norway over the years. Thus, it includes the accumulated experience and refined techniques developed for planning, as well as investigation, contracting and excavation of underground structures.

NMT is basically empirical, observationally based tunnelling, and the contract system is structured accordingly, based on the principle that the contractor is paid for the amount of work which actually has been performed and needed according to the ground conditions encountered. Flexible rock support adjusted to the actual rock mass conditions plays an important role.

Norway has been in the forefront with regard to developing or improving tunnelling methods and in the use of innovations. An effective use of modern equipment combined with thorough planning has reduced the working crew to only 3 men. Another important aspect that has contributed to this is the good workmanship performed by skilled tunnelers who have refined the tunnelling activities, such as drilling, charging, and rock support, as well as loading and mucking out.

A prerequisite is that all the parties involved in the design and execution of the project - design and supervising engineers plus the contractor's engineers and foremen - must understand and accept the NMT approach and adopt a co-operative attitude with regard to making decisions and resolving problems. NMT thus encourages interplay between all the main aspects of modern tunnelling.

The ever-present uncertainty of what actual ground conditions are like has been met in Norwegian tunnelling contracts by a risk-sharing system. This system gives the parties a tool for converting work into time-equivalents. With this tool, any contingency arising from changing ground conditions can be met, reducing future discussions over adjustments to the construction time where needed, or over costs incurred. A main feature of this system is that all types of rock support and other work that may possibly be needed, are described. Each of the activities is specified in the Bill of Quantities as a separate cost-item. In addition, any work activities expected to influence tunnelling progress and construction time can be converted into time using time-equivalents. There are also provisions in the contract for adjusting the contractual construction time, calculated by means of the work volume presented in the Bill of Quantities (the reference ground conditions) and the time-equivalents. The main benefits of the Norwegian risk-sharing contract system are reduced construction costs and fewer lawsuits or arbitrations over changed ground conditions.

9 Stability of rock slopes

Rock slopes may be *natural slopes* (mountain and valley sides) or *cuts* (quarry and open pit mine walls, road cuts, etc.). As will be discussed in the next chapter, important differences exist in the philosophy of analysing the stability of the two categories due to their different prehistory.

9.1 Classification of stability problems

When analysing rock slope stability, it is important to be aware of the basic difference between:

- *Short-term stability,* meaning stability over a limited period of time (a few tens of years), and
- *Long-term stability,* indicating stability over a considerably longer time-perspective (most commonly with reference to natural slopes).

There are basic differences in the way short-term and long-term stability problems are analysed. Weathering and creep, for instance, may be of major importance for long-term stability problems, but normally has only a minor influence on the short-term stability.

For high rock slopes it is often logical to distinguish between:

- *Total stability* or *overall stability*, indicating the stability of the entire slope, and
- *Detail stability*, indicating the stability of local areas, for instance one or a few benches of a large open pit mine.

Depending on the volume (V) of the instable mass, the following terms and classification are often used (Hestnes, 1980):

- *block-fall*: $V < 100\ m^3$
- *block-flow*: $100 < V < 10{,}000\ m^3$
- *rock slide*: $V > 10{,}000\ m^3$

Basically, it is the orientation of discontinuities relative to the rock slope that will define the potential *failure mode*. Depending on the relative orientation of discontinuities and the volumes involved, five main modes of failure may be identified as shown in Figure 10.1.

a) *Plane failure*
This is the most common failure mode in hard, unweathered rocks. Sliding may occur along one single discontinuity, or along a failure plane consisting of several discontinuities, sometimes separated by bridges of intact rock and/or with a tension joint at the back.

b) *Wedge failure*
Two intersecting discontinuities release a wedge-shaped rock mass that slides in the direction of the intersection line of the two discontinuities.

c) *Toppling*
The condition for toppling is a distinct set of discontinuities striking parallel to the slope and dipping steeply in the direction opposit the slope inclination.

d) *Circular failure*
This is a common failure mode in soil, particularly cohesive materials. Not particularly relevant for hard rock slopes, but it may occur in very heavily jointed and/or weathered rock.

e) *Block-fall and -flow*
Generally caused by surface processes such as weathering and erosion. Frequent block-falls may be advance warning of a major slide.

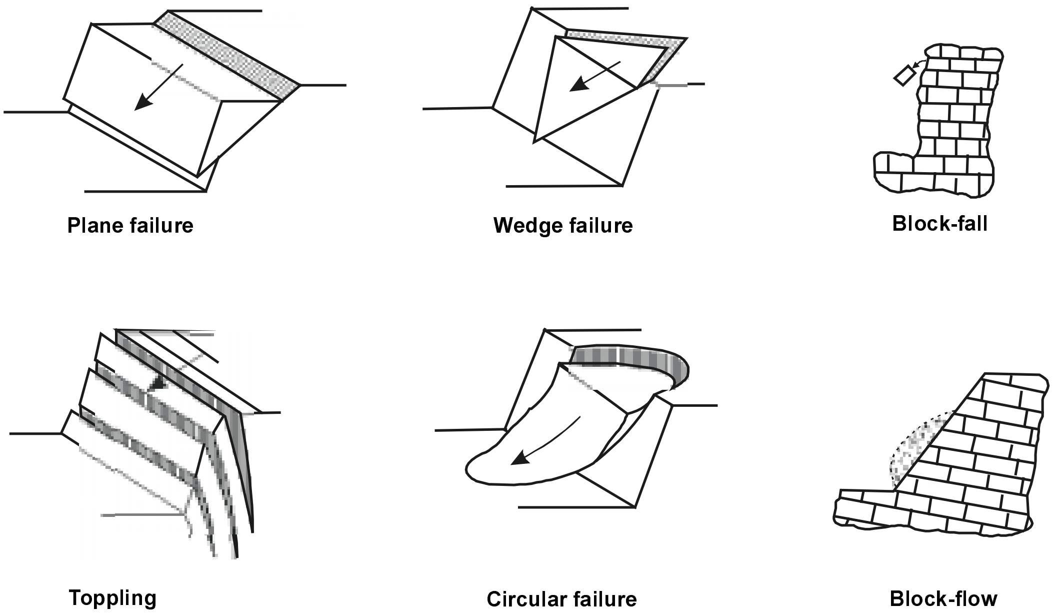


Figure 10.1 Potential failure modes (based partly on Hoek & Bray, 1991).

9.2 Factors affecting the stability

In principle, rock slope stability is governed mainly by the following geological and non-geological factors:

- Rock type boundaries and mechanical properties.
- Faults and weakness zones.
- Detailed jointing.
- Groundwater and climatic conditions.
- Rock stresses.
- Geometrical conditions.
- Blast vibrations and potential earthquake activity.

The relative importance of each of these factors may vary considerably, even within limited areas. However, in most cases the orientations and characteristics of discontinuities and the groundwater conditions are the most important factors; the discontinuities defining the potential failure mode as well as, together with the groundwater pressure, the risk of failure.

To analyse rock slope stability it is vital to have knowledge of how each of the factors influence the stability. Therefore, before performing stability analyses, a systematic mapping of each of the factors has to be carried out.

9.3 Stability analysis

A variety of sophisticated tools for analysing rock slope stability exist, and calculation can be performed with a high degree of accuracy. Still, a great deal of uncertainty is often connected to such analyses. This is not due to the principles of calculation, but to the steps of analysis that precede calculation.

Rock slope stability analysis is much more than just calculation. Typically, the analysis represents 3 steps as indicated in Figure 10.2:

1) Definition of the potential stability problem.
2) Quantification of input parameters.
3) Calculation of stability.

9.3.1 Definition of the potential problem

This first step is a crucial part of the analysis. Obviously, severe errors may be the result of misinterpretations at this stage, and one should always make sure that sufficient effort is spent here.

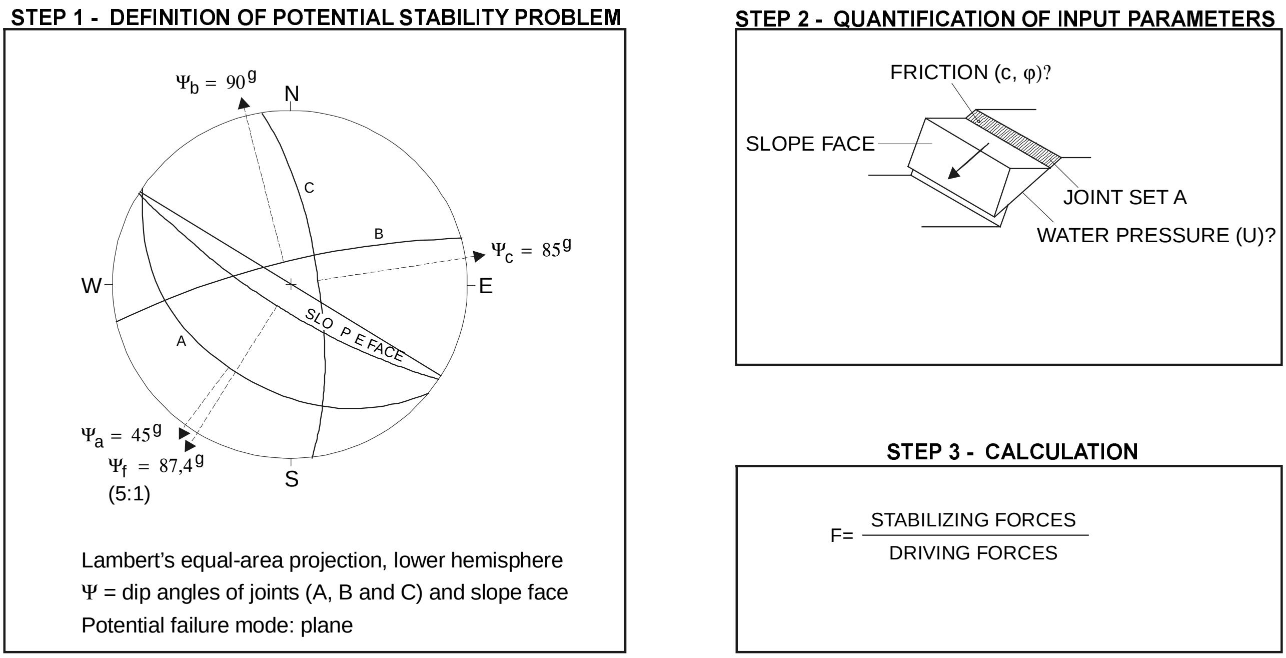


Figure 10.2 Typical step-wise procedure for analysing rock slope stability (from Nilsen, 1996).

Engineering geological mapping is a key factor here, and since the great majority of slides occur along discontinuities, the main emphasis is on the mapping of joints. In most cases, particularly in hard rock, the potential failure mode (i.e., plane, wedge or toppling failure) is very quickly evident. Occasionally, *stereonet analysis* as illustrated in the left part of Figure 10.2 may be required to define the potential failure mode, and in potential wedge failure situations, for instance, stereographic projection is also useful in defining the geometry.

In hard rock, the definition of a potential stability problem is normally not the most critical part of the analysis. Conscientious engineering geological mapping is the main way to eliminate major mistakes here. In the great majority of practical cases, plane failure is the potential instability.

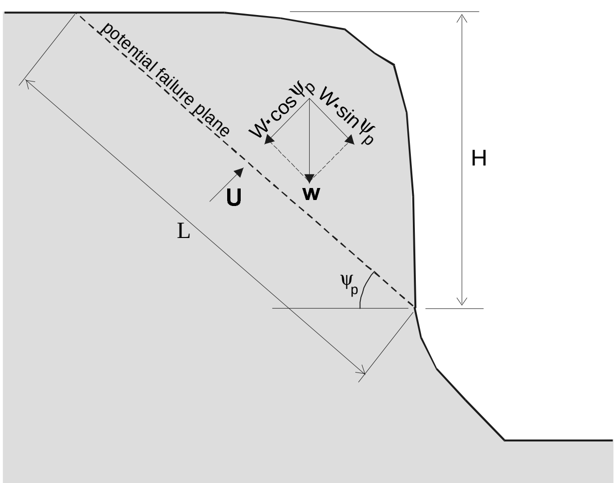
In some cases the geometry of a potential sliding body is more complex, and occasionally the sliding plane may be stepped or contain bridges of intact rock. In such cases, definition of potential stability problems is more difficult, but normally far from impossible, provided that the rock mass is not heavily weathered or covered by soil or vegetation.

9.3.2 Quantification of input-parameters

The main input data for limit equilibrium analyses are as defined in Figure 10.3. Although this example represents a very simple and idealised case, the main input will always have to do with:

- Geometry
- Friction
- Water pressure

The geometrical data (H, 1 and ψ_p in Figure 10.3) are defined by field mapping, occasionally supplemented with stereographic projection techniques as described in Section 10.3.1, and normally do not represent critical parts of the analysis. In most cases the main problems are to quantify the friction parameters and the water pressure.



H = height of unstable slope
ψ_p = inclination of potential failure plane (40^o in example)
L = H/sinψ_p = length of potential failure plane
γ_r = specific gravity of rock (2.7 t/m^3 in example)
γ_w = specific gravity of water (1.0 t/m^3)
W = $\gamma_r H^2/(2\tan\psi_p)$ = weight of potentially sliding rock (t/m)
U = resultant of cleft water pressure (t/m)
σ_n = W$\cos\psi_p$U/l = normal stress acting on failure plane (t/m^2)
ϕ_a = active friction angle (c = 0)
F = $(W\cos\psi_a - U)\tan\varphi_a / W\sin\psi_p$ = factor of safety against sliding

Stable according to the partial factor principle if: $(W\cos\psi_a - U)\tan\phi_a > W\sin\psi_p$ (partial factors to be applied on actions and ground properties as shown in Table 7.2).

Figure 10.3 Principle for limit equilibrium analysis of potential plane failure.

9.3.2.1 Friction parameters

The great majority of discontinuities in rock masses have irregularities (often referred to as *first-* and *second-order irregularities,* or "waviness" and "bumps"), see also Section 3.2.6.

Because of the dilation and shearing of irregularities that takes place during shear displacement, and the transition between the two effects, a non-linear relationship exists between shear strength and normal stress of rough joints as indicated in Figure 10.4. This is a crucial point that should never be neglected in limit equilibrium analyses.

Because of the non-linearity of the shear strength curve, the general Coulomb's equation will not be valid. However, for a defined normal stress level (σ_{nl}), and by introducing the parameter "*apparent cohesion*" (c_l), a modified Coulomb's equation can be applied (see Figure 10.4):

$$\tau_l = c_l + \sigma_{nl} \times \tan\phi_l$$

where ϕ_l = friction angle.

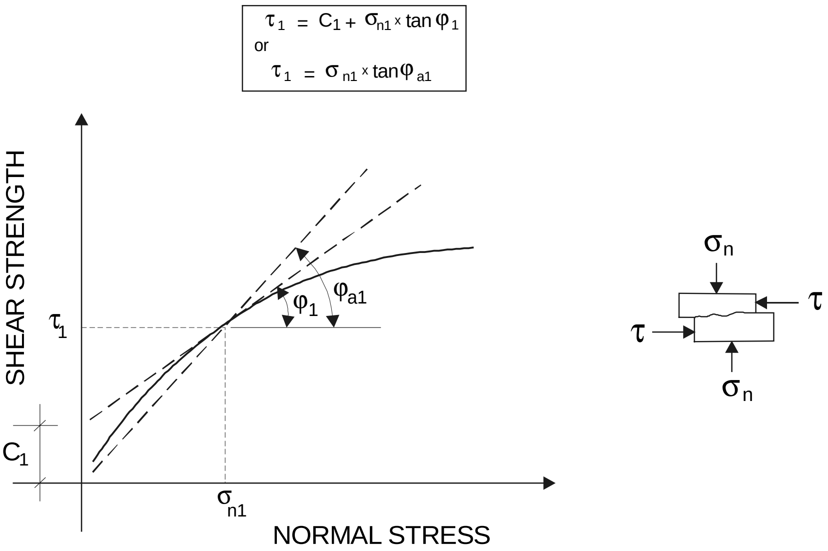


Figure 10.4 Definition of apparent cohesion (c_l), and total or "active" friction angle (ϕ_{al}), (from Nilsen, 1985).

The parameter c_l may be interpreted as being the contribution to the shear strength caused by shearing through irregularities, while ϕ_l is the contribution due to frictional resistance during shearing.

Introducing the parameter *active friction angle* (ϕ_a) may be an alternative to using c_l/ϕ_l. In this case, the effect on shear strength due to irregularities will also be included in the friction angle (consequently, the cohesion c = 0). Referring to Figure 10.4, the active friction angle is defined by the equation:

$$\tau_l = \sigma_{nl} \times \tan\varphi_{al}$$

Using ϕ_a simplifies the arithmetic of the stability analysis. For both alternatives it is, however, essential that the shear strength parameters are in accordance with the normal stress level in question. Far too often, shear strength parameters are based on stress levels that are too high.

In principle, there are 4 main alternatives for finding the friction parameters:
1. Laboratory shear testing,
2. Field shear testing,
3. Back analysis, and
4. Empirical methods.

Laboratory shear testing has the general disadvantage that only a small portion of a large potential sliding plane is being tested. Large scale field testing, as described by Hoek & Bray (1991) may give reliable results, but has the disadvantage of often being too complicated to carry out and very expensive. *Back analysis* of former slides may be an attractive alternative, but a limitation here is that the geological conditions have to be practically identical for the method to be feasible.

As a result of shortcomings in the alternatives, *empirical methods* have become more and more recognised. Most commonly used today, is the method described by Barton & Bandis (1990), taking into consideration the joint roughness and alteration according to the following equation:

$$\tau = \sigma_n \tan\left[JRC \times \log\left(\frac{JCS}{\sigma_n}\right) + \phi_b \right]$$

where τ = shear strength
σ_n = normal stress
JRC = joint roughness coefficient
JCS = joint compressive strength
ϕ_b = basic friction angle

Correction due to *scale effects* is essential, particularly for high joint roughness, see Figure 10.5. Detailed descriptions of these corrections are given by Barton & Bandis (1990).

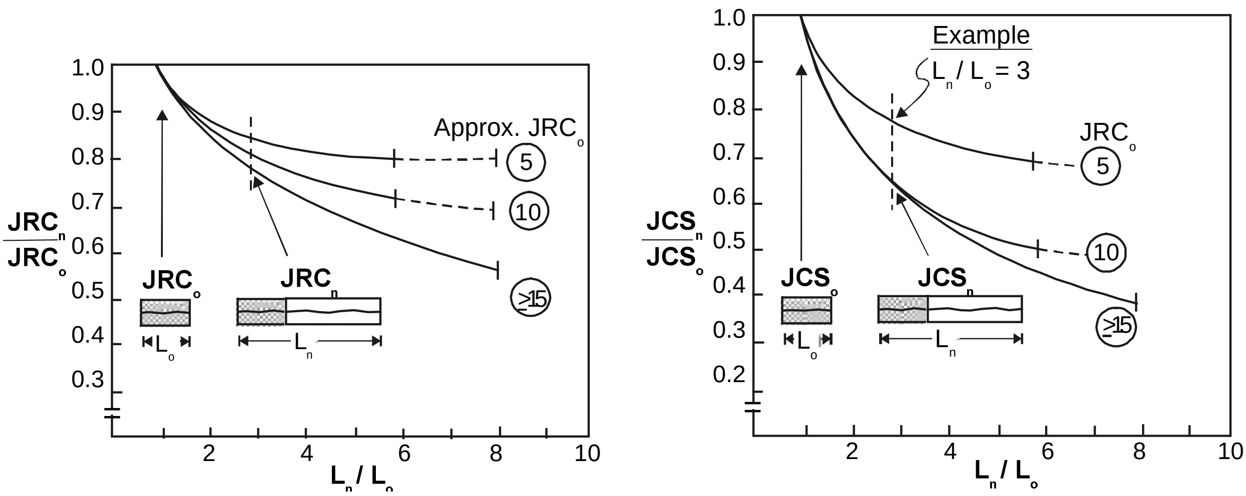


Figure 10.5 Corrections of JRC and JCS due to scale effects (from Barton & Bandis, 1990).

In any case, it is crucial for the stability analysis that the friction parameters are adjusted to the actual normal stress level. To illustrate the non-linearity of the *shear strength curve*, some shear testing results of strike joints in mica schist are shown in Figure 10.6. (A portable shear machine manufactured by Robertson Research Ltd was used for the test, the shear machine and the testing procedure are described in Hoek & Bray, 1991.) The plots represent water-saturated condition and *peak shear strength*. The stippled curve to the left in Figure 10.6 represents power curve fit with a coefficient of determination (r^2) of 0.88.

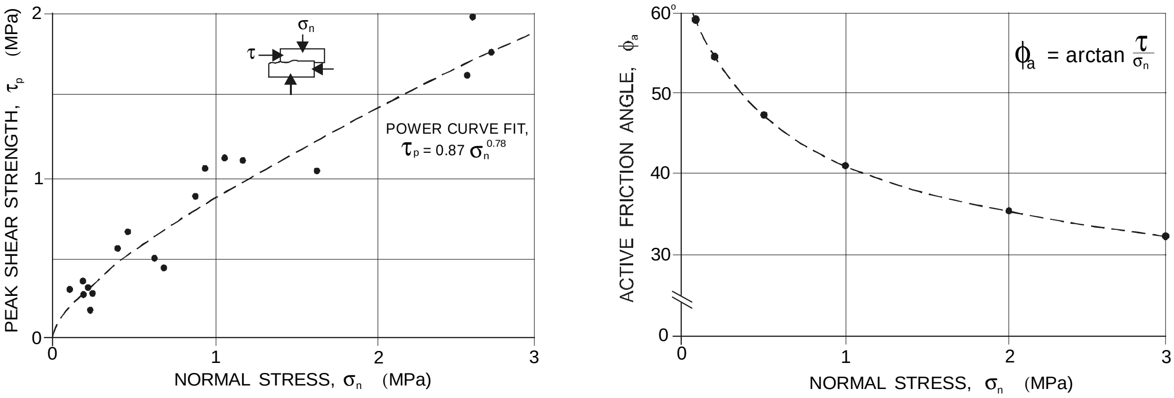


Figure 10.6 Shear strengths and active friction angles of foliation joints in mica schist (from Nilsen, 1985). Each indicated value represents one single shear test at a constant normal stress. The joints had medium to low roughness, many of them a thin coating of chlorite/mica, and some were slickensided. Specimens were 32 mm and 35 mm drill cores.

9.3.2.2 Water pressure

The *water pressure* of the potential sliding plane is crucial for the stability. For the situation in Figure 10.3, the water pressure tends to reduce the normal stress, and hence the frictional resistance. In some cases water may act in tension joints, and thus directly represent a driving force.

The idealised extremes concerning water pressure are as indicated in Figure 10.7 a) and c). In case a) there is no water present and consequently the water pressure is 0. In case c) water is assumed to enter freely on the top of the slope, and increase hydrostatically along the discontinuity to a maximum value at the toe. Case b) also represents a situation where water enters freely on the top, but is fully drained at the toe after having reached a maximum pressure corresponding to the hydrostatic at a height corresponding to 50% of the slope height.

Case c) represents a very high water pressure, and is realistic only if the discontinuity is completely blocked at the toe of the slope, for instance by ice or concrete. Experience from cold climates indicates that such blocking due to ice is rather rare.

The triangular distribution of case b) is idealised, and almost certainly does not match perfectly with the situation during heavy rainfall. This is, however, the configuration most commonly used to model the resultant water pressure during heavy rainstorm. Practical experience indicates that the triangular distribution probably in most cases exaggerates the resultant pressure because joints and cracks often cause drainage towards the slope face. Very rarely the opposite, due to blocking of the discontinuity near the toe of the slope, is the case.

Reliable modelling of the water pressure distribution is in most cases the most difficult part of a rock slope stability analysis. If comprehensive field testing is not technically or economically feasible, assumptions according to the triangular distribution in Figure 10.7 c) are most relevant for modelling water pressure during heavy rain conditions. Concerning field testing or measurement, this may be realistic in existing slopes, while at the planning stage it is more difficult to obtain reliable input.

9.4 Calculation/evaluation

In principle, the following alternatives exist for quantifying or evaluating the slope stability:

- Empirical analysis
- Limit equilibrium methods
- Numerical analysis
- Physical models

In the great majority of cases, a simple two-dimensional limit equilibrium method as shown in Figure 10.3 is most relevant for analysis.

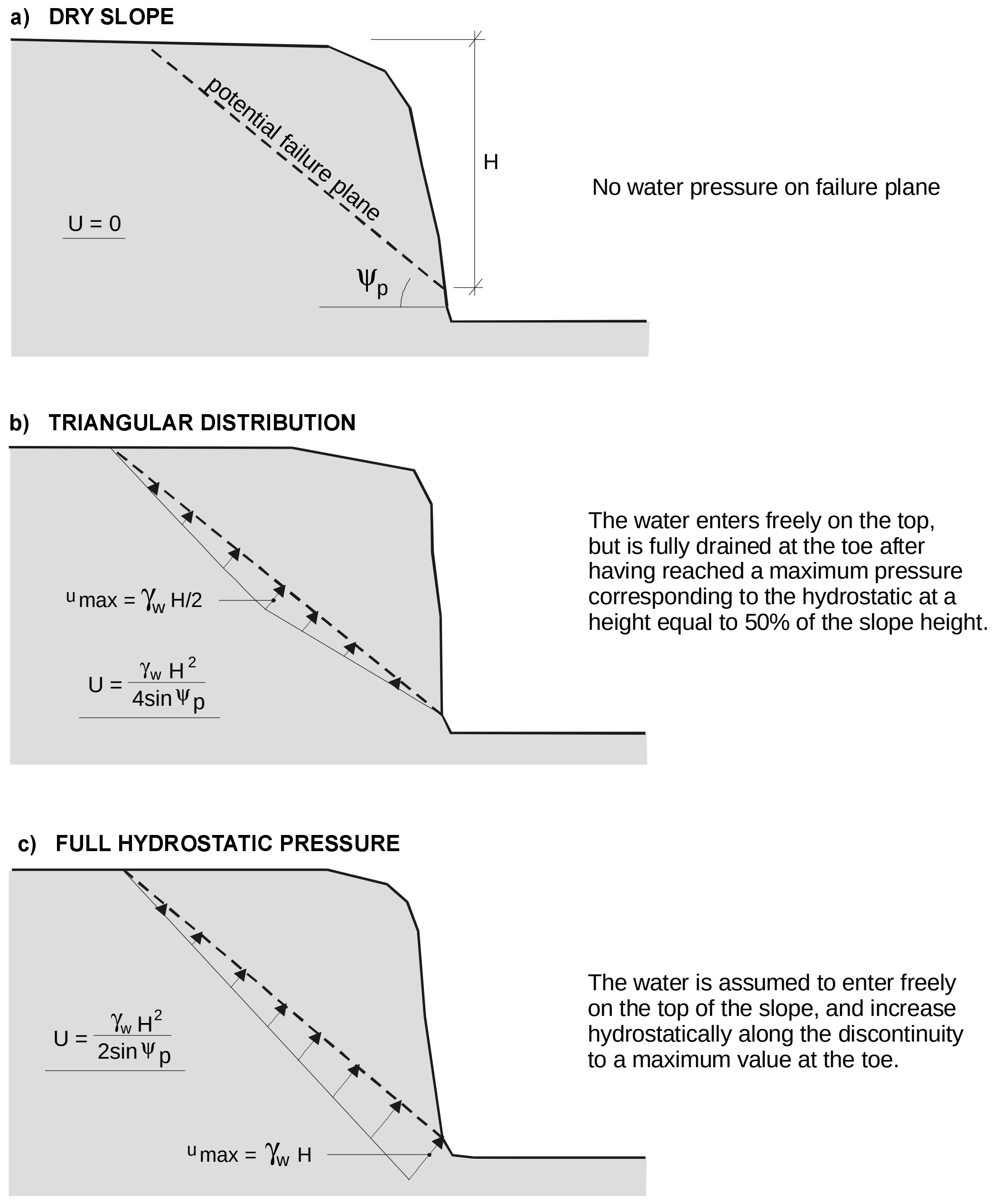


Figure 10.7 Alternative configurations of water pressure along potential sliding plane. Maximum and resultant pressures are given (u_{max} and U, respectively), from Nilsen (1985).

9.5 Consequences of erroneous input data

The simple case in Figure 10.3 will be used here to illustrate the importance of reliable shear strength and water pressure parameters for analysing rock slope stability. For the case of saturated condition, a triangular distribution of water pressure as shown in Figure 10.7 b) is assumed. For the sake of simplicity, the degree of stability in the example is defined by applying the factor of safety instead of the limit state principle, which, as described in Section 7.2, should be used in future cases.

As friction parameter the active friction angle (ϕ_a) is applied. The shear strength curve shown in Figure 10.6 is assumed to be representative of the shear strength of the potential failure plane.

The analysis is based primarily on the non-linear relationship represented by the power curve fit in Figure 10.6. As an alternative the assumption c = 0 and $\phi = 45^o$, representing an apparently reasonable linear curve fit for the normal stress regime in question, is used, see Figure 10.8. This assumption of linearity represents a misunderstanding which, unfortunately, is rather common when shear strengths are estimated.

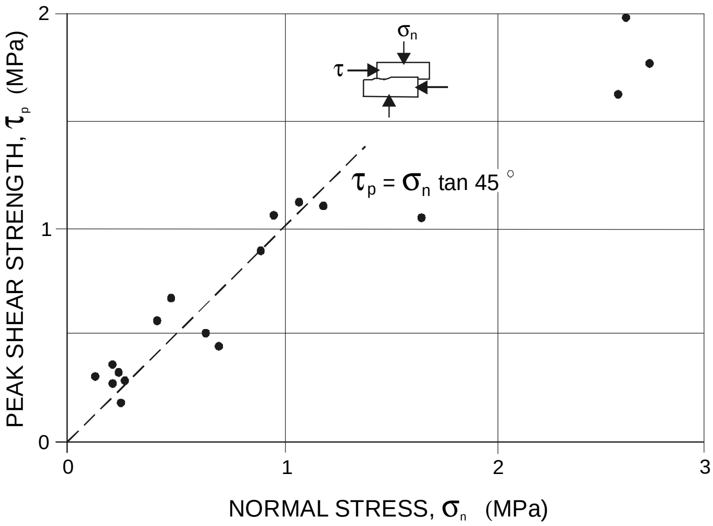


Figure 10.8 Example of erroneous interpretation of the shear strength data shown in Fig. 10.6.

The normal stress (σ_n) has been calculated for slope heights 20, 50, 100 and 200 m, and for saturated and dry conditions, see Table 10.1. The respective active friction angles (ϕ_a) corresponding to the various normal stresses according to Figure 10.6 (lower part), and the resulting factors of safety (F) according to the equation in Figure 10.3, are also shown.

Table 10.1 Estimation of normal stress (σ_n), active friction angle (ϕ_a) and corresponding safety factor (F) based on Figures 10.3 and 10.6.

H (m)	Saturated slope			Dry slope		
	σ_n (MPa)	ϕ_a	F	σ_n	ϕ_a (MPa)	F
20	0.11	59.0^o	1.36	0.16	56.5^o	1.80
50	0.27	52.5^o	1.06	0.40	49.5^o	1.40
100	0.54	46.5^o	0.86	0.79	43.0^o	1.11
200	1.09	40.0^o	0.68	1.59	37.0^o	0.90

In Figure 10.9 the resulting factors of safety for this example are plotted against slope heights. As can be seen, for dry as well as for saturated slope, the factor of safety decreases considerably as the slope height increases. It can also be seen that the difference in safety factor between dry and saturated condition is very considerable (about 25% reduction from dry to saturated condition).

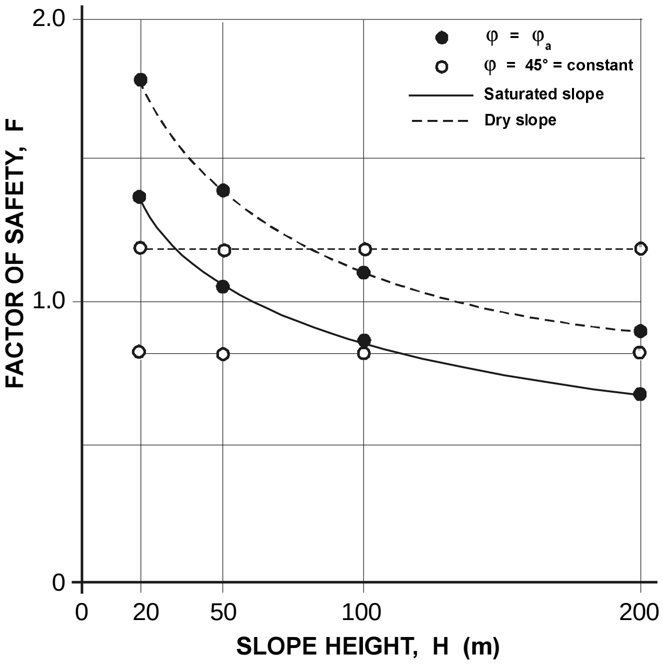


Figure 10.9 Factors of safety for $\phi = \phi_a$ according to Figure 10.6 and for the erroneous assumption in Figure 10.8 of ϕ = constant = 45°.

By substituting for U and W in the F-equation, H (the slope height) is eliminated for dry as well as saturated condition. Hence, for the case of ϕ = constant (independent of normal stress), the factor of safety generally is independent of slope height. This obviously erroneous result is represented by the horizontal lines in Figure 10.9.

10 Ground vibrations

Vibrations in the ground may be caused by natural phenomena such as plate tectonics and faulting, or construction activities such as blasting, boring, piling, etc. If sufficiently strong, the ground vibrations represent a major risk to life and property.

In this Chapter, emphasis is put on the two categories of vibration of main importance in rock engineering:

1) Earthquake vibrations.
2) Blast vibrations.

10.1 Earthquake vibrations

As a result of in situ stresses (see Chapter 5), the earth is continuously undergoing deformation. If the stresses continue to build up over a long time, sudden fracturing and release of energy may cause an *earthquake*. The energy most commonly is released from a fault in a confined region below the surface, called the *focus*. The point on the surface vertically above this point is called the *epicenter*, see Figure 11.1.

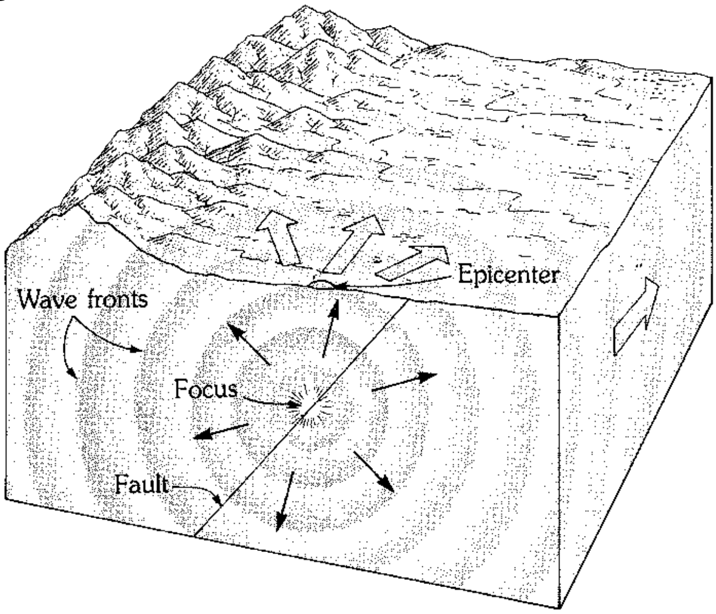


Figure 11.1 Principle sketch of the mechanism of an earthquake (from Tarbuck & Lutgens, 1986)

Earthquakes are known to originate at depths ranging from about 5 to nearly 700 kilometres. The common classification according to depth is as shown in Table 11.1.

Table 11.1 Classification of earthquake depth (after Tarbuck & Lutgens, 1986).

Depth of focus	**Classification**
< 60 km	Shallow
60-300 km	Intermediate
> 300 km	Deep

About 90% percent of all earthquakes occur at depths of less than 100 km, and all very strong quakes appear to originate at shallow depths.

10.1.1 Origin and types of waves

The major part of the earthquake energy is released through elastic waves spreading spherically from the focus as shown in Figure 11.1. When these waves propagate within the rock mass (or rock body), they are referred to as *body waves*. Of this category there are two types, see also Figure 11.2:

1) Primary, or *P-waves*, which are longitudinal, i.e., as they spread out alternately push (compress) and pull (dilate) the rock mass. As indicated by the name, these are the fastest of the waves. P-waves involve change of volume, and the waves can travel through solid rock as well as soft and liquid materials.

2) Secondary, or *S-waves*, which as they propagate, shear the rock mass sideways at right angles to the direction of travel. S-waves involve distortion without change of volume.

When the wave front reaches a sharp boundary between media of different properties, such as the ground surface, the energy is partly reflected, and partly refracted. Resulting waves with motion restricted to near the surface, i.e., with amplitude decreasing with depth, are called *surface waves*. As shown in Figure 11.2, there are two types of this category as well:

3) *Love waves*, which are generally the fastest of the surface waves. The motion is essentially the same as for S-waves, with ground movement from side to side and no vertical displacement These waves are possible only for non-uniform conditions, i.e., when the S-wave velocity of the surface layer is less than for the layer below (as is the case in a situation with soil overlaying solid rock).

4) *Rayleigh waves*, which are much like rolling ocean waves, with particle movement both vertically and horizontally. These waves can be generated in uniform ground conditions also.

10.1.2 Earthquake intensity and magnitude

Earthquake intensity refers to human reaction and degree of damage to the surroundings. A reliable scale for intensity classification was first developed by Guiseppe Mercalli in 1902. Today, a modified form of the *Mercalli intensity scale* is widely used, see Table 11.2.

Classification based on intensity is not always adequate for comparison. Many factors, such as distance from the epicenter, nature of the surface materials and building design, cause variations in the amount of damage sustained. In addition, the epicenter of earthquakes does not in many cases coincide with populated areas. For such reasons, methods have been established for describing quantitatively how much energy is released during an earthquake, referred to as *earthquake magnitude*.

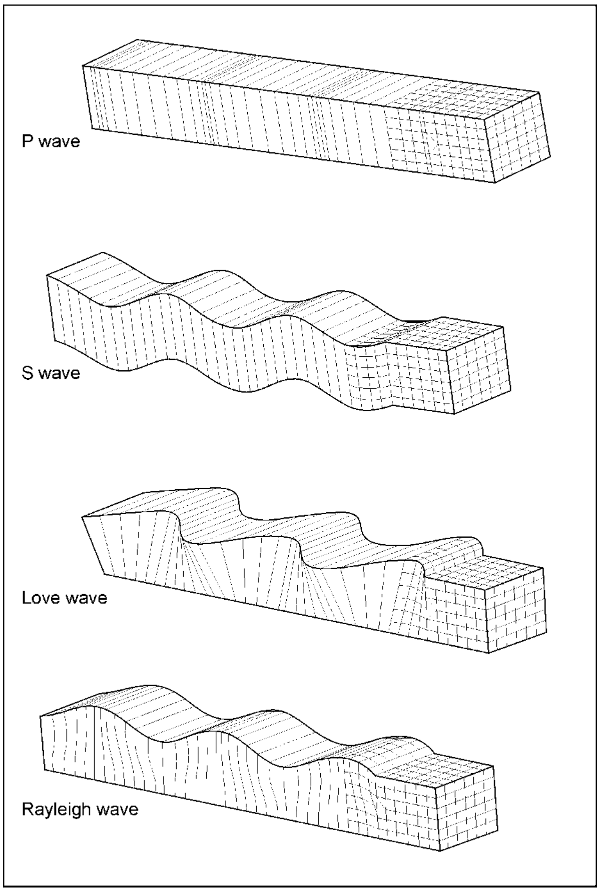


Figure 11.2 Principle sketches of the different types of ground motion for the four types of near surface earthquake waves (from Jacobs, 1981).

Table 11.2 The modified Mercalli intensity scale (after U.S. Coast and Geodetic Survey).

Intensity	Description
I	Not felt, except by a very few persons under especially favourable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognise it as an earthquake.
IV	During the day felt indoors by many, outdoors by few. Feeling as though a heavy truck was striking the building.
V	Felt by nearly everyone, many awakened. Disturbances of trees, poles and other tall objects sometimes noticed.
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or badly designed structures.
VIII	Damage slight in specialy designed structures, considerable in ordinary substantial buildings with partial collapse, great in poorly built structures. (Fall of chimneys, factory stacks, columns, monuments, walls.)
IX	Damage considerable in specially designed structures. Buildings shifted off foundations. Ground cracked conspicuously.

Table 11.2 The modified Mercalli intensity scale. *Continued.*

Intensity	Description
X	Some well built wooden structures destroyed. Most masonry and frame structures destroyed with foundations. Ground badly cracked.
XI	Few, if any (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground.
XII	Damage total. Waves seen on ground surfaces. Objects thrown upward into air.

Magnitude can ideally be determined from the amount of material that slides along the fault and the distance it is displaced. In practice, however, to determine it this way is rarely possible, and therefore intensity is determined by measuring the amplitude of the largest wave recorded on a seismograph.

A modified version of a method originally developed by Charles Richter in 1935 is used worldwide to determine earthquake magnitude. Today's refined *Richter scale*, based on 100 kilometres as standard recording distance and the Wood-Anderson instrument as the standard recording device, is shown in Table 11.3.

Table 11.3 Earthquake magnitudes and expected world incidence (from Tarbuck & Lutgens, 1986).

Richter Magnitudes	Earthquake effects	Estimated earthquakes/yr
< 2.5	Generally not felt, but recorded.	900,000
2.5 - 5.4	Often felt, but only minor damage detected.	30,000
5.5 - 6.0	Slight damage to structures.	500
6.1 - 6.9	Can be destructive in populous regions.	100
7.0 - 7.9	Major earthquakes. Inflict serious damage.	20
≥ 8.0	Great earthquakes. Produce total destruction to nearby communities.	One every 5 – 10 years

The largest earthquakes ever recorded (Chile in 1960, Alaska in 1964), had Richter magnitudes of about 8.6. (corresponding to approximately 10^{26} erg energy release, or equivalent to the detonation of 1 billion tons of TNT). The weakest quakes possible to record with today's sensitive instruments have magnitudes down to -2 (in comparison, magnitude 2.5 is the limit for what is usually felt by humans).

To accommodate the huge variation in earthquake amplitude and energy, the Richter scale is logarithmic, with an increase of one on the magnitude scale representing a tenfold increase in wave amplitude, and roughly a 30-fold increase in the energy released. Thus, a magnitude 6 earthquake has amplitude 10 times greater/releases 30 times more energy than a magnitude 5 quake, and has 100 times greater amplitude/roughly 900 times more energy than a magnitude 4.

10.1.3 Effects of earthquakes

The major effects of earthquakes may be broadly grouped into two general classes:

1) *Faulting*, i.e., direct primary displacement of bedrock.
2) *Shaking*, i.e., ground displacement due to wave propagation.

Faulting may or may not carry through to the surface, and generally is limited to relatively narrow, active fault zones. Typical displacements vary from a few centimetres to metres.

Obviously, tunnels should not be located across active faults if possible, since they are almost certain to be completely ruptured, and potentially damaged, in any major earthquake. In seismically active areas one should also keep in mind that even those faults thought to be inactive, may come alive.

Concerning shaking, shear movement represents the greatest potential risk of damage. In underground structure analyses the S-waves therefore are of prime interest, and in surface structure analyses the Love waves. Except for in the immediate fault area, tunnels in competent rock are very rarely seriously damaged by earthquake shaking. In poor rock formations, and particularly close to faults, shaking due to a strong earthquake may, however, loosen the rock and cause stability problems, also underground.

To conclude, it is a general experience from major earthquakes that for a given intensity of shaking, tunnels and underground structures are much safer than surface installations, and safer the deeper they are. Only minor damage to rock tunnels, mainly concentrated to portal areas, has been experienced in areas subjected to earthquake intensities of VIII to IX, although damage to surface structures has been extensive. This is due to the effects of surface waves.

10.1.4 Principles of risk assessment, design and analysis

For risk assessment seismological as well as geotechnical factors have to be taken into consideration. Important seismological information includes:

- Historical data on earthquake occurrence intervals, magnitudes and associated parameters.
- Proximity of faults.
- Historic evidence of slippage of the faults and magnitudes of actual offsets with their recurrence intervals.

Similarly, the required geotechnical information includes:

- Stratigraphic section and properties of the various layers.
- Location of the water table.
- Geophysical data, especially seismic shear wave velocity.

In *seismic design*, i.e., design taking into consideration the potential consequence of earthquakes, information on anticipated magnitude is essential. In this connection, the following terms are generally used (Monsees, 1996):

- *Maximum Design Earthquake* (*MDE*), representing the earthquake event with a return period of several thousand years. It has a small probability of exceedance, approximately 5% or less, during facility design life. The MDE defines the level at which critical elements continue to function to maintain public safety, preventing catastrophic failure or collapse and loss of life.
- *Operating Design Earthquake* (*ODE*) is the event with a return period of several hundred years. It can reasonably be expected to occur during the facility design life, with a probability of exceedance of about 40%. In the ODE, critical elements of the facility maintain function, and the overall system continues to operate normally.

In most cases, it is impractical to design a structure to survive the conditions expected from the most severe earthquake that statistically might occur during the useful life of the project. If the design life is in the range of 50 years, for example, it is usually inconsistent to design to survive an earthquake with a 500-year recurrence interval.

For analysing potential consequences of earthquakes, numerical analyses are commonly used (for instance the ABAQUS-program mentioned in Section 12.4.1). In many cases limit equilibrium methods are also used, with *seismic acceleration* as the key input parameter. Seismic acceleration (a_s) is given as a fraction of acceleration of gravity (g), for instance: $a_s = 0.25$ g.

In rock slope stability analysis, the so-called *pseudo-statical principle* is commonly used, i.e., to consider the maximum earthquake load as an equivalent horizontal load, as this direction is generally the most unfavourable. For the situation in Figure 10.3 the equivalent horizontal load F_s is:

$$F_s = m \times a_s \qquad (F_s = 0.25W \text{ for } a_s = 0.25 \text{ g})$$

where W = weight of the unstable rock mass

In limit equilibrium calculation, the earthquake load is decomposed the same way as other forces affecting stability (for instance W).

10.2 Blast vibrations

In a properly designed blast, the major part of the energy is used in crushing and breaking the rock. Some energy, however, also causes *air shock* and *blast vibration*, i.e., vibrations in the surrounding rock.

The portion of blast energy causing vibrations depends on the character of the rock mass, the geometry and the history of the blast. Thus, for tough, massive rocks and very confined blast rounds the vibrations will generally be much greater than for weaker rocks and less confinement. Particularly strong vibrations are experienced in cases when one or several holes of a blast do not detonate properly (so-called *miss(fir)ed round)*.

10.2.1 Wave propagation

When a blast detonates, shock waves propagate spherically from the centre of the blast as so-called *body waves*, in principle in the same way as shown in Figure 11.1. Some of the energy is reflected from discontinuities and any free faces (such as neighbouring tunnels or caverns, for instance).
When the wave front reaches the surface, *surface waves* are created. Thus, the components of waves from a blast basically are the same as described for earthquakes (see Section 11.1.1):

- Body waves:
  - Primary, or *P-waves*.
  - Secondary, or *S-waves*.

- Surface waves:
  - *Love waves*.
  - *Rayleigh waves*.

The resultant wave is characterised by particles on the surface moving around their points of equilibrium, and in principle may be compared to waves in water. There is no mass movement involved in the wave propagation.

As described in Section 11.1.1, the P-waves are the fastest of the components. S-waves have a velocity around 50% of the P-waves, and for Rayleigh waves, which are generally the most important of the surface waves in this connection, the velocity is somewhat lower still. The P and S-waves, however, attenuate much more quickly than the Rayleigh waves, and often at some distance away from the blast, only the latter may be possible to identify.

In addition to different *attenuation characteristics*, the different wave components also have different phases, amplitudes and frequencies. The resultant wave therefore seldom has amplitude higher than the peaks of the individual components.

The vibrations of a blast generally are complex and irregular. In practice, to simplify calculation and analyses of recorded data, the vibrations are described by harmonic oscillations (sinus-waves), see Figure 11.3.

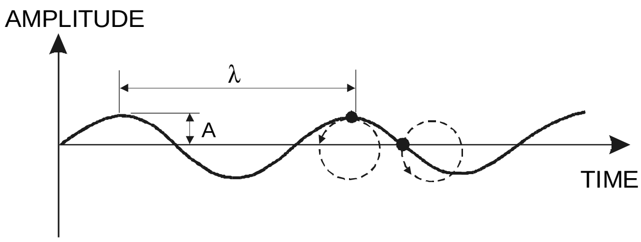


Figure 11.3 Principle sketch of harmonic wave movement. Circles indicate particle movement for Rayleigh wave.

For harmonic waves, the following relationships apply between characteristic parameters:

$$c = 2\pi \times A \times f$$
$$a = 4\pi^2 \times f^2 \times A$$
$$v = f \times \lambda$$

where A = amplitude (μm)
f = frequency (Hz)
λ = wave length (μm)
c = particle velocity (mm/s)
a = particle acceleration (m/s^2)
v = seismic velocity (m/s)

Seismic velocity (v) and *particle velocity* (c) have completely different meanings, and it is crucial not mix the two parameters. The former represents the travelling velocity of the wave, and in hard rock has a typical value in the order of 4,000 - 6,000 m/s (see also Section 8.3.5). The latter indicates the distance travelled per time unit of a surface particle around it's equilibrium point, and is given in mm/s. Thus, the seismic velocity (v) is in the order of 4^{th} to 5^{th} power greater than the particle velocity (c).

The resultant wave contains a wide spectrum of different frequencies. Thus, the frequency is not defined as single value, although it normally is within a relatively narrow, so-called *dominating frequency domain*. The vibration frequency spectrum depends mainly upon:

1) The distance from the blast. The frequency generally tends to increase dramatically with decreasing distance. Thus, at a distance of one meter or less, the dominating frequency in

solid hard rock is often several thousands of Hz, while at a distance of more than 10 m it is normally within the region of 100-200 Hz.

2) The ground conditions. Compared to the 100-200 Hz of solid hard rock, the dominating frequency in soft rock or heavily fractured rock at some distance from the blast is typically 40-70 Hz, and in soil around 40 Hz. (Tamrock, 1989).
3) The degree of simultaneous detonation (charge weight per detonator interval).

The *natural frequency* of tall buildings, according to Berthelsen (1992) may be estimated as:

$$f_b = \frac{46}{H}$$

where f_b = natural frequency of building (Hz)
H= height of building (m)

As indicated by this equation, amplification of blast vibrations due to *resonance* in tall buildings is not very likely to occur, regardless of foundation conditions (while for earthquake vibrations, having a much lower frequency, the situation is quite different).

10.2.2 Vibration acceptance criteria

Criteria for avoiding damage due to blast vibrations, so-called *blast vibration acceptance criteria*, have to take into consideration several factors, such as type of structure, technical installations and occupancy and dominant frequency.

Most criteria for this purpose are based on recording the maximum particle velocity (c), for which an empirical equation of the following type is used for calculating the anticipated, maximum value of c:

$$c = k\,\frac{\sqrt{Q}}{R}$$

Where: k = the "rock constant"
Q = charge per delay interval (kg)
R = distance from the blast (m)

The so-called "*rock constant*" (k) is far from constant, but depends on several factors, of which the following are the most important:

- The distance from the blast: k generally decreases distinctly with increasing distance.
- Discontinuities in the rock mass: k is generally lower for heavily jointed rock than for massive rock.
- Anisotropy: in anisotropic rocks, k depends upon wave direction relatively to rock structure (and generally is highest parallel to the bedding, foliation or other planar feature).

Despite the non-constant character of k, empirical equations such as the one above represent a valuable tool in the calculation of acceptable charge. The common procedure is to estimate k in each individual case based on trial blasts, and then calculate permissible charge per delay interval and for a given limit of particle velocity (c).

Alternatively, the calculation of permissible charge can be based on a similar empirical equation for amplitude.

Vibrations are measured using vibrographs, which record the particle velocities or particle acceleration and dominating frequencies. The sensors are mounted in three orthogonal directions on building basements or other points on structures and register separately.

In most cases the vertical surface waves have the highest amplitude, and the main emphasis is therefore put on monitoring these. However, the horizontal component of the resultant wave also has to be taken into consideration. This is particularly the case in unfavourable topography, as for a building located in a hillside to the side of the blast. In such cases the horizontal component may have amplitude and frequency of the same order of magnitude as the vertical. In addition, buildings are often more vulnerable to horizontal movement than vertical.

The *acceptance limits* depend upon the frequency range of the dominant vibrations and type of structure or installation, for which particle acceleration, velocity or amplitude may be used. For structures, *peak particle velocity* and *maximum amplitude* are normally applied as limits, and are specified for the full frequency range. For rotating machinery and computer installations *peak particle acceleration* represents the limit (and is also the key parameter for assessing dynamic stability of slopes). Typical safe limits in tripartite logarithmic graph are shown in Figure 11.4.

Humans are very sensitive to vibrations. Thus, as compared to the limits shown in Figure 11.4, for a frequency of 50 Hz, an amplitude of only 3 μm is easily felt by humans, while the limit for damage on buildings is about 200 μm.

Different countries have different criteria for what are acceptable limits concerning blast vibrations. Some countries have developed criteria taking into consideration a wider range of relevant factors. Thus, according to the *Norwegian Standard NS 8141* (NBR, 1993), maximum permissible particle velocity is calculated according to an equation taking into consideration the ground conditions as well as type of structure, distance from the blast and duration of the work:

$$c = c_o \times F_k \times F_d \times F_t$$

where c = maximum permissible peak particle velocity (mm/s).
c_o = uncorrected peak particle velocity for the actual seismic velocity (mm/s)
F_k = correction factor based on construction type and quality = $F_b \cdot F_m$.
F_b = factor defined by construction type.
F_m = factor defined by material used.
F_d = correction factor based on distance from the blast.
F_t = correction factor based on duration and character of work.

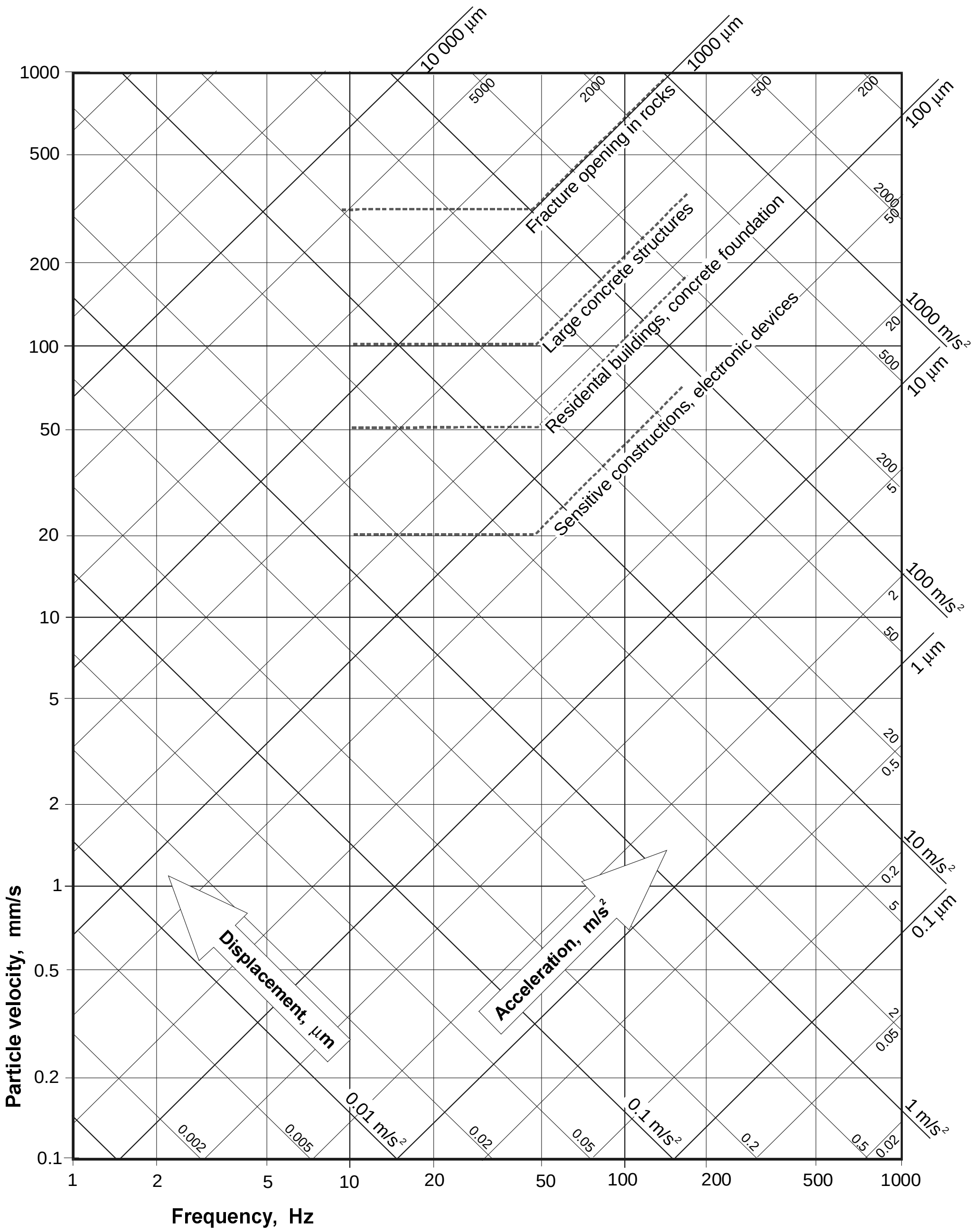


Figure 11.4 Typical safe vibration limits (from By, 1985).

The correction factors are defined as shown in Table 11.4. As an example, for blasting work of 10 months duration at a distance of 60 meters from a residential building made of concrete, the limit for acceptable vibration according to this criterion is c = 60 mm/s if the building is founded on solid hard rock, and only 15 mm/s if it is founded on clay.

Table 11.4 Values of uncorrected peak particle velocity (c_o) and correction factors (F_k, F_d and F_t) according to NS 8141 (based on NBR, 1993)

Uncorrected peak particle velocity, c_o		
Ground condition		*Value of v_o (mm/s)*
Very soft ground/soft clay		to be defined individually
Loosely layered moraine, sand, gravel, clay (seismic velocity $v < 2000$m/s)		18
Firmly layered moraine, soft rocks like shale, limestone, etc. ($2000 < v < 4000$ m/s)		35
Hard rocks like granite gneiss, quartzite, etc. ($v > 4000$ m/s)		70
Construction factor, F_b		
Class	*Type of construction*	*Value of F_b*
1	Massive constructions like bridges, piers, fortifications, etc.	1.70
2	Industrial and office buildings	1.20
3	Residential buildings	1.00
4	Particularly sensitive buildings like museums, large spans, etc.	0.65
5	Historical buildings, fragile ruins, etc.	0.50
Factor F_m		
Class	*Main construction material*	*Value of F_m*
1	Reinforced concrete, steel, wood	1.20
2	Unreinforced concrete, concrete elements, bricks, etc.	1.00
3	Light concrete, etc.	0.75
Distance factor, F_d		
Distance from blasting, d		*Value of F_d*
$d < 5$ m		to be defined individually
$5\text{m} < d < 200\text{m}$		$0.5 + 0.5(200 - d)/195$
$d > 200$ m		0.5
Time factor, F_t		
Duration of blasting work		*Value of F_t*
Less than 12 months		1.00
More than 12 months		0.75

10.2.3 Remedial measures

The obvious measure to eliminate or reduce problems due to blast vibrations is to optimise the blast design; i.e., to reduce the unit charge (charge per delay interval). The easiest way to achieve this is by increasing the number of delay intervals by using *non-electric* (*Nonel*) ignition systems or by mixing different types of electric detonators. Also, reduction of the round length may be considered, or dividing the cross section into two or more sections.

As illustrated by the indicative cost curve in Figure 11.5, the extra costs of adapting blast design to strict limits, can be very high. The figures shown apply to cavern excavation in Hong Kong, and the cost effect, according to Berthelsen (1992), may be somewhat reduced by using non-electric ignition.

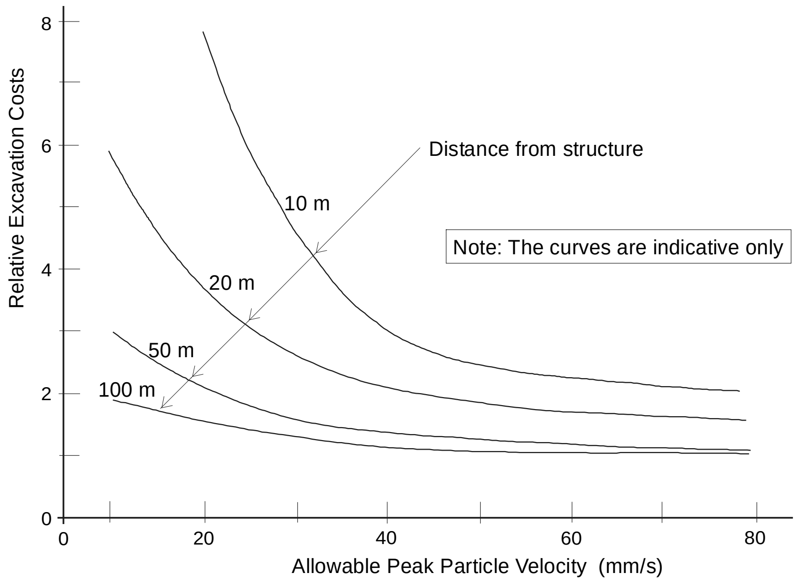


Figure 11.5 Cost related to vibration restrictions for cavern excavation in Hong Kong (from Berthelsen, 1992).

Even when restricted according to engineering criteria, blast vibrations can be a nuisance to the public. Potential problems can be reduced, however, if efficient communication is established with people in the local environment, informing them about the type of activity, times of blasting, effects to be anticipated, etc. Finally, to be prepared in case of claims, registration and documentation of the conditions of surrounding buildings is vital.

11 Numerical modelling

Numerical modelling means *discretization* of the rock mass into a large number of individual elements and involves handling of such a large quantity of data that powerful computers have to be used. In rock engineering numerical methods are used mainly for analysing rock stresses and deformations. Other important issues such as heat- and water-flow may, however, also be analysed using basically the same methods.

11.1 Definitions

Numerical methods represent a sub-group of the so-called *analytical methods*, in which analysis is based on calculation or modelling, as opposed to *empirical methods*, in which analysis is based purely on experience and comparison. As shown in Figure 12.1, there are two main categories of numerical models:

1. Continuous models
2. Discontinuous models

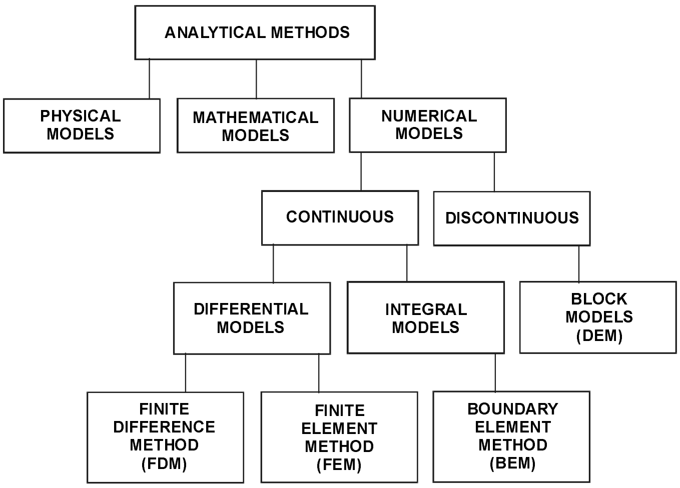


Figure 12.1 General classification of numerical methods for rock mass analyses (based on Bieniawski, 1984).

Continuous model means that the rock mass is modelled as a basically continuous medium. Only a limited number of discontinuities (joints, faults, etc.) may be included here. This is the most commonly used category of numerical models.

Discontinuous model (or *block model*) means the rock mass is being modelled as a system of individual blocks interacting along their boundaries. Obviously, when compared with the nature of rock masses, the concept of discontinuous modelling represents important advantages. The method belonging to this category is referred to as the *Distinct Element Method* (*DEM*).

Based on whether the entire model or just the portion closest to the actual project is discretized, continuous models are divided into:

- *Differential models*, represented by the *Finite Element Method* (*FEM*) and the *Finite Difference Method* (*FDM*) (the difference between them represented mainly by the mathematics applied to solve the equations).
- *Integral models*, represented by the *Boundary Element Method* (*BEM*.

It should be stressed that numerical analysis often has the character of parameter study more than exact calculation with definite answers. For this reason it is crucial that evaluation by experienced engineering geologists is always included in the interpretation of the results from such analyses.

11.2 Main principles of modelling

The first step of numerical modelling is to define a *geological model* of the actual area. The next step is to generate the *element mesh*, and thus the *element model*. In stress analyses, the magnitudes and directions of stresses for all nodal points of the element mesh are finally computed based on input on rock properties and boundary conditions.

The basic principle of numerical analysis is illustrated by the geological and element models of a planned open pit mine in Figure 12.2. Based on the results of geological investigations, the rock mass surrounding a planned open pit mine is here divided into 4 different rock mass sectors. The discretization into elements follows the rock type boundaries of the geological model, and each element is assigned mechanical properties according to test results of the actual rocks. Concerning *boundary conditions*, the nodal points at the bottom of the model are free to move horizontally only, while the nodal points at the unloaded vertical boundary are free to move vertically only.

An example of a simple FEM-model (homogeneous and isotropic conditions) for a planned underground project is shown in Figure 12.3.

The size of the elements is getting smaller close to the contour of the excavation. This is generally done in numerical modelling because these are the areas of prime interest when analysing stability and planning rock support. A useful feature of the model is the step-wise removal of elements of critical areas, making it possible to study the stress development during the last 200 meters of excavation.

The computed directions and magnitudes of principal stresses are normally presented as shown in Figure 5.2. The magnitudes and directions of the major and minor principal stresses are given by the vector lengths and directions, respectively, of each of the crosses.

In Figure 12.3, results of non-linear FEM analysis of the 61 m span Gjøvik cavern built for the 1994 Winter Olympic Games in Norway are shown. In addition to calculated directions and magnitudes of principal stresses, calculated displacements are also shown here. The numerical analyses demonstrated the general stability of the cavern and during construction, fairly good agreement was found between numerical predictions and field measurements (Lu et al., 1994).

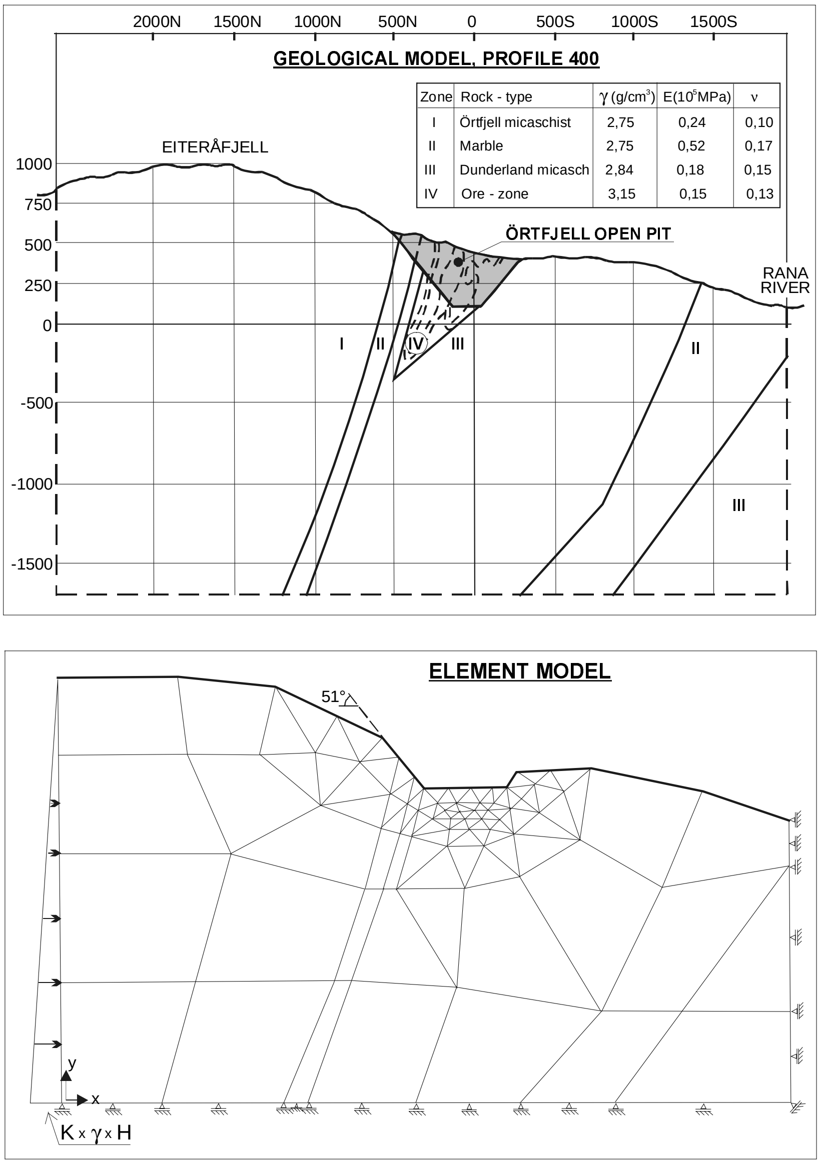


Figure 12.2 Example of geological model (upper part) and FEM-model of planned open pit mine (from Nilsen, 1979).

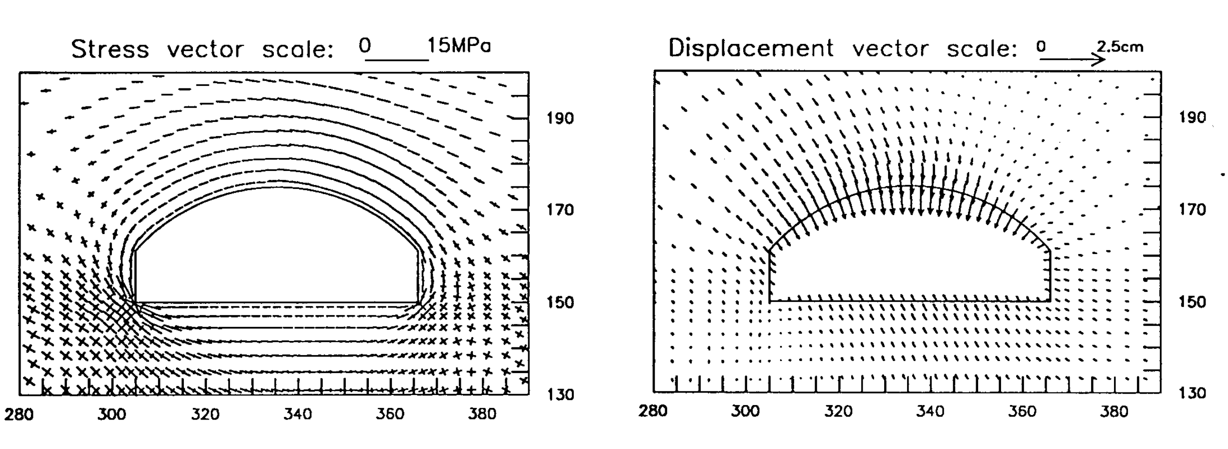


Figure 12. 3. Results from FEM analyses of the Gjøvik ice-hockey cavern (from Lu et. Al.,1994).

11.3 Input parameters, uncertainties of analysis

The relevant mechanical input parameters are as indicated in Figures 12.2, specific gravity (γ), modulus of elasticity (E) and Poisson's ratio (ν).

Vertically, as in the above examples, the numerical model is loaded normally by gravity only, and the vertical stress is:

$$\sigma_v = \gamma \times H$$

where σ_v = vertical stress (MPa)
H = distance below surface

Horizontal load is applied at one of the vertical model boundaries by introducing a so-called "*K-value*", including the effect of gravitational as well as tectonic stress:

$$F_H = K \times \gamma \times H$$

where F_H = horizontal boundary load
K = $K_o + K_t$ = "K-value" (not to be confused with the "k-value" described in Section 5.1.3).
K_o = $\nu/(1-\nu)$ = gravitationally induced component
K_t = tectonically induced component.

In most cases, the tectonically induced component contributes considerably to the K-value. Thus, in Norway, a K-value of 0.5 (representing $K_t = 0.17$ for a common rock mass Poisson-value of $\nu = 0.25$) or higher is found relevant in most cases

Numerical modelling can be carried out for practically any material model (linear or non-linear elastic, plastic, elasto-plastic, visco-elastic, etc.) and failure criterion, and is therefore very useful for many purposes, including design evaluation and stability analyses as shown by the above examples. It is important to be aware, however, that restrictions and uncertainties are also connected to such modelling. The main uncertainty is represented by the difficulty of obtaining reliable input parameters. The following are generally the most difficult to quantify:

- The magnitudes and directions of the virgin stresses.
- The properties of the in situ rock mass.

The problem of obtaining reliable input concerning stresses is considerably reduced, of course, if underground access exists. Alternatively, rock stress measurements may be carried out at an early stage of excavation, and decisions concerning details of the design postponed until this is done. In any case, a valuable basis for *comparative analysis* (i.e., study of the effect of changes in design) can be obtained by numerical analyses.

The uncertainty connected to rock mass properties is represented mainly by the representativity of sampling and scale effects. Considerable scale effects particularly are connected to the Young's modulus, for which the laboratory value often is in the order of 50% higher than for the in situ rock mass.

One should always keep in mind that the reliability of the analysis will never be better than the reliability of the input parameters. An important thing to remember is also that for two-dimensional analyses, which are still most commonly used, the computed stresses due to the lack of confining effects may be up to 50% higher than for the more realistic three-dimensional analyses.

11.4 Some relevant computer programs

For continuous ground conditions, FEM-programs are most commonly used, but BEM and FDM are also used in many cases. For cases of discontinuous ground where single joints may strongly influence the behaviour of the structure, the use of DEM programs is recommended.

A variety of good quality computer programs are available on the market. In the following, brief comments are made on some of the most commonly used programs.

11.4.1 Continuous models

Concerning *FEM-programs*, the following are among the most common:

- *ABAQUS*
  The program is designed for linear as well as non-linear, three-dimensional stress analysis of structures of any dimension. Normally, rock bolts, joints and other elastic or plastic details are not described distinctly, but as the behaviour of an element including the distinct values distributed through its volume. When required, distinct description can, however, also be used.

  Abaqus modelling may be useful in design since it serves as a numerical simulation of the non-structural environment. Even though it may be difficult to obtain an "exact solution", the methodology gives the designer a very realistic, physical understanding of the behaviour of soil and rock masses. Programs are available for both two and three-dimensional modelling.

- *ANSYS*
  This is a program for solving the full range of design/analysis problems. Both linear and non-linear stress analyses can be performed for isotropic as well as non-isotropic material properties. The program includes a special concrete/rock element that can be used to analyse crack patterns in concrete and rock. It is a powerful tool for determining displacements, stresses, forces, temperature distribution, magnetic fields, fluid flow, pressure distribution and other important design issues.

Among *BEM-programs*, one of the most commonly used is:

- *BESOL* (Boundary Element Solutions)
  The program can be used to compute the stresses, strains and displacements around excavations of any shape in a variety of geological settings. The BESOL-system is based on three different boundary element methods: 1) the fictitious stress method, 2) the displacement discontinuity method and 3) the direct boundary integral method. Programs are available for both two and three-dimensional modelling.

A commonly used FDM-program in rock engineering is:

- *FLAC* (Fast Langrangian Analysis of Continua)
  An FDM-program like FLAC may in some cases have advantages over finite element programs since they handle large grid distortions (geometric non-linearity) and non-linear material models in almost the same calculation time as small-strain linear problems. Furthermore, plastic collapse and flow are modelled very accurately. The drawbacks of the *explicit formulation* (small timestep and choice of damping) are overcome by automatic

inertia and automatic damping. The explicit method is ideal for geomechanics problems that consist of several stages (loading, excavation, etc.) because it models the stages in the same way that they occur in reality. Programs are available for both two and three-dimensional modelling.

11.4.2 Discontinuous models

The most commonly used program of this category is:

- *UDEC* (Universal Distinct Element Code)
  This is a discontinuum modelling method specifically developed for blocky structures in which the mechanical discontinuities play an important role in the overall deformation behaviour, or where joint conductivity (leakage) is important. The method represents a tool for modelling and understanding the mechanical behaviour of jointed rock masses. The intact rock between joints can be modelled as linear elastic or plastic. Discontinuities can be modelled either with linear elastic properties or can be given bilinear or non-linear properties from the Mohr-Coulomb, Cundall's continuously yielding, or Barton-Bandis joint models.

  UDEC modelling can be carried out as in two or three dimensions based on the following main principles:

  1) Discontinuous systems are modelled as assemblages of blocks interacting through corner and edge contacts. The blocks can have different deformability, and either be rigid, simply deformable (3 degrees of freedom) or fully deformable (each block automatically subdivided into finite difference zones);

  2) Discontinuities are regarded as boundary interactions between the blocks and joint behaviour is prescribed for these interactions;

  3) Non-linear constitutive models including dilatant and non-dilatant behaviour describe the behaviour of the intact rock and discrete joints.

DEM-analyses may also be used to simulate stresses and deformations of planned rock support, and hence as a tool in support design as shown by the example in Figure 12.4. The discretization into block elements is shown in the upper part of the figure, rock bolt locations in the left tunnel and computed tensional forces acting on rock bolts in the lower part (as lines to the left and arrows to the right, respectively).

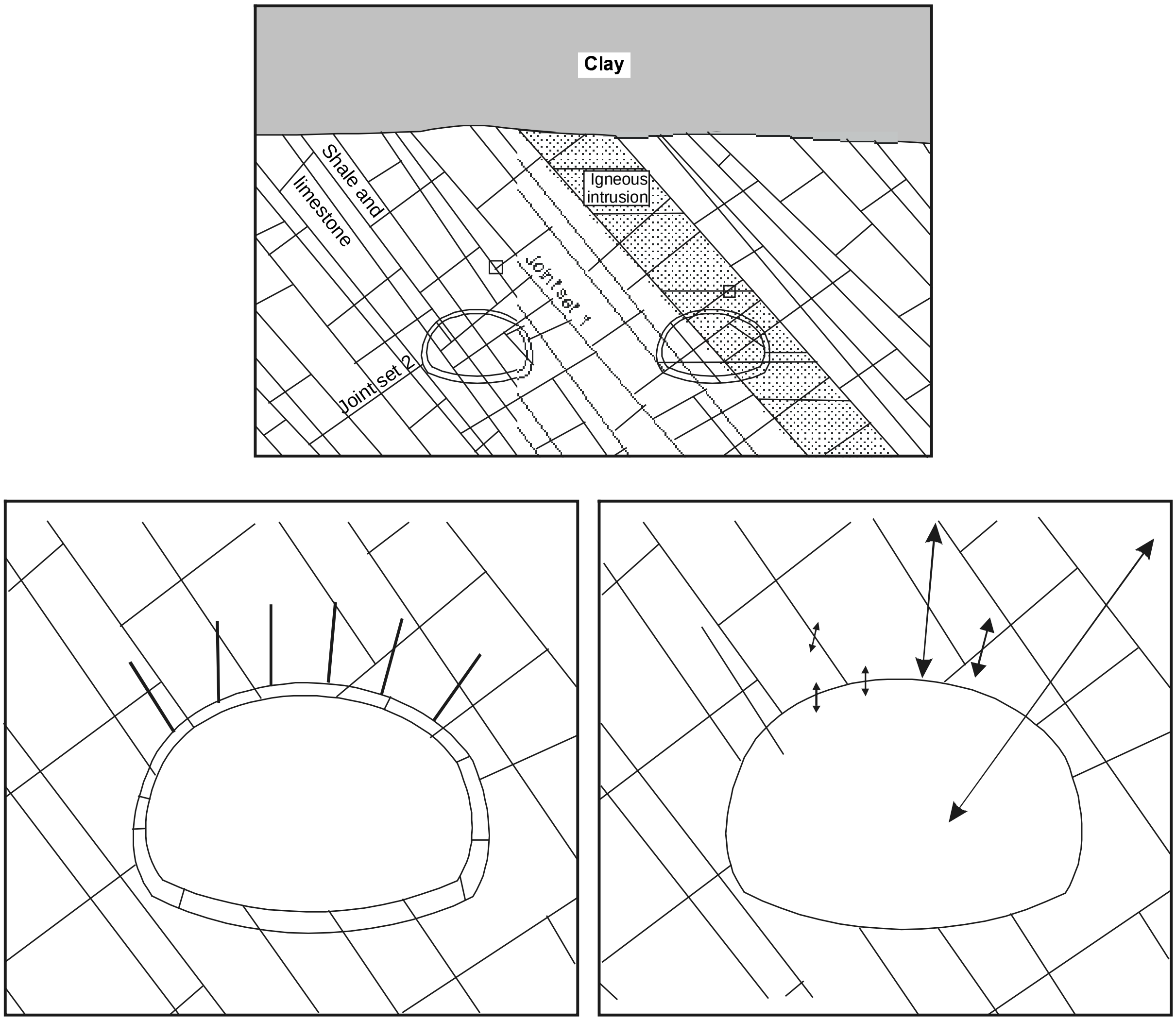


Figure 12.4 UDEC modelling of the Oslo road tunnel (from Makurat, 1988).

12 Miscellaneous

12.1 Classification summary

Table 13.1 shows a compilation of classification of some of the most used parameters in rock engineering and engineering geology. Many of the classifications have been suggested by the Norwegian Rock Mechanics Group (NBG) in the 1985 Norwegian Handbook.

Table 13.1 Classification of various ground features (revised from NBG, 1985).

	PARAMETER	SYMBOL	UNIT	CLASSIFICATION					REFERENCE
				very low or small	low or small	medium // moderate	high or large	very high or large	
ROCKS	Density	ρ	t/m^3	< 2.4	2.4 - 2.6	2.6 - 2.8	2.8 - 3.0	> 3.0	NBG 1985
	Unit weight	γ	kN/m^{3}x10						
	Uniaxial compressive strength	σ_c	MPa	1 – 5	5 – 15	15 – 50	50 – 120	> 120	ISRM
	Point load strength	Is	MPa	< 0.1	0.1 – 0.3	0.3 – 1	1 – 3	> 3	ISRM
	E-modulus	E	GPa	< 10	10 – 30	30 – 70	70 –100	> 100	ISRM
	Modulus ratio (E_{stat}/σ_c)	-	-		< 200	200 - 500	> 500		NBG, 1985
	Flakiness value	f	-		< 1.3	1.3 – 1.45	> 1.45		NBG, 1985
	Brittleness value	S_{20}	-		< 45	45 – 65	> 65		NBG, 1985
	Drilling rate index	DRI	-	< 33	33– 42	43– 57	58 – 69	> 69	NTNU, 1998
	Bit wear index	BWI	-	< 21	21 – 30	31–44	45 – 55	> 55	NTNU, 1998
	Cutter life index	CLI	-	< 6	6-7.9	8–14.9	15–34	> 34	NTNU, 1998
	Porosity	n	%	< 0.5	0.5 – 2	2 – 5	5 – 20	> 20	NBG, 1985
	Foliation anisotropy	fA	-	1 – 1.2	1.2 – 1.5	1.5 - 2	2 – 2.5	> 2.5	Palmström, 1995
	Permeability coefficient	k	m/s x 10^{-7}	< 0.001	0.001 – 0.1	0.1 - 10	10 – 100	> 100	NBG, 1985
	Weathering of rock	-	%		< 10	10 – 35	35 – 75	> 75	ISRM
	Slaking (two cycles)	I_{d2}	%	< 30	30 - 60	60 - 90	90 - 98	98 - 100	ISRM
	Mineral grain size	-	mm	< 0.02	0.02 – 0.6	0.6 - 6	6 - 20	> 20	NBG, 1985
JOINTING	Joint spacing	S	m	< 0.06	0.06 – 0.2	0.2 – 0.6	0.6 – 2	> 2	ISRM
	Joint persistence (length)	l	m	< 1	1 – 3	3 – 10	10 – 20	> 20	Bieniawski, 1984
	Joint separation	d	mm	< 0.1	0.1 – 0.5	0.5 – 2.5	2.5 - 10	> 10	Bieniawski, 1984
	Angle of friction for joint surfaces	ϕ	degree	< 15^o	15 – 25^o	25 – 35^o	35 – 45^o	> 45^o	ISRM
	Volumetric joint count	Jv	joints/m^3	< 1	1 – 3	3 – 10	10 – 30	> 30	Palmström, 1982
	Block volume	Vb	m^3	< 0.001	0.001-0.03	0.03 – 1	1 – 30	> 30	Palmström, 1996
	RQD-value	RQD	%	< 25	25 – 50	50 – 75	75 – 90	90 – 100	ISRM
	Block shape factor	β	-	27 - 32	32 - 50	50 100	100 - 500	> 500	Palmström, 1995
ROCK MASSES	Weakness zone thickness	Tz	m	< 1	1 – 3	3 – 10	10 – 30	> 30	NBG, 1985
	Rock mass strength (approx.)	RMi	MPa	< 0.01	0.01 – 0.1	0.1 - 1	1 - 10	> 10	Palmström, 1995
	Rock mass quality	Q	-	< 0.1	0.1 - 1	1 – 10	10 – 40	> 40	Barton et al, 1974
		RMR	-	< 25	25 - 50	50 – 70	70 - 90	90 - 100	Bieniawski, 1974
	Stand-up time in tunnels	-	day	1/24 - 1	1 - 30	30 – 360			NBG, 1985
			year			1/12 - 1	1 - 10	> 10	
	Cost for rock support in tunnels	-	% *	< 5	5 - 15	15 - 50	50 - 200	> 200	NBG, 1985
	Rock stress ratio	σ_c/σ_1	-	< 5	5 – 10	10 – 20	20 – 200	> 200	NBG, 1985
	Seismic velocity in rock masses	v	km/s	< 2.5	2.5 – 3.5	3.5 – 5	5 - 7	> 7	NBG, 1985
	* % of excavation cost (drilling, blasting, mucking)								

Table 13.1 Classification of various ground features. *Continued.*

	PARAMETER	SYMBOL	UNIT	CLASSIFICATION					REFERENCE
				very low or small	low or small	medium // moderate	high or large	very high or large	
CLAY GOUGE	Free swelling (of swelling clay)	S_s	% *		< 100	100 – 140	140 – 200	> 200	NBG, 1985
	Hygroscopic moisture	-	% *		< 8	8 – 15	15 –25	> 25	NBG, 1985
	Swelling pressure	σ_s	MPa		< 0.1	0.1 – 0.3	0.3 – 0.75	> 0.75	NBG, 1985
GROUND WATER	Capacity of drilled well	q_w	l/h	< 3.6	3.6 – 36	36 – 360	360 – 3600	> 3600	NBG, 1985
	Leakage into tunnels // caverns	q_l	l/min/m	< 0.05	0.05 – 0.3	0.3 – 2	2 – 10	> 10	NBG, 1985
	Water leakage test in bore hole	L	Lugeon	< 0.1	0.1 - 1	1 - 10	10 - 100	> 100	NBG, 1985

* % of dry material;

12.2 Graphical symbols

To avoid confusion and misunderstandings, consistency in the use of graphical and technical symbols is very important. Based on established practice and existing national as well as international guidelines, recommendations on the use of symbols are given in the following.

SOILS

grey	Fill / Made ground
violet	Boulders and cobbles
*	Gravel
yellow	Sand
yellow	Silt
blue	Clay
brown	Peat
yellow	Silty sand

SEDIMENTARY ROCKS

yellow	Chalk
blue	Marl
blue	Limestone
blue	Dolomite
*	Conglomerate
*	Breccia
yellow	Sandstone
yellow	Siltstone
yellow	Mudstone
green	Greywacke
yellow	Arkose
yellow	Shale
brown	Pyroclastic (volcanic ash)
*	Gypsum Rocksalt etc
grey	Alum schist

METAMORPHIC ROCKS

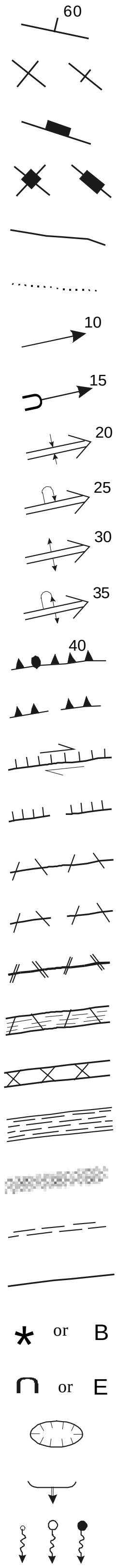
orange	Gneis, migmatite, granulite
brown	Greenstone, greenschist
green	Phyllite, mica schist
brown	Amphibolite
blue	Marble
yellow	Quartzite, arkosite
violet	Serpentinite, talc schist, soapstone

IGNEOUS ROCKS

red	Acid rocks (granite / rhyolite; granodiorite / dacite)
red	Intermediate rocks (syenite / trachyte; monzonite) (diorite / andesite)
brown	Basic rocks (gabbro / dolerite/ basalt) (anorthosite)
violet	Ultrabasic rocks (dunite, peridotite / ultrabasic lava)
*	Lava
*	Tuff

* = colour or pattern depends on the composition or type of material

STRUCTURES ETC.



Strike and dip (60^g) of rock, (bedding, foliation, schistocity, etc)

Horizontal / Vertical

Strike and dip of discontinuities (joints, faults, etc)

Horizontal / Vertical

Rock boundary

Rock boundary, assumed location

Lineation with plunge (10^g)

Fold axis with plunge (15^g)

Synform with plunge (20^g)

Overfolded synform with plunge (25^g)

Antiform with plunge (30^g)

Overfolded antiform with plunge (35^g)

Thust zone with dip (40^g)

Thrust zone, assumed location

Fault with direction of movement

Fault, assumed location

Weakness zone (crushed zone, weak layers, etc.)

Weakness zone, assumed location

Large / wide weakness zone

Very large weakness zone (> 10 m wide)

Crushed zone with given thickness

Dense, parallel jointing

Area with dense jointing

Joint zone

thickness of line may indicate size /width of zone

for maps in large scale

Single joint, seam or shear

Rock burst, spalling

Exfoliation

Karst feature

Unstable rock mass or block

Water leakage (minor, moderate, large)

INVESTIGATIONS ETC.

Photo (No 4)

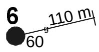
○ Sounding

✡ Rock control drilling

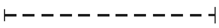
✡ 12.8 / -5.7 18.5 — Rock control drilling with: surface elevation (12.8) - above line)
drilled depth in soil (18.5) - end of line
rock elevation (-5.7) - below line

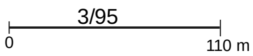
⊠ Test pit

4 ◉ Sample location with sample number

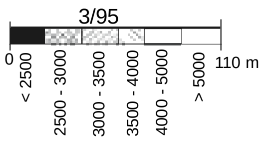


Core hole (No 6) with hole length (110 m), dip (60^g) and direction


Seismic profile (planned)



Measured seismic profile (profile No 3 in 1995) with measured length



Seismic profile with given velocities (in m/s)


Ground surface


Rock surface

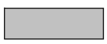

Ground wate table

ROCK SUPPORT

x Rock bolts

x—x—x Rock bolts and steel straps

ⓧ Anchors or tendons


Unreinforced shotcrete


Net reinforced shotcrete


Fibre reinforced shotcrete (fibrecrete)


Rib of shotcrete

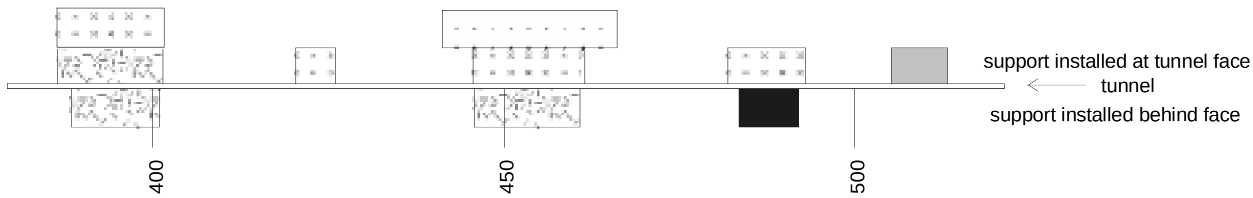

Concrete lining

● Grout hole

○ Drainage hole


Assumed area/volume grouted

Example of rock support presented in small scale



Some suggested abbreviations for joint filling (upper case) and joint coating (lower case):

CA, ca	= calcite	MI, mi	= mica	SM, sm	= smectite
CL, cl	= clay	MO, mo	= montmorillonite	SS, ss	= slickenside
CHL, chl	= chlorite	ru	= rust	SWC, swc	= swelling clay
EP, ep	= epidote	QZ, qz	= quartz	TA, ta	= talc
FE, fe	= feldspar	SE, se	= serpentine	ZE, ze	= zeolite

12.3 Technical symbols

This manual covers a number of topics that are very wide and it therefore contains symbols representing a great variety of different properties and features. As a consequence, the same symbol may sometimes have different meanings in different chapters of the book. In the following, all symbols and their meanings are listed alphabetically. Reference is also made to the list of SI symbols (see Section 14.1).

12.3.1 Roman letter symbols, upper case

A area
A amplitude
A roof factor (in Hoek-Brown tangential stress calculations)
AV, AVS abrasion value (hard metal/steel)
B wall factor (in Hoek-Brown tangential stress calculations)
BWI bit wear index
C gravity adjustment factor for tunnel roof and walls (in the RMi system)
C bolt capacity
CF continuity factor for the rock mass (in the RMi system)
Cg competency factor for continuous ground (in the RMi system)
CLI cutter life index
Co orientation factor for joints, seams and zones (in the RMi system)
D sample diameter
D_{50} sample with 50 mm diameter
Db block diameter
De equivalent sample diameter
DRI drilling rate index
Dt diameter (span width) of tunnel or cavern
E Young's modulus
Em deformation modulus of rock masses
ESR excavation support ratio (in the Q system)
F force/load
F factor of safety
F_b, F_d, F_k, F_m, F_t correction factors in the Norwegian blast vibration criterion
G geometry factor (in water flow analysis)
G shear modulus (modulus of ridgity)

Gc	ground condition factor for discontinuous ground (in the RMi system)
H	height, overburden
H_c	overpressure, constant load
Id2	slake-durability index
Is	point load strength index
Is50	point load strength measured on standard 50 mm thick sample
Ja	factor for joint alteration and filling (in the Q system
JCS	the joint wall compressive strength
Jn	factor for joint set number (in the RMi system)
JP	jointing parameter (in the RMi system
Jr	factor for joint roughness (in the Q system
jR	factor for joint roughness (in the RMi system)
JRC	joint roughness coefficient
Jv	volumetric joint count
Jw	factor for joint water pressure or inflow (in the Q system)
K	permeability (specific permeability)
K	boundary load in numerical modelling
K_s	peak shear stiffness
L	length
L	Lugeon value
Lb	bolt length
L_{max}	maximum block dimension
L_{min}	minimum block dimension
M_d	dimensioning strength
M_k	characteristic strength
Nl	1-D joint frequency, i.e., the number of joints intersecting a defined length
P_{arch}	rock load on tunnel roof/arch
P	measured load at failure
Q	flow rate
Q	charge per delay interval
Q	value in the Q classification system
RMi	value in the rock mass index system
RMR	value in the rock mass rating (or Geomechanics) classification system
RQD	rock quality designation
RSR	rock structure rating system
S	joint spacing
S20	brittleness value
Sb	density of bolts (bolt spacing)
SJ	Siever's J-value
SL	stress level factor used for discontinuous ground (in the RMi system)
SRF	stress reduction factor (in the Q system)
Sr	size ratio (in the RMi system)
T	drill hole rupture strength of rock
Tz	the width (thickness) of weakness zone (in the RMi system)
U	acting force due to (resultant) water pressure
V	volume
Vb	block volume (in the RMi system)
Vf	longitudinal seismic field velocity
Vl	longitudinal seismic laboratory velocity

V_0 basic seismic (laboratory) velocity for intact rock
Vn maximum or 'natural' velocity in crack- and joint-free rock
W weight
Wt wall height of tunnel or cavern

12.3.2 Roman letter symbols, lower case

a attraction in soil
a constant in the modified Hoek-Brown failure criterion (1992)
a particle acceleration (m/s^2)
a_s seismic acceleration
c particle velocity (mm/s)
c_o uncorrected peak particle velocity for the actual seismic velocity (mm/s)
c cohesive strength
c_i' instantaneous cohesive strength
c_m cohesion of rock mass
c_u undrained shear strength
d diameter (in mm) of the actual specimen
d_{50} diameter of 50 mm cylindrical samples
e joint opening (aperture)
f frequency
f_A rock anisotropy factor
f_i factor for the angle between a joint and the observation plane or borehole
f_W rock weathering and alteration factor
g acceleration of gravity (9.81 m/s^2)
i dilation angle for a joint plane
i hydraulic gradient
jA factor for joint alteration and filling (in the RMi system)
jC joint condition factor (in the RMi system)
jL factor for joint length and continuity (in the RMi system)
js joint smoothness factor (in the RMi system)
jw joint waviness factor (in the RMi system)
k ratio between horizontal and vertical stress (σ_h / σ_v)
k correlation factor between compressive and point load strength ($k = \sigma_c / Is$)
k rock constant in calculation of vibrations from blasting
k hydraulic conductivity (coefficient of permeability)
k_{50} correlation factor related to 50 mm thick samples ($k_{50} = \sigma_{c50}/Is_{50}$)
kl correlation factor from 1-D frequency measurement to block volume
ks factor representing in situ conditions in seismic velocity assessments
m mass
m undisturbed material constant in the original Hoek-Brown failure criterion
m_b constant in the modified Hoek-Brown failure criterion (1992)
m_i material constants in the Hoek-Brown failure criterion for intact rock
m_r material constants in the Hoek-Brown failure criterion for broken rock mass
n porosity
nj the number of joint sets (in the RMi system)
p water pressure
p_f, p_r, p_s fracture initiation, re-opening, and shut-in pressure
p_0 initial pore water pressure.
p_i tunnel support pressure

q	pumping rate
r	radius
r_i	tunnel radius
r_e	broken zone radius (used in ground reaction curves)
s	undisturbed material constant in the original Hoek-Brown failure criterion
s	disturbed material constant in the original Hoek-Brown failure criterion
s_r	material constants in the Hoek-Brown failure criterion for broken rock mass
t	thickness of shotcrete
u	undulation factor of a joint plane
u	water pressure
u_i	tunnel deformation
v	flow velocity
v	seismic velocity
w	water content, dry weight basis
wJd	weighted joint density

12.3.3 Greek letter symbols

β	block shape factor (in the RMi system)
γ	specific gravity
γ_f	partial factor for action
γ_m	material factor
δ	slip magnitude
δ	the angle between the observation plane or drill core and the individual joint
ε	strain
λ	wave length
μ	friction coefficient (= tan ϕ)
μ	dynamic viscosity
ν	Poisson's ratio
ν	cinematic viscosity
ρ	density
σ'	effective stress
σ_o	initial stress
σ_c	uniaxial compressive strength of intact rock material
σ_{c50}	uniaxial compressive strength for 50 mm diameter sample size
σ_{cf}	strength of large samples / blocks
σ_{cm}	the compressive strength the rock mass,
σ_{cz}	compressive strength of rock material in weakness zone (in the RMi system)
σ_x, σ_y, σ_z	stress in horizontal (x/y) and vertical (z) directions
σ_1, σ_2, σ_3	principal stresses; $\sigma_1 > \sigma_2 > \sigma_3$
σ_h	horizontal stress
σ_n	normal stress
σ_r	radial stress around underground openings
σ_z or σ_v	vertical stress
σ_θ	tangential stress around underground openings
τ	shear strength
τ_p	peak shear strength
ϕ, φ	friction angle

ϕ_m	friction angle for rock masses
φ_a	active friction angle
ϕ_b	basic friction angle
φ_i'	instantaneous friction angle
ψ_p	inclination of potential failure plane

12.4 Technical Units

12.4.1 The SI units

The SI SYSTEM or International system of units is derived from the metric system of units. It was adopted by the 11th General Conference on Weights and Measures in 1960, and is abbreviated SI in all languages. Its seven basic units, from which other units are derived, are given in the table below.

Dimension		Symbol	SI unit		Comment
			Name	Unit	
Basic units in SI system	Length	l	Metre	m	
	Mass	m	Kilogram	kg	
	Time	t	Second	s	
	Electric current	I	Ampere	A	
	Temperature	T	Kelvin	K	
	Luminous intensity	Iv	Candela	cd	
	Molar heat capacity	n	Mol	mol	
	Angle	a	Radian	rad	Additional unit
Diverted SI units	Frequency	f	Hertz	Hz	s^{-1}
	Force	F	Newton	N	$1 N = 1 kgm/s^2$
	Pressure, stress	p, s,	Pascal	Pa	$1 Pa = 1 N/m^2$
	Energy, work, heat	E, W, Q	Joule	J	$1 J = 1 N \times m$
	Effect	P	Watt	W	$1 W = 1 J/s$

12.4.2 Decade prefixes

Prefix	Symbol	Value
Tera	T	10^{12}
Giga	G	10^9
Mega	M	10^6
Kilo	K	10^3
Hecto	H	10^2
Deca	Da	10^1

Prefix	Symbol	Value
Deci	d	10^{-1}
Centi	c	10^{-2}
Milli	m	10^{-3}
Micro	µ	10^{-6}
Nano	n	10^{-9}
Pico	p	10^{-12}

12.4.3 The Greek alphabet

Α	α, ∝ *)	alpha	(a)	Ι	ι	iota	(i)	Ρ	ρ	rho	(r, rh)
Β	β	beta	(b)	Κ	κ	kappa	(k)	Σ	σ	sigma	(s)
Γ	γ	gamma	(g, n)	Λ	λ	lambda	(l)	Τ	τ	tau	(t)
Δ	δ, ∂ *)	delta	(d)	Μ	μ	mu	(m)	Υ	υ	upsilon	(y, u)
Ε	ε	epsilon	(e)	Ν	ν	nu	(n)	Φ	ϕ, φ *)	phi	(ph)
Ζ	ζ	zeta	(z)	Ξ	ξ	xi	(x)	Χ	χ	chi	(ch, h)
Η	η	ëta	(e, ë)	Ο	ο	omicron	(o)	Ψ	ψ	psi	(ps)
Θ	θ, ϑ *)	theta	(th)	Π	π	pi	(p)	Ω	ω	omega	(o, õ)

*) Old-style character

12.5 Various units and conversions

Dimension	Name	Symbol	Unit	Conversion
Time	minute	min	s	1 min = 60 s
	hour	h	h	1 h = 60 min
	day	day	day	1 day = 24 h
	week	wk	wk	1 wk = 7 days
	year	yr	yr	1 yr = 365.242 days = 52.17 wk
Length	Angstrom	A	m	1 A = 10^{-10} m
	nautical mile		n mile	1 n mile = 1852 m
	wave length	λ	m	
Force	load	G (P,W)	N	1 kp = 9.807 N
	gravity	g	N	1 g_n = 9.80665 m/s^2 (exact, at 45^o N; at 60^{o}N g = 9.82 m/s^2)
	atmosphere		atm	1 atm
Modulus	elasticity modulus	E	Pa	1 kPa = 0.1 N/cm = 0.102 ton/m^2
	deformation modulus	G	Pa	1 MPa = 10,2 kg/cm^2
	shear modulus	K	Pa	
Density	mass density	ρ	kg/m^3	
	force density	γ	kN/m^3	
Flow	Lugeon	Lugeon	l/min/m	1 Lugeon = approx. 10^{-7} m/s (see definition)
	permeabilty coefficient	k	m/s	
	Darcy		m/s	1 Darcy = 1 cm^3/s through 1 cm^2 at 1 atm and viscosity η = 0.01 poise
Viscosity	dynamic viscosity	η, μ	poise	1 poise = 0.1 Pa × s
	kinematic viscosity	ν	m^2/s	
Angle	degree	^{o}		1^o = π/180 rad = 0.017453 rad; 1 rad = 57.296^o = 63.622^g
	new degree (gon)	^{g}		1^g = π/200 rad = 0.015708 rad
Energy	calory		cal	1 cal = 4.1868 J = 4.1868 x 10^7 erg
	erg		erg	1 erg = 10^{-7} J
	kilowatthour		kWh	1 kWh = 3.6 x 10^6J = 3.6 MJ
Heat	heat capacity	C	J/K	
	specific heat capacity	c	J/(K × kg)	
	thermal conductivity	l	W/K × m^2	
Pressure	atmosphere (normal)		atm	1 atm = 760 mm Hg = 1.013 bar = 1.033 kg/m² = 0.101325 MPa
	bar		bar	1 bar = 10^5 Pa
Others	moment of force	M	N × m	
	strain	ε, e	none	1 ε = $\Delta l/l_o$
	Poisson's number	ν, μ	none	
	efficiency	η	none	
	factor of safety	FS	none	
	angle		rad	1 rad = 57.296^o = 63.622^g
	horsepower	hp		1 hp = 0.7457 kW

12.5.1 English and American units and their conversions

Dimension	Name	Unit	Conversions	Special for USA
Length	inch	in	1 in = 25.4 mm	
	foot	ft	1 ft = 12 in = 0.3048 m	
	yard	yd	1 yd = 3 ft = 0.9144 m	
	mile	mi	1 mi = 1609 m	
	league	league	1 league = 0.3333 n mile = 617.3 m	
	link	link	1 link = 20.12 cm	
	chain	chain	1 chain = 20.12 m	
	fathom	fathom	1 fathom = 6 ft = 1.829 m	
	hand	hand	1 hand = 4 in = 10.16 m	
Area	perch	perch	1 perch = 25.29 m^2	
	are	are	1 are = 100 m²	
	rood	rood	1 rood = 40 perch = 10.12 are = 1012 m²	
	acre	acre	1 acre = 4 rood = 10 $chain^2$ = 4840 yd^2 = 4047 m^2	
Weight	grain	grain	1 grain = 64.8 mg	
	carat	carat	1 carat (metric) = 0.200 g	
			1 carat (1877) = 3.168 grain = 0.2053 g	
	pound	lb	1 lb = 0.4536 kg	
	ounce	oz	1 oz = 28.35 g	
	(long) ton	ton	1 ton = 1016,05 kg (= 2240 lb)	
	short ton	sh cwt		1 sh cwt = 907.185 kg (=2000 lb)
	dram	dram	1 dram = 1.772 g	
	kip	kip	1 kip = 1000 lb = 453.6 kg	
	slug	slug	1 slug = 32.17 lb = 14.59 kg	
			1 slug (metric) = 0.98 kg	
Volume	fluid ounce	fl.oz	1 fl.oz = 28.412 ml	1 fl.oz = 29.573 ml
	gill	gill	1 gill = 0.142 l	1 gill = 0.1182 l
	pint	pt	1 pt (UK) = 4 gill = 0.568 l	1 pt (liq)(US) = 0.473 l
				1 pt (dry) = 0.473 l
	quart	qt	1 qt (UK) = 2 pt = 1.136 l	1 qt (US) = 2 pt = 0.946 l
				1 qt (dry) = 1.101 l
	gallon	gal	1 gal (UK) = 4 qt = 4.546 l	1 gal (US) = 4 qt = 3.785 l
	peck	pk	1 pk = 2 gal (UK) = 9.092 l	1 pk = 8 qt = 8.81 l
	bushel	bu	1 bu (UK) = 8 gal (UK) = 36.37 l	1 bu = 4 pk = 35.24 l
	barrel 1)	bbl	bbl (UK) = 36 gal (UK) = 163.7 l	1 bbl (US) = 42 gal (US) = 159 l 2)
			1 bbl (dry) = 115.6 l	1 bbl (dry) = 115.6 l
	quarter	quarter	1 quarter = 8 bu = 290.9 l	
Velocity	mile per hour	mile/h	1 mile/h = 0.447 m/s	
	knot	kn	1 kn = 0.514 m/s	
Flow velocity	permeability	ft/year	1 ft/year = 9.659 × 10^{-9} m/s	
	coeffisient	ft^3/s	1 ft^3/s = 0.02832 m^3/s	
Pressure, stress	pound-force per $inch^2$	psi	1 psi = 6.895 kPa	
	dyne	dyne	1 dyne = 10^{-5} N	
	atmosphere	atm	1 atm = 760 mm Hg = 1.013 bar = 1.470 psi = 0.101325 MPa	
	bar	bar	1 bar = 14.495 psi	
Effect	British thermal unit	Btu	1 Btu = 2.52 x 10^2 cal = 1055 J	
	British horsepower	hp	1 hp = 745.7 kW (= 550 ft × lbf/s)	
Heat	degree Fahrenheit	^oF	$^oF = {}^oC \times 9/5 + 32$	
	degree Celcius	^oC	$^oC = (^oF - 32) \times 5/9 =$ K - 273.16	

1) unit of both liquid and dry measure in UK and US Customary systems ranging from 31 to 42 gallons

2) for petroleum products

13 Literature List

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14 Glossary

This glossary is intended to provide the user with a guide to unfamiliar terms that may be encountered in geological and engineering geological literature. The main emphasis here has been on terms that frequently cause difficulties and a major part of the glossary is therefore related to geological terminology. Since this guide is intended for a rather specific group of people, many commonly used words that would apply in a complete glossary of engineering geology and rock mechanics are not included here.

Many of the following definitions are based on Geoguide 3 (1988), issued by the Geotechnical Engineering Office, Hong Kong. Others have been taken from previous editions of NBG[5] handbooks, textbooks such as Legget and Karrow's (1983), dictionaries such as Webster's, or are the authors' own.

NOTE: *Terms that are explained in the main text and can be found in the Index, are usually not included among the definitions of this section.*

abutment — The point in a tunnel or cavern where the wall and roof (back) meet.

access tunnel — Tunnel from surface to underground work site.

accessory — A term applied to minerals occurring in small quantities in a rock, and whose presence or absence does not affect its mechanical properties.

acid — Chemical term for an igneous rock containing more than 62% silica and usually less than 20% dark minerals. (Contrast with "intermediate" and "basic").

actinolite — Mineral. A monoclinic variety of amphibole.

activity (of a soil) — Ratio of plasticity index to clay fraction.

adit — Tunnel (gallery) driven from a surface portal to an underground working place. Used for drainage, ventilation, transportation, or giving access to an ore body or working place.

aegirine — Mineral. A variety of monoclinic pyroxene.

aeolian — A term applied to deposits whose constituents have been carried by, and laid down from, the wind.

agglomerate — Pyroclastic rock composed predominantly of rounded bombs of material of greater than 60mm average dimension. (Contrast with "pyroclastic breccia").

aggregate — 1) A mixture of different materials which may be separated mechanically.

[5] Norsk Bergmekanikkgruppe (Norwegian Rock Mechanics Group)

2) Mineralogic material (i.e., sand gravel, crushed rock etc) which may be mixed with cement, bitumen or epoxy to form concrete or asphalt.

aggregate abrasion value A measure of the resistance of an aggregate to abrasion; the lower value, the more resistant to abrasion is the aggregate.

aggregate soundness test A chemical test to measure the relative resistance of rocks to repeated wetting and drying and crystallisation of salts within pores.

albite Mineral. The most sodium rich variety of pyroxene feldspar ($NaAlSi_3O_8$).

alkali feldspar Group of feldspars composed of mixtures, or mixed crystals, of potassium feldspar ($KAlSi_3O_3$) and sodium feldspar ($NaAlSi_3O_8$). (See "feldspar").

allochthonous rocks Rocks having been thrusted in connection with mountain range formation.

alluvium Detrital material of any grain size transported and deposited during comparatively recent geological time by a river or stream.

alteration Change of the mineralogical composition of a rock, typically brought about by the action of hydrothermal solutions. The term joint alteration includes both weathering and alteration. It is included in the description of joint alteration in the following main categories:
- the alteration of the rock in the joint wall/surface;
- the coating on the joint surface when it occurs; and
- the filling in joints with separation.

alum schist A black shale variety containing pyrrhotite and carbon.

amorphous Term for a mineral or other substance that lacks crystalline structure and has no characteristic external form.

amphiboles A group of rock-forming minerals.

amphibolite Homogeneous to slightly schistose, dark, metamorphic rock with 20-50% plagioclase, 20-80% amphibole, 0-50% mica, minor amounts of quartz, garnet and epidote.

amphibolitic gneiss Gneiss with > 20% amphibole.

andesite A fine-grained igneous rock formed essentially of plagioclase, together with the mafic minerals biotite, hornblende, and pyroxene (the effusive rock of diorite).

angular Shape term for a rock particle with sharp edges and corners.

anhydrite White or light-coloured mineral consisting of calcium sulphate ($CaSO_4$). Essentially a slightly harder and less soluble form of gypsum.

anisotropy Characterises a material which has different properties in different directions.

anorthite Mineral. The most calcium rich variety of plagioclase feldspar

anorthosite A plutonic rock composed almost entirely of plagioclase.

anthophyllite Mineral. A variety of rhombic amphibole.

anticline Fold in the form of an arch whose core contains the stratigraphically older rocks.

antiform Fold with the convex side up (may be inverted, as opposed to anticline).

aperture Perpendicular distance between adjacent rock walls of a discontinuity, in which the intervening space is air or water filled.

aphanitic Textural term for a rock in which the individual constituents are not visible to the naked eye.

aplite	Light-coloured, fine-grained igneous rock of granitic composition. Very uniform and smooth-textured appearance. Commonly occurs in the form of narrow dykes.
aquiclude	A (relatively) impermeable barrier to groundwater flow.
aquifer	Permeable layer of water-bearing ground.
arenaceous	Applied to rocks that have been derived from sand or that contain sand.
arenite	Consolidated rock of the texture of sand, irrespective of composition; see psammite.
arenite	Arenaceous sedimentary rock containing less than 15% silt and clay material. (Contrast with "wacke").
argillaceous	Applied to all rocks composed of clay or having notable portion of clay in their composition, as roofing slate, shale, etc.
arkose	Sedimentary rock. Sandstone with 0-75% quartz, 10-90% feldspar and rock fragments, 0-20% clay minerals.
arkosite	Schistose to slightly schistose, metamorphic rock with 0-70% quartz, 10-90% feldspar, 0-30% mica and chlorite. (Metamorphic variant of arkose.)
artesian	Relating to a confined aquifer (usually one for which the pressure surface is above ground level)
ash	Pyroclastic rock material of sand-, silt- and clay-size (i.e., < 2mm), subdivided into coarse ash for sand-size and fine ash for silt- and clay-size. Descriptive term for tuff composed wholly or predominantly of these grain sized.
astenosphere	The part of the Earth's mantle beneath the lithosphere.
augen gneiss	Gneiss with feldspar appearing as eyes in the bedrock.
augite	Dark-coloured silicate mineral in the pyroxene group often found in basic igneous rocks.
aureole	Zone surrounding an igneous intrusion in which the country rock shows the effects of thermal or contact metamorphism.
authigenic	Formed where found (used of mineral particles of rocks formed by crystallisation in the place they occupy)
autochtonous rocks	Rocks not having been thrusted in connection with mountain range formation.
axial plane	Plane that connects the points of maximum curvature of the bedding planes or other structural rock surfaces in a fold.
banded	Structural term for a rock with alternating layers of material of differing colour or texture, possibly of differing mineral composition also.
bar block	See columnar block
basal	Pertaining to, situated at, or forming the base of a geological structure. "Basal layer" refers to the lowest layer in a layered rock or soil.
basalt	Dark coloured, fine-grained igneous rock composed mainly of plagioclase feldspar and mafic minerals. Often occurs in the form of lave flows. (The fine-grained equivalent of gabbro.)
basic	Chemical term for an igneous rock containing 44 to 54% silica and usually more than 30% dark minerals. (Contrast with "acid" and "intermediate").
batholith	A body of plutonic igneous rock, usually greater than 80 km^2 (sometimes many thousand km^2 in surface area)

bauxite	Residual soil layer relatively rich in aluminium oxide, which is formed in tropical area of high rainfall.
bedded	Structural term for a sedimentary rock or superficial deposit formed, arranged or deposited in layers or beds > 20 mm thick.
bedrock	Solid rock underlying soil, sand and other unconsolidated material.
bentonite	An expansive clay formed from the decomposition of volcanic ash.
biotite	Black, dark brown or dark green mineral of the mica group. Forms distinctive shiny thin prisms or flakes. Very common in crystalline igneous and metamorphic rocks.
black cotton soil	A dark clay soil which shrinks and cracks in dry weather.
black shale	Sedimentary or slightly metamorphic, schistose rock containing graphite.
block.	1) Rock fragment derived from the sides of a volcanic vent. Commonly angular or subangular. Restricted to pyroclasts > 60 mm diameter. 2) General term for individual pieces of rock bounded by discontinuities in a rock mass.
block angles	The angles between the sides of the block (orthogonal, oblique, acute, rhomboedric, etc.).
blocky	Shape term for a rock mass with three approximately orthogonal and equally-spaced joint sets, such that individual rock blocks tend to be roughly equidimensional.
blocky blocks	Approximately equidimensional blocks.
bomb	Partly molten material from a volcanic vent which solidifies in flight or shortly after landing. Restricted to pyroclasts > 60mm diameter.
borrow pit	An excavation to provide material usually for fill elsewhere / A test pit
boulder clay	Local British term for till; considered inaccurate because neither boulders nor clay are essential constituents.
boulders	Rock fragments greater than 200mm in size.
break	A general term used in mining geology for any discontinuity in the rock, such as fault, fracture, or a small cavity.
breccia	Coarse-grained rock composed of angular broken rock fragments held together by a mineral cement or in a fine-grained matrix. (Contrast with "conglomerate"). May be of sedimentary or pyroclastic origin, or may be formed by crushing of any type of rock in a fault zone.
brittle failure	Failure characterised by sudden loss of strength.
brittleness	1) A material condition characterised by reduced ability to carry load as the strain increases. 2) The parameter resulting from the brittleness test.
calcrete	Calcium carbonate concentrated by water movement and precipitated into a hard cement of soil matrix.
caliche	See calcrete
cataclastic	Term for the structure of a rock which has been broken up severely by strong dynamic metamorphism or faulting. Common features are bent, broken or ground-up minerals. "Cataclasite" is the name for any rock showing cataclastic structure.
calcareous	Term applied to a rock containing an appreciable amount of calcium carbonate, e.g., calcareous sandstone.

calcite — White, light grey, yellow or blue, common carbonate mineral: the carbonate of calcium ($CaCO_3$). Glassy appearance. Effervesces in hydrochloric acid. The principal constituent of chalk and most limestones.

Cambrian-Silurian — Abbreviation for Cambrian, Ordovician and Silurian. Commonly used also for rocks of that age.

carbonate — Term applied to a mineral compound characterised by an ionic structure of CO_3^{2-}. Calcite and dolomite are examples of carbonate minerals. Also applied to a rock consisting chiefly of carbonate minerals. Limestone and dolomite are examples of carbonate rocks. (See also "calcareous").

cataclastic — Texture with the original mineral grains crushed and/or deformed , and often occurring as bands or stripes.

cemented — Term for a sedimentary rock whose grains are bound together in a coherent mass by mineral cements. Most cements are chemically precipitated. The most common cement are iron oxides, silica (quartz, opal, chalcedony), carbonates (calcite, dolomite) and clay minerals.

chalk — A soft, usually white, fine-textured limestone

charnockite — Massive, igneous or metamorphic rock. Rich in feldspar and with some pyroxene.

chert — Hard, dense, dull to slightly shiny, cryptocrystalline sedimentary rock consisting of organic or inorganic precipitates of silica. Occurs commonly as small irregular lumps in limestones and dolomites, but may also form extensive bedded deposits.

chlorite — Group of platy micaceous minerals, usually green in colour and containing much ferrous iron. Often associated with and resembling biotite; crystals cleave into small thin flakes. Widely distributed in low-grade metamorphic rocks, or found as alteration products of ferromagnesian minerals in any rock type.

chroma — Brilliance or intensity of a colour.

clastic — Term for a rock composed of broken fragments that are derived from pre-existing rocks or minerals and that have been transported from their places of origin.

clay — A fine-grained unconsolidated material which has the characteristic property of being plastic when wet and which loses its plasticity and retains its shape upon drying or when heated.

clay fraction — The fraction by weight of particles of size less than 0.002 mm effective spherical diameter; mineral composition is variable.

clay minerals — A group of alumino-silicate minerals with characteristic sheet structure. Characterised by small particle size and the ability to absorb large quantities of water and of exchangeable cations.

claystone — Sedimentary rock composed predominantly of clay-size particles. Texture and composition similar to shale, but lacks fine lamination or fissility. (See also "mudstone").

clean joint — Joint surface without filling or coating.

cleavage — Property or tendency of a rock to split easily along aligned, usually closely-spaced fractures produced by a metamorphism or deformation. Cleavage planes are secondary features and may differ in spacing and orientation from primary rock structures such as bedding.

cobbles — Rock fragments 60 to 200 mm in size.

coherent — Descriptive of two or more similar parts or organs of the same series touching one another more or less adhesively but not fused.

cohesive — Term for a soil which possesses cohesion. (Contrast with "granular").

colluvium — Deposits formed by the downslope movement of earth materials essentially under the action of gravity.

columnar blocks — Long or bar block with one dimension considerably larger than the other two.

competent rock — Rock which is stronger than adjacent rock and relatively less liable to deformation. It therefore tends to fracture when the load exceeds a certain limit.

competent ground — Rock mass strength is higher than the ground stresses imposed, see incompetent ground

conchoidal — A term used originally in descriptive mineralogy to describe the shell-like surface produced by the fracture of a brittle substance.

concordant — Describes various strata with bedding planes more or less parallel and probably conformable.

concretions — Nodular or irregular concentrations of certain authigenetic constituents in sedimentary rocks and tuffs.

conformable — Describes sedimentary rocks succeeding one another without signs of intervening tectonism or erosion.

conformal — Parallel beds/layers in unbroken order.

conglomerate — Coarse-grained sedimentary rock composed of rounded to subangular fragments larger than 2mm average dimension set in a sand or finer-grained matrix which is often cemented. (Contrast with "sedimentary breccia").

conjugate joints — Two sets of joints nearly at right angles to one another, produced by the same process.

connate water — Water (often saline) trapped for long periods in rock pore-space, usually beneath the present or a pre-existing sea.

contact metamorphism — Metamorphism due to high temperature close to the contact of intruding magma.

continental crust — Less dense, more siliceous and thicker crust under continents; contrasted with oceanic crust.

cooling joint — Joint formed by the cooling of an igneous, pyroclastic or other heated rock body.

core — 1) Part of the Earth more than 2900 km beneath ocean surface, with a relatively density of more than 10 as interpreted from the records of seismic waves.
2) Cylinder of rock obtained by drilling.

core barrel — A length of pipe next to the cutting bit of a core drill and which contains the core. A double core barrel is a core barrel with two concentric pipes so arranged that in very soft rock the inner tube does not rotate and so damage the core.

core recovery — 1) The percentage of a length of drilling which is represented by solid core samples recovered from the drill.
2) The retrieval and storing of rock cores from a core drilling machine.

corundum — A hard aluminium oxide mineral. Sapphire and ruby are the gem varieties.

country rocks — The rocks surrounding an igneous body.

crack — A small, partial or incomplete defect.

creep — Continuous (very slow)deformation under load.

cross bedding — Structure formed by a series of bedding planes inclined at an angle to the main planes of stratification in a sedimentary deposit. Planes are usually curved and truncated in cross-section by overlapping sets. See current bedding crushed rock Heavily jointed to "sugar cube".

crust Outermost part of the solid Earth, of relative density 3.0 or less, with a maximum thickness of about 50 km.

cryptocrystalline Textural term for a rock consisting of crystals that are too small to be recognised and distinguished separately under an ordinary microscope. (i.e., glass and chert).

cubical blocks Block with six equal square sides.

current bedding Bedding which is formed at an angle to the horizontal by the action of swift local currents of water or air (syn.: cross bedding). Known also as false-bedding because observed inclination cannot be used as an indication of the nature of Earth movements subsequent to deposition.

cut-off drain A drain which collects and discharges surface water across the natural drainage slopes.

dacite Medium-coloured, very fine-grained, acid igneous rock. The fine-grained equivalent of granodiorite. Often contains megacrysts of quartz and feldspar.

dappled. Term for non-uniform colour distribution of a rock or soil where the secondary colour constituent forms irregularly-shaped blotches or marks of widely differing size.

decomposed rock 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.

deformation Alteration of shape and/or volume

denudation The erosion of mountain ranges and flattening of the land surface.

detrital Term for a rock or sediment formed of fragmental material which is derived from older rocks and moved from its place of origin by weathering and erosion.

diabase The dyke rock variant of gabbro (“fine-grained gabbro”). (Equivalent to dolerite).

diapiric Describing an anticline or dome whose rocks have become ruptured by the upward movement of less dense plastic material, i.e., salt.

diastrophism The process or processes by which the crust of the earth is deformed, and by which continents and ocean basins, plateaus and mountains, flexures and folds of strata, and faults are produced.

dilatancy Tendency of a volume increase under increasing shear stress.

diopside Mineral. A variety of monoclinic pyroxene.

diorite A coarse-grained igneous rock. Typically 0-5% quartz, 0-10% alkali feldspar, 30-50% hornblende, biotite, augite, 30-60% plagioclase.

discoloured rock The colour of the original fresh rock material is changed.

discontinuous joints Joints that terminate in solid rock.

discordant Describes relationships of a separate rock body to an organised sequence of rock layers with which the body is in contact.

disintegrated rock 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.

dolerite Dark-coloured, medium-grained, basic igneous rock with the same composition as basalt and gabbro, but with a texture of intergrown plagioclase and pyroxene.

dolomite 1) Mineral. Generally white, sometimes slightly yellow, brown, pink or grey: the carbonate of calcium and magnesium ($CaMg(CO_3)_2$). Forms curved, saddle-like crystals.
2) Sedimentary rock with more than 50% of the mineral dolomite.

dolostone A sedimentary rock composed chiefly of magnesium/calcium carbonate.

dome A dome-shaped structure, usually associated with igneous or other diapiric cause.

drift 1) A horizontal tunnel. A drift follows the vein, as distinguished from a crosscut, which intersect it, or a level or gallery, which may do either.
2) Any rock material, such as boulders till, gravel, sand silt, or clay, transported by a glacier and deposited by or from the ice or by or in water derived from the melting of the ice. Generally used of the glacial deposits of the Pleistocene epoch. Detrital deposits.

ductility A material condition characterised by ability to resist permanent strain without loosing the ability to carry load.

dunite Ultrabasic, metamorphic rock consisting mainly of olivine.

dyke Sheet- or table-like body of intrusive igneous rock which cuts across the bedding or other structural planes of the country rock.

eclogite Highly metamorphic, ultrabasic rock consisting mainly of pyroxene and garnet.

elasticity The capability of a material to regain its original shape or condition after the removal of an applied load.

elastic limit The point on the stress/strain-curve defining the transition from elastic to non-elastic behaviour.

elongate Shape term for a rock particle in which the longest diameter is much greater than the intermediate or shortest diameter. Expressed quantitatively as "flatness ratio" > 0.66 and "elongation ratio" <0.66.

elongation ratio Ratio of the intermediate to longest diameters of a particle.

eluviation Downward movement of soluble or suspended material in a soil or superficial deposit by groundwater percolation.

en echelon jointing Joints in relatively narrow zones, where one joint is replaced by another joint which is slightly off-set. At their ends, joints are sometimes bifurcated and sometimes linked to the adjacent joint in the zone.

endogenetic A term applied to geological processes originating within the earth and to rocks that owe their origin to such processes.

enstatite Mineral. A variety of rhombic pyroxene.

epidote Mineral. Common in metamorphic rocks.

eolian Deposited by wind.

erosion The wearing and removal of soil or rock fragments caused by glaciers, running water and wind.

epigene A general term for geological processes originating at or near the surface of the earth.

epigenetic A term now generally applied to ore deposits of later origin than the rocks among which they occur.

equidimensional blocks Small differences in dimensions, caused by three dominant sets of joints, approximately orthogonal, with occasional irregular joints.

equigranular Textural term for a rock characterised by crystals or grains of the same size or approximately the same size.

essexite Plutonic rock, typically with 10-65% plagioclase, 0-30% alkali feldspar, 5-45% feldspathoids, 30-60% mica, hornblende, augite.

eutaxitic — Structural term for some pyroclastic rocks characterised by a streaked or banded appearance, due to pumice clasts or other material being stretched out whilst still in a hot viscous state, and subsequently preserved by welding.

evaporite — Sedimentary rock consisting of minerals resulting from the evaporation of saline water.

exfoliation — Process by which thin, curvilinear scales or shells or rock are successively spalled or stripped away from the bare surface of a rock mass or boulder under the action of mechanical and/or chemical weathering and release of confining pressure by erosion. Often results in a rounded rock mass. Commonly seen in granite corestones.

exogenetic — A term applied to geological processes originating at or near the surface of the earth and to rocks that owe their origin to such processes.

exposure — A visible rock outcrop.

extrusive — Term for an igneous rock that has been erupted onto the earth's surface (e.g., rocks formed from lave flows). Also applies to all pyroclastic rocks. (Contrast with "intrusive").

failure criterion — Theoretical or empirical stress or strain relationship defining the failure condition of a material.

false bedding/schistosity — See current bedding.

fan — Gently-sloping mass of detrital material deposited at locations of abrupt decrease in slope gradient. Forms a part-cone shape in cross-section and is fan-shaped in plan. Of alluvial or colluvial origin.

feldspar — Group of abundant alumino-silicate rock-forming minerals of general composition $MAl(Al,Si)_3O_8$ where M is commonly potassium, sodium or calcium. Crystals are usually white or nearly white (but frequently coloured by impurities), translucent, and possess good cleavage in two directions intersecting at or near 90^o. They occur commonly in many rock types and decompose readily to clay.

feldsparphyric — Textural term for a rock containing large megacrysts or feldspar, e.g., feldsparphyric rhyolite.

feldspathic — General term for any rock or other mineral aggregate containing feldspar.

feldspathoid — Group of minerals mainly occurring in ultrabasic rocks (most common: nepheline and leucite).

felsic — General term for light-coloured minerals (e.g., quartz, feldspars, muscovite), or an igneous rock composed chiefly of these minerals. (Contrast with "mafic").

felsitic texture — Texture with mineral grains in a fine grained matrix, making distinction between the individual grains difficult.

femic — Term for rocks having a high content of iron and magnesium. Normally dark, basic rocks.

ferralites — Prominently iron-rich residual soils.

ferromagnesian — Term for any rock-forming minerals containing iron or magnesium.

ferruginous — An adjective applied to rocks with a prominent iron content.

fissile — Capable of being readily split along closely spaced planes

fissility — Property possessed by some rocks, such as shale, of splitting easily into thin layers along closely-spaced, approximately planar, parallel surfaces. Its presence distinguishes shale from mudstone.

fissure — Open crack or fracture in a rock or soil mass. Also used to describe a volcanic vent in the form of a crack.

flakiness The ratio between width and thickness of particles as determined in laboratory.

flatness ratio Ratio of the shortest to intermediate diameters of a particle.

flint Dark grey or black variety of chert.

fluorite The mineral calcium fluoride.

flow-banded Structural term for a rock formed by alternating layers of different colour, composition and/or texture as a result of the flow of molten rock. Most common in igneous rocks, but sometimes visible in pyroclastic flow deposits.

foidite Common term for plutonic rocks rich in feldspathoids.

fold axis The line along the ridge of an anticline or the bottom of a syncline.

folding Curving of layers in the bedrock.

flysch The widespread deposits of sandstones, marls, shales and clays (originally used for deposits located in the southern borders of the Alps, later used as a general term)

foliated Structural term for the layered, planar arrangement of the constituent grains of a rock formed by flattening of minerals due to metamorphism.

foot wall The wall beneath an inclined discontinuity or ore body.

formation A term applied stratigraphically to set of strata possessing a common suite of lithological characteristics.

fracture cleavage A capacity to part along closely spaced, parallel surfaces of fracture or near-fracture, commonly in a single set, but occasionally in intersecting sets. It is closely related to joint structure, but the joints are so closely spaced as to give the rock a distinctive structure not ordinarily to be described in terms of joints.

fracture index Ratio of seismic velocity for intact rock samples to seismic velocity of rock mass in situ.

fracture intercept The mean distance between successive fractures as measured along an intersecting straight line at exposed surfaces or in core samples. All fractures are counted, whether or not they belong to the same set.

fragment A rock or mineral particle larger than a grain.

free swelling Laboratory index describing the ability of clay materials to swell.

fresh rock No visible sign of weathering/alteration of the rock material.

friable Term for a soil that crumbles very easily in the hand.

gabbro Dark-coloured, fine- to coarse-grained, basic intrusive igneous rock. with typical composition: 30-70% plagioclase, 30-70% pyroxene, 0-30% olivine, biotite, and hornblende, negligible content of quartz.

garnet A group of hard minerals. Most common varieties: grossular, almandine pyrope and spessartine.

gentle fold Fold with an inter-limb angle between 120^o and 180^o.

geode Sub-spherical hole in a lava, often later filled with crystals.

geognosy The science which treats the solid body of the earth as a whole and of the different occurrences of minerals and rocks of which it is composed and of the origin of these and their relations to one another.

geosyncline A mobile downwarping of the Earth's crust, perhaps 100 km or more across, in which sediments (and often volcanic rocks) are deposited simultaneously with the downwarping.

glacial striation/striae Striation of bedrock caused by the movement of glacier containing rocks and cobbles.

glacio — A combining form frequently used with other words to denote formation by or relationships to glaciers.

glaciofluvial — Pertaining to streams flowing from glaciers or to deposits made by such streams.

glassy — Shape term for a rock particle with a surface texture that looks and feels like glass or quartz. Surface is typically shiny, straight or smoothly curved and lacks distinct crystal shapes.

gneiss — Coarse-grained foliated rock formed by regional metamorphism, in which bands of granular minerals alternate with bands of flattened, elongated minerals showing preferred orientation parallel to the banding.

gneissic granite Granite with slight parallel-orientation of mineral grains due to deformation and metamorphism.

gouge — Fine-grained (silt- and clay-size) material in a fault. Also known as "rock flour".

graben — A trough-shaped parallel-sided valley usually bounded by faults.

graded bedding Structure evident in a bedded sedimentary deposit in which each layer shows a gradual and progressive change in particle size, usually from relatively coarse at the base of the bed to relatively fine at the top (e.g., fine sand grading to clay, cobbles grading to coarse sand).

grading — Particle size distribution, defined as the percentages of the various grain sizes present in a soil as determined by sieving and sedimentation.

granite — Light coloured, fine- to coarse-grained, acid igneous rock. Typical composition15-55% quartz, 15-75% alkali feldspar, 0-50% plagioclase and 5-20% mica (mainly biotite). Commonly forms both major intrusive bodies and minor intrusions such as dykes.

granitic gneiss — gneiss of granitic composition.

granodiorite — Medium-coloured, fine- to coarse-grained, acid igneous rock. Typical composition 15-75% plagioclase feldspar, 15-55% quartz, 0-25% alkali feldspar, 5-40% biotite, hornblende or pyroxene. Typically contains more mafic minerals than granite.

granular — 1) Engineering term for a cohesionless soil, i.e., one which cannot form a coherent mass. (Contrast with "cohesive").

2) Geological term for the texture of a rock that consists of mineral grains of approximately equal size.

granulite — Gneiss with no or little mica. Main minerals: quartz, feldspar, pyroxene and garnet.

graphite — Grey to black, opaque, shiny, six-sided mineral. A naturally-occurring crystalline form of carbon. Common as crystals or thin flakes in veins and in many metamorphic rocks.

graphite schist — Mica schist with >15% graphite.

gravel — Soil particles 2 to 60mm in size.

greenschist — Metamorphic, schistose rock with mineral composition approximately as in greenstone.

greenstone — Metamorphic, homogeneous rock with 50-90% amphibole, 0-50% plagioclase, chlorite, epidote.

greywacke — Arenaceous sedimentary rock containing more than 15% silt and clay. A "dirty" sandstone. (Contrast with "arenite").

grit — 1) Sand especially coarse sand.
2) Coarse-grained sandstone.

.groundmass Relatively fine-grained glassy or crystalline material between the megacrysts in a megacrystic igneous rock. Also known as the "rock matrix".

grout A fluid used for injection (grouting) of rock masses or soils with the purpose of sealing off water. The fluid may be a cement slurry, a mix of cement and sand and other additives, or a mixture of special chemicals.

gunite A patent name for a cement and sand mixture used as shotcrete.

gypsum White or colourless soft mineral composed of hydrous calcium sulphate ($CaSO_4.2H_2O$). The commonest sulphate mineral. Often forms extensive beds of evaporite interstratified with limestone, shale and clay.

haematite A red iron oxide mineral (Fe_2O_3).

hade To deviate from vertical; said of a vein or fault.

halite Evaporite mineral composed of sodium chloride (NaCl). Also known as "rock salt" or "common salt".

halloysite Clay mineral made up of very small hollow tubes, as shown by the electron microscope.

hanging wall The wall rock above an inclined discontinuity or orebody.

healed joint The joint has been "welded" together by filling material such as quartz, calcite, epidote, etc.

hedenbergite Mineral. A variety of monoclinic pyroxene.

hidden layer A layer of ground with low seismic velocity which cannot be detected by seismic refraction (syn.: blind zone)

hornblende Black, dark green or greenish black, ferromagnesian silicate mineral. Crystals may be granular, fibrous or columnar. Primary constituent of may acid and intermediate igneous rocks, and a common metamorphic mineral in gneiss and schist.

hornblende schist Metamorphic, schistose rock with a high content of hornblende (equivalent with amphibolite).

hornfels Glassy, generally very fine-grained, rock produced by contact metamorphism. Shows no cleavage, schistosity or alignment of minerals.

horst A topographically high area bounded by faults.

hue Basic colour or a mixture of basic colours.

humus Strongly altered, organic material.

hydration Chemical reaction that results in the transfer of water from the fluid phase into the structure of a mineral.

hydraulic gradient The ratio of the pressure difference between two points and the distance between them.

hydrolysis Chemical process where a mineral reacts with water.

hydrothermal activity Circulation of hot fluids and gases, usually associated with movement of magma. Fluids often contain various minerals in solution which precipitate in rock joints and fissures.

hygroscopic Tending to absorb water.

hypabyssal A general term applied to minor intrusions, such as sills and dykes, and to the rocks of which they are made, to distinguish them from volcanic rocks and formations, on one hand, and plutonic rocks and major intrusions such as batholiths, on the other.

hyperite — Fine grained variety of gabbro with more or less ophitic texture. Contains rhombic as well as monoclinic pyroxene.

hypersthene — Mineral. A variety of rhombic pyroxene.

hypogene — A general term intended to include both plutonic and metamorphic classes of rocks, i.e., a term used for rocks formed within the earth.

hysteresis — The stress/strain curves are different for loading and unloading.

ignimbrite — Rock composed of compacted volcanic ejecta (*see* tuff).

illite — A clay mineral.

incipient joints — Joints that may develop parallel to bedding or parallel to foliation or cleavage.

incompetent rock — Rock which is weaker than adjacent rock and relatively more liable to deformation.

incompetent ground — Overstressed rock masses.

inclined fold — Fold whose axial plane is inclined from the vertical. One fold limb is commonly steeper than the other, but the steeper limb is not overfolded.

inclusion — A general term for foreign bodies (gas, liquid, glass, or mineral) enclosed by minerals; also extended in its English usage to cover enclosures of rocks and minerals within igneous rocks.

induration — The hardening of rock by the action of heat, pressure or cementation.

inequigranular — Textural term for a rock characterised by a mixture of crystals or grains of significantly different sizes.

inlier — An area or group of rocks surrounded by rocks of younger age.

in situ — Latin words (in situ) meaning "in place" or "in its original position". Distinguishes rocks and soils found in their original position of formation, as opposed to transported materials.

interbedded — Structural term for beds in a sedimentary deposit with mean spacing >20 mm lying between, or alternating with, other beds of different character (e.g., sand with interbedded clay).

intercalation — Lens or thin layer of other material occurring in sediment or sedimentary rock which is otherwise uniform.

interlaminated — Structural term similar to "interbedded", except applied to very thin beds with mean spacing <20 mm.

interlocking — Being joined or connected so that neither part can be operated independently.

intermediate — Chemical term for an igneous rock containing 54 to 62% silica and usually less than 50% dark minerals. (Contrast with "acid" and "basic").

interstratified — General structural term for sedimentary deposits consisting of alternating layers of different character. Covers both "interbedded" and "interlaminated".

intrusive — Term for an igneous rock that has been forced into pre-existing rocks and solidified from magma underground. (Contrast with "extrusive").

inverted bedding — Older layers are on top of younger.

irregular blocks — Wide variations of block sizes and shapes. See also polyhedral block.

irregular jointing — Wide variations of block size and shape.

isoclinal fold — Fold whose limbs are parallel (i.e., the inter-limb angle is 0^o).

isostacy	State of level compensation between masses of rocks in adjacent sectors of the crust.
isotropic	Term for rock and soil with the same physical properties in all directions.
joint	A discontinuity of natural origin along where it has been no visible displacement (geol.) In this book, joint is used as a scale term and therefore also includes minor shear ruptures.
joint coating	A thin (< 0.1 mm) mineral layer on discontinuity surfaces.
juvenile water	Groundwater believed to have come directly from magmatic sources.
kaolin	Group of clay minerals derived mainly by alteration of alkali feldspars and micas. Appearance is that of a soft, white or nearly white nonplastic clay. Commonly occurs as a thin coating or filling in joints in igneous rocks, but extreme alteration may convert whole rock mass to kaolin.
kaolinized	Alteration term for a rock containing minerals, especially feldspars and micas, replaced by, or altered to, kaolin as a result of hydrothermal activity.
karst topography	Topography characterised by sinkholes, caves, solution features and large underground drainage systems. Common in limestones, rare in other rocks.
keelformed overbreak	Characteristic shape of overbreak caused by high, anisotropic rock stress.
Keyper marl	A fine-grained red siltstone forming the higher part of the Triassic in north-west Europe, typically in Germany. (The term 'marl' is here incorrectly used.)
laccolith	A dome-shaped intrusion with both floor and roof concordant with the bedding planes of the invaded formation; the roof is arched upward as a result of the intrusion.
lacustrine sediments	Sediments deposited or formed in a lake.
lahar	Mudflow in volcanic material. Caused by water saturation (e.g., by intense rainfall) of unlithified lava or pyroclastic deposits on the flanks of a volcano.
laminated	Structural term for a sedimentary rock or superficial deposit formed, arranged or deposited in very thin layers <20mm thick.
lamprophyre	Dark coloured, very fine- to coarse-grained, basic rock characterised by high percentages of mafic minerals which often form megacrysts in a finer matrix of similar minerals plus altered feldspars.
landslide	Rapid downhill movement of rock and/or soil by gravity.
lapilli	Pyroclastic rock material of gravel size (i.e., 2 to 60mm). Descriptive term for tuff composed wholly or predominantly of this grain size.
laterite	Residual soil, usually reddish in colour, rich in secondary oxides of iron and/or aluminium. A product of intensive in situ rock weathering through leaching of more soluble elements. Common in tropical areas with strong seasonal rainfall.
latite	Volcanic rock with low quartz content (< 15%; i.e., monzonitic).
lava	General term for molten, extrusive magma erupting non-explosively from a volcanic vent or fissure. Also, the term for the rock solidified from this magma.
leaching	Separation and removal of the soluble constituents in a rock by the natural action of percolating groundwater.
lenticular	shaped approximately like a double convex lens. When a mass of rock thins out from the centre to a thin edge all around, it is said to be lenticular in form

lenticular bedding Beds in a sedimentary rock or superficial deposit formed by discontinuous lens-shaped bodies of one material surrounded by another type of material, e.g., sand lenses in a clay deposit. Lenses are usually double convex in cross-section.

leucite Mineral. A variety of feldspathoid.

leucocratic Light-coloured as applied to igneous rocks. Most fine- to coarse-grained acid rocks are leucocratic. (Contrast with "mesocratic" and "melanocratic").

lignite (Brown coal) A brownish-black coal, intermediate between peat and black coal.

limb One flank or side of a fold. A simple fold has two limbs.

limestone Sedimentary rock composed wholly or predominantly (50-100%) of calcium carbonate, mainly in the form of the mineral calcite. Other minerals: quartz, feldspar, mica.

limonite Usually dark brown or yellowish brown (may be yellow, red or nearly black), amorphous hydrated iron oxide material (ferric oxide). A very common weathering (oxidation) product of all iron-bearing minerals.

lineament Distinct, linear topographic features, often reflecting weakness zones. Easily recognisable on aerial photos

lineation General term for any rock structure arranged in lines. Also, the term for the appearance of stretched-out, flattened minerals in metamorphic rocks.

liquid limit Moisture content at which a soil passes from the plastic to the liquid state, as determined by the liquid limit test.

lithic Relating to or made of existing rock fragments. Term for a tuff composed predominantly of fragments of previously-formed rocks.

lithified Term for a rock which has been converted into a coherent solid mass from a newly-deposited loose sediment by such processes as cementation, compaction and crystallisation. Lithification may occur concurrent with, soon after or long after deposition.

lithology The character of rocks as based on the megascopic observation of hand specimens. In its French usage, the term is synonymous with petrography.

lithosphere That part of the Earth consisting of the crust and the upper mantle to a depth of about 100 km, forming the crustal plates.

loam Soil with a wide range of particle sizes.

lobate Term for a long, rounded, tongue-like shape. Often applicable to the shape of colluvial deposits.

loess Silt-size dust-like deposit washed out of the atmosphere by rain and accumulating only in grass plain regions.

loose deposits General term used for all kinds of soil.

macrostructure Structural features of a soil mass which can be identified by the naked eye. (Contrast with "microstructure").

mafic General term for dark-coloured, ferromagnesian minerals, or an igneous rock composed chiefly of these minerals. (Contrast with "felsic").

magma Molten rock material formed within the earth. Solidifies at or near the earth's crust to produce extrusive and intrusive igneous rocks Extrusive magma becomes "lava".

magnesite Carbonate mineral ($MgCO_3$).

magnetite Black iron oxide mineral with strong magnetic properties (Fe_3O_4).

major joint	Large joints, but smaller than the master joints.
mantle	Part of the Earth at depths between 10 km under ocean surface (more under land) and 2900 km, with a relative density between 3.3 and 5.7 as interpreted from records of seismic waves.
marble	Metamorphosed limestone. Generally light coloured (often stained by impurities), fine- to coarse-grained crystalline metamorphic rock consisting mainly of recrystallized calcite and/or dolomite.
marine limit	The highest level of the ocean. (In Norway: by the end of the last glaciation).
marl	Unconsolidated sediment of argillaceous and calcareous materials.
massive	Structural term for an igneous or metamorphic rock with homogeneous texture over large areas, i.e., with no layering, foliation or other planar structures. May also be applied to sedimentary rocks with no evidence of stratification (i. e. no bedding or lamination).
massive rock	Few joints or very wide joint spacing.
master joints	Joints that cut through a number of rock units and can be followed several tens or hundreds of metres.
matrix	Finer-grained material enclosing, or filling the spaces between, the larger grains or particles in a mixed sedimentary rock or superficial deposit. Synonymous with groundmass in an igneous rock.
megacryst	Any crystal or grain in an igneous or metamorphic rock that is significantly larger than the surrounding groundmass of matrix. A general, non-genetic term.
megascopic	A general term, more appropriate than macroscopic, applied to observations made on minerals and rocks by means of the naked eye or pocket lens but not with a microscope.
melanocratic	Dark-coloured, as applied to igneous rocks. All basic rocks are melanocratic. (Contrast with "leucocratic" and "mesocratic").
mesocratic	Medium-coloured (i.e., composed of roughly equal amounts of light and dark constituents), as applied to igneous rocks. Most intermediate rocks are mesocratic. (Contrast with "leucocratic" and "melanocratic").
meta-	Prefix used with an igneous, pyroclastic or sedimentary rock name to indicate that the rock has been partially metamorphosed, e.g., meta-tuff.
metamorphism	Alteration of mineralogy, texture and structure of the rock due to high pressure and/or temperature.
metasomatism	A process of replacement of minerals in a rock by additional constituents to form fresh minerals.
mica	A group of rock-forming silicate minerals, all with good cleavage in one plane only.
mica gneiss	Gneiss with > 40% mica.
mica quartzite	Quartzite with > 10% mica.
mica schist	Metamorphic, schistose rock, typically with 25-75% mica (biotite and muscovite), < 70% quartz, 0-25% feldspar and minor amounts of chlorite, epidote and garnet.
microcline	A variety of alkaline feldspar.
microcrystalline	Textural term for a crystalline rock with crystals that are too small to be seen by the naked eye, but which can be distinguished separately under an ordinary microscope.
micro-fissure	Minute defects from cm length downward in size into truly microscopic range. (Not included in the term joint)

micro-fractures General term for all small-scale discontinuities in the rock fabric. Includes cracks, fissures and planes of separation through or between individual grains.

microstructure Structural feature of a soil mass which cannot be identified completely by the naked eye; the use of a microscope is required for full assessment. (Contrast with "macrostructure").

migmatite A composite rock which includes igneous and metamorphic constituents.

mineral boxwork Weathering feature resulting from hard mineral deposits formed in rock joints standing out prominently on a weathered surface.

mineralised Transformation term for new minerals formed either by conversion of existing minerals, or by filling of discontinuities with new substances.

minor joints Smaller, relative unimportant joints.

Mohs' scale of hardness A scale divided into 10 classes defined by certain index minerals; from talc (hardness 1, softest) to diamond (hardness 10, hardest).

Mohr's envelope The envelope of Mohr's circles, representing failure at various stress levels.

molasse A provincial Swiss name for a soft green sandstone associated with marl and conglomerates, belonging to the Miocene Tertiary period, extensively developed in the lower country of Switzerland, and composed of Alpine detritus.

montmorillonite A swelling clay mineral belonging to the smectite group.

monzonite Plutonic rock with 0-15% quartz, 15-55% alkali feldspar, 10-55% plagioclase, 0-10% feldspathoids, 15-45% mica, hornblende, pyroxene.

moraine Materials deposited chiefly by direct glacial action. See drift.

mottled Term for non-uniform colour distribution of a rock or soil where the secondary colour constituent forms blotches or marks of approximately equal size.

mudstone Sedimentary rock composed predominantly of silt- and/or clay-size particles. A more general term than "siltstone" or "claystone".

muscovite Colourless, yellow or light brown mineral of the mica group. Forms distinctive shiny thin prisms or flakes. Very common in gneisses and schists, and some acid igneous rocks.

mylonite Very fine-grained crystalline metamorphic rock with streaked or banded texture produced by shearing and fracturing of original grains during intense dynamic metamorphism.

nepheline Mineral. A variety of feldspathoid.

nepheline syenite Plutonic rock with 5-55% nepheline, 10-80% alkali feldspar, 0-40% plagioclase, 10-45% hornblende, pyroxene, mica.

neutral fold Fold with its axial plane more or less horizontal. Neither an anticline nor a syncline.

nodule A small, irregular, rounded lump of a mineral or rock, usually contrasting in composition with the material in which it is embedded e.g., nodular chert in limestone.

non-systematic joints Random, often irregular or curved joints. Non-systematic joints do not have any definite pattern and frequently terminate at joints belonging to a set.

norite Plutonic rock with composition as gabbro, except for rhombic in stead of monoclinic pyroxene.

olistostrome Large (unsorted) fragments of volcanic and sedimentary rocks in a clayey/sandy matrix

olistostromal rocks Characterised by a chaotic mixture of sharp to partly rounded grains/particles made of volcanic rocks, turbiditic sandstones and limestones set in a sandy and limy matrix.

olivine	A green aluminium silicate mineral found in basic and ultrabasic igneous rocks.
olivinite	Plutonic, ultrabasic rock consisting mainly of olivine.
oolite	Limestone composed chiefly of ooliths, which are small (0.25 to 2 mm diameter) spherical particles.
opal	Amorphous silica mineral. Softer, less dense, less transparent and lacks crystalline structure compared with quartz. Occurs in nearly all colours. Transparent coloured varieties used as gemstones.
open fold	Fold with an inter-limb angle between 70^o and 120^o.
ophitic	A term applied to microscopic rock texture to designate a mass of longish, interlacing crystals, the spaces between which are filled with minerals of later crystallisation.
ore	Rock containing ore minerals, and economically mineable.
ore minerals	Minerals with density higher than $5 \cdot 10^3$ kg/m^3.
orogeny	The process of mountain formation by major folding.
orthoclase	Mineral. A variety of alkali feldspar.
orthogneiss	Gneiss of igneous origin.
orthotropic	Of a material which has different mechanical properties in one direction compared with the other two.
outcrop	Area with bedrock, ore body or weakness zone at the surface.
outlier	Outcrop of younger strata surrounded by older strata.
overbreak	Rock excavated beyond specified cross-section of a tunnel.
overburden	Overlying rock, or soil.
overcoring	A technique for isolating as far as possible a cylinder of rock carrying instrumentation from the surrounding rock; carried out with core drilling equipment.
overfolded	Term for a fold, or the limb of a fold, that has tilted beyond the perpendicular.
oxidation	Chemical weathering process involving the reaction between rocks and atmospheric oxygen, the oxygen usually being dissolved in water. The main products are oxides and hydroxides. Iron is the mineral most obviously affected; iron oxidation products are characteristically brown, red and yellow in colour. (Contrast with "reduction").
paragneiss	A gneiss of sedimentary origin.
parasitic fold	Small fold on the limb of a larger fold.
parent material (of a soil)	The pre-existing sediment or rock from which a soil is formed by weathering.
parting	1) Small discontinuities in rock, such as foliation or bedding partings. 2) A small joint in coal or rock.
peat	A deposit largely formed of dead vegetation which may be in course of consolidation.
pebble	A rock fragment between 4 and 64 mm in diameter, which has been rounded or otherwise abraded by the action of water, wind or glacier ice.
pegmatite	Light coloured, very coarse-grained igneous rock, generally of granitic composition. Commonly occurs as irregular dykes or veins, especially around the edges of large intrusions.
pelagic deposits	Ocean sediments without land-derived material.
peneplain	Plain (or almost a plain) which is the culmination of a cycle of erosion.

percolation — The movement of groundwater in the zone of saturation under hydrodynamic forces, generally with a dominant lateral component.

peridotite — Ultrabasic, igneous rock, formed essentially of the mineral olivine.

persistence — Joint length or continuity normally measured as trace length. May give a rude measure of the area extent or penetration length of a discontinuity. Termination in solid rock or against other discontinuities reduces the persistence.

persistent joint — Used to describe a dominating (master) joint.

pervasive — To pass through, used for prevalent (major) joints crossing other joints.

phanerocrystalline — A term applied to igneous rocks in which all the crystals of the essential minerals can be distinguished individually by the naked eye; contrasted with aphanitic.

phenocryst — A term applied to isolated larger crystals visible to the unaided eye and lying in a finer mass of a rock of igneous origin.

phonolite — Volcanic rock containing more than 60% feldspathoids.

phreatic — Relating to groundwater in the saturated zone.

phyllite — Fine-grained metamorphic rock with well-developed slightly undulating cleavage. Commonly green, grey or reddish brown in colour. Chlorite and sericite crystals often form a distinctive shiny, smooth surface on cleavage faces.

physiography — The shape of landforms, a synonym for geomorphology.

piezometer — Instrument for measuring pore pressure in soil and rock masses.

piping — An underground flow of water with a sufficient pressure gradient to cause scour along a preferred path.

pitted — Shape term for a rock particle with an uneven surface texture characterised by numerous small depressions. Commonly caused by preferential weathering and erosion of different minerals.

plagioclase feldspar — Group of sodium-calcium feldspars of general composition $(Na,Ca)Al(Si,AL)Si_2O_8$. (See "feldspar").

planar — A flat, level, even surface (the curvature is zero or minimal).

planarity — Character of the joint surface as related to an ideal plane. Deviations from planar surfaces can be on several scales from small scale (smoothness) to large scale (waviness).

plane strain — State of two-dimensional strain with zero strain perpendicular to the actual strain plane.

plane stress — Two-dimensional stress condition with zero stress perpendicularly to the actual stress plane.

plastic limit — Moisture content at which a soil becomes too dry to be in a plastic condition, as determined by the plastic limit test.

plasticity — Property which enables a soil or other material to be deformed continuously and permanently without rupture.

plate tectonics — A tectonic theory of lateral movement of lithosphere plates, to account for recorded movements of crustal sectors in geological time.

platy block — See tabular block.

Pleistocene — Geological time period between approximately 2 million and 8,000 to 10,000 years ago, i.e., immediately prior to the Holocene.

plunge — The inclination of a line.

plunging — Used to describe an anticline or syncline with an inclined axis.

pluton — A large mass of igneous rock inferred to have been formed by slow cooling in a pressure confined situation deep in the crust, i.e., a batholith.

plutonic — Pertaining to, or the general term for, any rock formed at considerable depth below the earth's surface by crystallisation of magma and/or by chemical alteration.

polygenic olistostromal rocks — Rocks characterised by a chaotic mixture of sharp to partly rounded grains/particles composed of volcanic rocks, turbiditic sandstones and limestones set in a sandy and limy matrix.

polyhedral — Shape term for a rock mass with no consistent joint sets, such that individual rock blocks usually vary widely in shape and size.

porosity — The ratio of the volume of voids in a rock or soil to its total volume.

porphyritic — Textural term for an igneous rock containing large crystals (phenocrysts) that are compatible in composition and mode of formation with the groundmass or matrix in which they occur. (Contrast with "xenocrystic").

porphyry — A general term used rather loosely for igneous rocks which contain relatively large isolated crystals set in a finer-grained mass.

primary joints — A set of joints being both larger and/or more frequent than joints of other sets in the same locality.

psammite — 1) Fine-grained, fissile, clayey sandstone (see arenite).
2) Any rock composed of sandy particles; sandstone.

pseudomorph — Mineral which occurs in the crystal form of another mineral as a result of alteration, or solution and replacement, within the same crystal shape.

pumice — Light-coloured glassy rock formed from acid lava. Contains abundant voids or cavities, which means it is often sufficiently buoyant to float on water.

pyrite — Light brown or dark yellow iron sulphide mineral (FeS_2). Often forms cube-shaped, striated crystals with a bright metallic surface. Common in veins and fault-zone rocks.

pyroclast — Individual rock fragment or particle ejected explosively from a volcanic vent. Classified by size into fine ash, coarse ash, lapilli, blocks and bombs.

pyroclastic — General term for any rock composed of material ejected explosively from a volcanic vent, including agglomerate, tuff, and ash.

pyroxene — Groups of mafic silicate minerals. Commonly appear as dark green or black prismatic crystals displaying cleavage in two directions parallel to the crystal faces and intersecting at approximately 90^o.

quartz — Colourless (often coloured by impurities), glassy, hard mineral composed of crystalline silica (SiO_2). Commonly appears either as six-sided transparent crystals or as a dense crystalline mass lacking distinctive shape. Very common in all types of rocks and mineral veins.

quartzite — A non-foliated metamorphic rock consisting mainly of quartz (> 75%). Formed by recrystallization of sandstone due to contact or regional metamorphism.

quartz latite — Medium-coloured, very fine-grained, intermediate igneous rock. The very fine-grained equivalent of quartz monzonite.

quartz monzonite — Medium-coloured, fine- to coarse-grained, intermediate igneous rock containing roughly equal amounts of plagioclase and alkali feldspar.

quartzphyric Textural term for a rock containing large megacrysts of quartz, e.g., quartzphyric rhyolite.

quartz syenite Medium-coloured, fine- to coarse-grained, intermediate igneous rock. Feldspar component is predominantly alkali feldspar.

quartz trachyte Medium-coloured, very fine-grained, intermediate igneous rock. The very fine-grained equivalent of quartz syenite.

recrystallization Formation of new crystalline mineral grains in a rock due to metamorphism or processes involving percolating groundwater. New crystals may have the same or a different composition from the original crystals.

recumbent fold Overturned fold whose axial plane is horizontal or nearly horizontal.

reduction Chemical process whereby oxygen is removed in rocks and the leached parts of soils. Related to the continuous presence of water, which makes oxygen scarce, e.g., by reducing ferric iron (Fe_2O_3) to ferrous iron (FeO). Characteristic colours of reduced soils are greens and greys. Often associated with strong bacterial activity in the soil. (Contrast with "oxidation").

regional metamorphism Large scale metamorphism connected to mountain range formation.

regular bedding Alternating layers of materials of different grain size in a bedded sedimentary deposit. Grain size within each layer is essentially uniform.

regular jointing Three joints sets orthogonally arranged as in igneous and metamorphic rocks.

remote sensing Measurements of the Earth's surface made from aircraft or satellite.

residual soils Soils remaining in their place of formation. (Contrast with "saprolite"; represents a more advanced stage of weathering then saprolite).

retrograde metamorphism Metamorphism creating rocks with new minerals which are stable at lower temperature and pressure than the original.

rhombic porphyry Volcanic rock with rhombic crystals of feldspar in a fine-grained matrix.

rhombohedral blocks Six-sided prisms each face of which is a rhombus.

rhomboidal blocks See rhombohedral blocks.

rhyodacite Medium-coloured, very fine-grained, acid igneous rock. Intermediate in composition between rhyolite and dacite. Contains less alkali feldspar than rhyolite and less plagioclase feldspar then dacite. Often contains megacrysts or quartz and feldspar.

rhyolite Medium-coloured, fine-grained, acid igneous rock. The very fine-grained equivalent of granite. Often contains megacrysts of quartz and feldspar.

rind Discoloured, relatively thin, often loose and flaky outer layer on the surface of a boulder or rock block caused by weathering.

rip rap Stones of irregular shape with specified limits of size, tipped or placed to protect an embankment from scour.

rock flour A general term for finely comminuted rock material corresponding in grade to mud, but formed by the grinding action of glaciers and ice sheets, and therefore composed largely of unweathered mineral particles.

rough Shape term for a rock particle with a surface texture that feels uneven, corrugated or lumpy, i.e., that lacks smoothness.

rounded Shape term for a rock particle with markedly rounded edges and corners.

rudaceous — Term for any sedimentary rock composed wholly or predominantly of gravel and larger-sized grains.

rupture — A fracture/break, here limited to have been developed by excavation activities.

salic — Common term for igneous rocks with high content of silicon and aluminium; i.e., light coloured, acid rocks.

sand — Soil particles 0.06 to 2 mm in size.

sandstone — Sedimentary rock composed predominantly of sand-size particles.

saprolite — Soil derived from in situ rock weathering which retains evidence of the original rock texture, fabric and structure. (Contrast with "residual soil").

saussurite — Alteration of feldspar into epidote and zoisite.

schist — Medium- to coarse-grained, foliated, crystalline metamorphic rock. Splits readily into flakes or slabs due to parallel arrangement of most of the constituent minerals. Coarser and more undulating foliation compared with "phyllite"; finer and often not banded compared with "gneiss".

schistosity — Foliation in schist or other coarse-grained crystalline metamorphic rock due to the parallel, planar arrangement of platy and prismatic mineral grains (e.g., mica).

scree — Large scale accumulation of talus near to the angle of repose.

seam — 1) A minor, often clay-filled zone with a thickness of some cm. When occurring as weak clay zone in a sedimentary sequence, a seam can be considerably thicker. Otherwise, seams may represent very minor faults or altered zones along joints, dikes, beds or foliation.
2) A plane in a coal bed at which the different layers of coal are easily separated.

seamy — A tunnel man's term and may be described as irregular schistose layers in crystalline rock; shale or clay layers commonly interbedded in sandstone or limestone; and also any rock with numerous clay-filled joints and fractures.

secant modulus — Young's modulus determined by the inclination of the line between origo and a given point on the stress/strain curve.

seismicity — Pertaining to earth vibrations or disturbances produced by earthquakes.

sericite — White, fine-grained mineral of the mica group. Similar composition to muscovite. Common in fault gouge and other rocks associated with dynamic metamorphism.

serpentine — A group of minerals, most commonly occurring in metamorphic rocks.

serpentinite — Metamorphic rock consisting mainly of serpentine.

shear plane — Surface along which differential movement has taken place parallel to the surface.

shear zone — Belt of rock of significant thickness that has been crushed and contorted by shear movement.

shrinkage joints — Joints caused by tensional forces set up in a rock body as a result of cooling (in an igneous rock) or desiccation (of a sedimentary rock).

sheeting joint — Joint developed more or less parallel to the surface of the ground. They probably arise as a result of pressure release due to removal of overlying rock by weathering and erosion. Also called an "unloading joint".

shoring — Form of prop or support, usually temporary, that is used in excavations. Shores are also used to support the forms for cast-in-place concrete slabs, beams, and girders in reinforced concrete frames.

shotcrete	Sprayed concrete (in USA. included gunite).
silica	Silicon dioxide (SiO_2). Occurs naturally as crystals (e.g., quartz), in cryptocrystalline form (e.g., chalcedony) and in amorphous form (e.g., opal). Combined in silicates as an essential constituent of many minerals.
silicate	Compound material consisting of one silicon and four oxygen atoms arranged in triangular pyramids, either isolated or joined through one of more of the oxygen atoms to form chains, sheets or three-dimensional structures with metallic elements such as aluminium. Silicate minerals are the most common rock-forming compounds and make up approximately 95% of the earth's crust
siliceous	Term of a rock containing abundant silica.
sill	Table-like body of intrusive igneous rock that conforms to the bedding or other planar structures of the country rock in which it is intruded.
sillimanite	Brown, grey, light green or white silicate mineral. Forms long needle-like crystals. Often found in high temperature, contact-metamorphosed sedimentary rocks.
silt	Soil particles 0.002 to 0.6 mm in size.
siltstone	Sedimentary rock composed predominantly of silt-size particles. (See also "mudstone").
skarn	Thermally metamorphosed impure limestone characterised by presence of silicate minerals containing calcium.
slaking	Breaking-up or disintegration of a rock or soil when saturated with or immersed in water.
slate	Fine-grained, metamorphic rock with a very well-developed parallel cleavage. Splits into very thin plates or flakes. Most slates are metamorphosed shales.
slickenside	The polished and striated surface that results from friction along a fault plane or other movement surfaces in a rock mass.
slump bedding	Beds in a sedimentary deposit which have been disturbed or deformed by slumping of the newly-deposited sediment under water, usually on a sloping surface.
smectite	A group of clay minerals with swelling properties.
smooth	An even or level surface; having no roughness or projections that can be seen or felt.
smoothness	The condition of asperities being the small scale joint surface roughness.
soapstone	Metamorphic, ultrabasic rock consisting mainly of talc, magnesite and chlorite.
solution	Chemical weathering process in which minerals are dissolved by percolating or static groundwater, e.g., removal of calcium carbonate in limestone or chalk by carbonic acid (weakly acid rainwater).
sorted	Term for a loose sediment or sedimentary rock composed of particles of essentially uniform size. "Well-sorted" refers to very uniform sorting. (Contrast with "poorly-sorted"). Note "sorted" in geological use is the opposite of "graded" in engineering use.
sparagmite	Arkosic sandstone of late Precambrian age.
spheroidal weathering	Mainly chemical weathering along fractures producing boulders of less weathered rock
spiling	Fan-shaped rock bolting ahead of the working face.
spilitic basalt	Basalt containing albitic feldspar, usually accompanied by chlorite, calcite, epidote, actinolite or other low grade greenstone minerals.
split	Separation lengthways, as along the direction of grain or layers. The term is used similar to parting.

splitting — 1) The property or tendency of a stratified (sedimentary) rock of separating along a plane or surface of parting.
2) A plane in a coal bed at which the different layers of coal are easily separated

spotted — Term for non-uniform colour distribution of rock or soil where the secondary colour constituent forms small rounded spots.

springline — The line defining the transition between wall and roof in a tunnel.

stand up time — The period of time a tunnel or cavern is stable without rock support.

stepped joint — The course of the joint is resembling a stair step.

stiffness — The ratio between stress and deformation.

strain — Relative elongation or shortening of a material as result of loading.

stratum — A layer that is separable along bedding planes from layers above and below; the separation arises from a break in deposition or a change in the character of the material deposited.

stratified — General structural term for a sedimentary rock or superficial deposit formed, arranged or deposited in layers or beds of any thickness. (See also "bedded" and "laminated

streaked — Term for non-uniform colour distribution of a rock or soil where the secondary colour constituent forms elongated, discontinuous, sometimes branching, lines

striated — Shape term for a rock particle with a surface texture characterised by a series of fine, parallel grooved lines. Caused, for example, by slickensiding in a fault zone.

striped — Term for non-uniform colour distribution of a rock or soil where the secondary colour constituent forms elongated, continuous, nonbranching lines.

structure — Manner of building. The arrangement or interrelation of all the parts of a whole. In geology a term reserved for the larger features of rocks.

structural domain — Portion of a rock mass characterised by a relatively uniform arrangement of discontinuities.

structural joints — Joints related to the structure or texture of the rock materials. They are mainly connected to the ability of the rock to split along schistosity, bedding or other weak layers or bands for example of mica, chlorite or other soft or anisotropy minerals.

structure — One of the larger features of a rock mass, like bedding, foliation, jointing, cleavage or brecciation; also the total sum of such features as contrast with texture. Also, in a broader sense, it refers to the structural features of an area such as anticlines or synclines. Not to be confused with “texture”.

sub — Prefix, which means: under, below, from, below, up, or near.

subangular — Shape term for a rock particle with slightly sharp (slightly angular) edges and corners.

subrounded — Shape term for a rock particle with slightly rounded edges and corners

subsidence — The sinking or caving in of the ground, or the settling of a structure to a lower level, essentially as a result of removal of support in an underground opening below.

supracrustal rock — Rock created at the surface (i.e., sedimentary and volcanic rocks).

syenite — Plutonic rock containing 0-20% quartz, 40-80% alkali feldspar, 0- 20% plagioclase, 0-10% feldspathoids , 10-35% hornblende, pyroxene, biotite.

syncline — Fold in the shape of a basin whose core contains the stratigraphically younger rocks

synform — Fold with the concave side upwards.

syngenetic	A term now applied to mineral or ore deposits formed contemporaneously with the enclosing rocks, a contrasted with epigenetic deposits, which are of later origin than the enclosing rocks.
talc	Mineral. Soft and very slippery. Most commonly occurring in basic and ultrabasic rocks, for instance soapstone.
talc schist	Metamorphic, highly schistose rock with a high content of talc, and also chlorite, hornblende and muscovite.
talus	Loose and incoherent deposits, usually at the foot of a slope or cliff.
tangent modulus	Young's modulus determined by the inclination of the tangent to the stress/strain curve.
tectonic joint	Joint formed by tectonic activity. The orientation of tectonic joints is usually controlled by the directions of the principal regional stresses.
throw	Amount of vertical displacement on a fault.
thrust	Low-angle reverse fault with a dip of less then 45^o.
thrust fault	A fault with a small dip angle, in which one set of rocks has been pushed over another set; an extreme type of reversed fault.
tight fold	Fold with an inter-limb angle between 0^o and 30^o
till	The unstratified or little-stratified deposits of glaciers.
tillite	A term applied to consolidated till formed during glacial epochs anterior to that of the Pleistocene
tonalite	Diorite with quartz constituting 5-20% of the light coloured minerals (quartz diorite).
trachyte	Volcanic rock consisting mainly of alkaline feldspar.
trachyandesite	Usually dark-coloured, fine-grained, intermediate igneous rock. Commonly contains megacrysts of alkali feldspar.
travertine	Calcium carbonate, $CaCo_3$, of light colour and usually concretionary and compact, deposited from solution in ground and surface waters. (Travertine forms the stalactites and stalagmites of limestone caves.)
tremolite	Mineral. A variety of monoclinic amphibole.
tufa	Precipitated limestone deposit found around springs issuing from a limestone formation.
tuff.	General rock name for all lithified pyroclastic rocks composed of rock fragments of gravel or finer size (< 60 mm). Subdivided according to dominant grain sizes into lapilli, coarse-ash and fine-ash types
tuffaceous	Term for a sedimentary rock containing up to 50% tuff material
tuffite	Mixed sedimentary/pyroclastic rock containing roughly equal amounts of sedimentary material and tuff material.
turbidite	A sedimentary deposit from a turbidity current, consisting of fragments from various rocks in a clayey/sandy matrix of volcanic/sedimentary composition.
unconformity	A surface of erosion which separates rocks of two substantially different ages.
undulation	A wavy, curving form or outline
uneven	Not uniform in height, breadth, etc. (an uneven floor)
unevenness	Small scale roughness that tends to be damaged during shear displacement unless the discontinuity walls are of high strength and/or the stress levels are low, so that dilation can occur on these small features.

unloading joint (See "sheeting joint").

upright fold Fold whose axial plane is vertical or near-vertical

vadose A term applied to seepage waters occurring below the surface and above the water table; contrasted with phreatic.

varves Thinly, laminated, fine-grain glacial lake sediments, reflecting seasonal changes in deposition.

vein Mineral filling a fault, joint or other fracture in a rock; the vein is formed later than the host rock. Commonly has a table- or sheet-like form. Often associated with alteration of the host rock. Most veins are of igneous origin

vent Opening at the earth's surface through which volcanic materials are extruded.

vesicle Cavity of variable shape in a lava, formed by the entrapment of a gas bubble during the solidification of the lava.

vermicullite Clay mineral usually formed from alteration of mica; it expands greatly when heated.

vesicular lavas Lavas containing gas bubbles which have been trapped during solidification, i.e., pumice. The bubbles may later be filled with minerals.

viscoelasticity The property of a material of partly elastic, partly viscous behaviour when applied to loading.

vitric Term for a pyroclastic rock composed predominantly of volcanic glass fragments.

voids ratio Ratio of volume of voids to volume of solids (of porosity).

volcanic General term for any extrusive igneous or pyroclastic rock

volcanic ash Deposits of sand grade and finer fragmental material derived from a volcano in eruption.

wacke (See "greywacke").

water content The ratio of weight of pore water to weight of solid material, expressed as percentage.

water-table Level up to which all rock pore-space is filled with undergroundwater.

waviness Large scale undulations/roughness which, if interlocked and in contact, cause dilation during shear displacement since they are too large to be sheared off.

wavy bedding Beds in a sedimentary deposit with markedly undulating bedding surfaces, i.e., the bed surfaces are not straight as in regular or graded bedding.

weathering pit Small shallow depression or basin in an otherwise flat or evenly sloping rock surface, caused by preferential weathering of specific rock fragments or crystals in rocks composed of mixtures of different fragments or crystals.

weathering rind (See "rind").

weathering zone Portion of a rock mass delineated on the basis of its degree of weathering in terms of, for example, relative content of rock and soil.

welded tuff Vitric tuff (i.e., with a high proportion of glass fragments) that has been compacted by the squeezing together of its glass fragments under the combined action of heat retained by its particles, weight of overlying material and hot gases within the rock.

wireline equipment Core-boring equipment which provides for the recovery of core tubes and cores by wire suspension through the drill rods.

wrench fault (See "strike-slip fault.

xenocrystic	Textural term for an igneous rock containing large crystals (xenocrysts) that are foreign in origin compared with the groundmass or matrix in which they occur. (Contrast with "porphyritic).
zeolite	Group of aluminium-silica minerals. Characterised by the ability of loosing crystalline water when heated, and regaining water when cooled.

15 Index

NOTE: Words in the text that appear in the index are indicated by itallics. Words not found here in the index, may possibly be found in Chapter 16: Glossary.

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16 Organisations, standards and enterprises

16.1 International organisations

16.1.1 International Society of Rock Mechanics (ISRM)

ISRM was founded in Saltzburg in 1962. The field of rock mechanics is defined by ISRM to include all studies relevant to the physical and mechanical behaviour of rocks and rock masses and the application of this knowledge for the better understanding of geological processes and in the fields of engineering.

ISRM have published Recommendations* and Suggested Methods for:

- Basic geotechnical description of rock masses* (April 1981).
- Blast vibration monitoring (March 1992).
- Deformability determinations using a large flatjack technique (April 1986).
- Design and analysis procedures for structures in argillaceous swelling rock* (October 1994).
- Determining in situ deformability of rock (1979)
- Geophysical logging of boreholes (February 1981).
- Hardness and abrasiveness of rocks (March 1977).
- In situ deformability of rock (September 1978).
- Laboratory testing of argillaceous swelling rocks (October 1989).
- Large scale sampling and triaxial testing of jointed rock (April 1985).
- Mode 1 fracture toughness using cracked chevron notched Brazilian disc (January 1995).
- Monitoring rock movements using borehole extensometers (November 1977).
- Monitoring rock movements using inclinometers and tiltmeters (December 1977).
- Petrographic description of rocks (March 1977).
- Point load strength (April 1985).
- Pressure monitoring using hydraulic cells (December 1979).
- Quantitative description of discontinuities in rock masses (October 1977).
- Rapid field identification of swelling and slaking rocks (October 1994).
- Rock anchorage testing (April 1985).
- Rock bolt testing (March 1974).
- Rock stress determination (February 1987).
- Shear strength (February 1974).
- Site investigation techniques* (July 1975).
- Sound velocity (March 1977).
- Strength of rock materials in triaxial compression, revised version (December 1983).
- Surface monitoring of movements across discontinuities (October 1984).
- Swelling rock* (October 1983).

- Tensile strength of rock materials (March 1977).
- Terminology* (July 1975).
- Uniaxial compressive strength and deformability of rock materials (September 1978).
- Water content, porosity, density, absorption and related properties and swelling and slake-durability index properties (October 1972).

Most of the Recommendations and Suggested Methods are printed in the ISRM's official Journal: International *Journal of Rock Mechanics, Mining Sciences & Geomechanics Abstracts* (see also the Literature List, Chapter 14).

Further information on ISRM and on how to order ISRM-documents may be obtained from the following web address:
☛ *http://www.lnec.pt*

16.1.2 International Association of Engineering Geology and the Environment (IAEG)

The IAEG was founded in 1964 and is affiliated to the International Union of Geological Sciences (IUGS). IAEG is a world-wide, scientific society with more than 5500 members in 66 National Groups as well as in individual memberships. The official languages of the IAEG are English and French. IAEG has a special membership fee for colleagues from developing countries, through their National Groups.

The aims of the International Association of Engineering Geology are:
- to promote and encourage the advancement of Engineering Geology through technological activities and research,
- to improve teaching and training in Engineering Geology, and
- to collect, evaluate and disseminate the results of engineering geological activities on a world-wide basis.

In its official journal, the *Bulletin of the International Association of Engineering Geology*, commissions established by IAEG have published a number of papers on site investigation and testing. Some of these are:
- Tunnelling in sulphate rocks. (1976) IAEG Bull. 13.
- Landslides and other mass movements. (1977) IAEG Bull. 16.
- Engineering geological mapping. (1979) IAEG Bull. 19.
- Soil and rock investigation by in situ testing. (1981) IAEG Bull. 25.
- Engineering geology related to nuclear waste disposal projects. (1986), IAEG Bull. 34.

IAEG has the following web address:
☛ *http://www.transport.ntua.gr/IAEG.html*

16.1.3 International Tunnelling Association (ITA)

ITA was established in 1974. The founders met in Oslo for meetings hosted by Norwegian tunnelling experts, who had established their own national society ten years earlier. The ITA's Secretariat was established early on in France and has remained there, supported by the host country.

ITA is bilingual (English and French) and over the years member nations have increased to 47, including members from all continents and the most important tunnelling countries.

Tunnelling may be too restrictive a description of the association's activities*. Association International des Travaux en Souterrain* is more descriptive of the current situation. ITA co-operates actively with the United Nations and is also closely connected to related organisations, especially the World Road Association (PIARC).

Important arrangements are the Annual General Meeting, The World Tunnel Congress and several regional conferences. ITA has a newsletter, *Tribune*, issued quarterly with a circulation of 3000 and the affiliated Tunnelling and Underground Space Technology (TUST) which publishes in-depth papers on selected topics.

The important work of ITA is undertaken by the Working Groups (WG). Today, the active WGs include:
WG 2 – Research
WG 3 – Contractual Practices in Underground Construction
WG 4 – Subsurface Planning
WG 5 – Health and Safety
WG 6 – Maintenance and Repair
WG 11 – Immersed and Floating Tunnels
WG 12 – Shotcrete Use in Tunnelling
WG 13 – Direct and Indirect Advantages of Underground Structures
WG 14 – Mechanised Tunnelling
WG 15 – Tunnels and Environment
WG 16 – Quality
WG 17 – Long Tunnels at Great Depth

The ITA has the following web address:
☞ *http://www.ita-aites.org*

16.1.4 European Federation of Explosives Engineers (EFEE)

This International Organisation puts emphasis on aspects of importance also to the tunnelling community, i.e. the explosives used, the handling of them, safety regulations and the training and licensing of the crew at face, especially the shotfirers. This organisation has limited its activities to major European countries.

16.2 Standards

16.2.1 Norwegian Standards (NS)

The Norwegian Council for Building Standardisation (NBR) has published a number of *Norwegian Standards* (*NS*) relevant for rock engineering, of which the following are the most important (see also Sections 7 and 11):

- NS 3420: Specification texts for building and construction, Chapter G - Rock (in Norwegian 1986, English edition 1993).
- NS 3480: Geotechnical planning (1988, in Norwegian).
- NS 8141: Vibrations and shocks in structures, Guidance limits for blasting-induced vibrations (1993, in Norwegian).
- NS-ENV 1997-1 NAD: National Application Document (NAD) for NS-ENV 1997-1:1997 Eurocode 7: Geotechnical design, Part 1: General rules (1997).

More information on NBR and how to order NS-Standards may be found on:
☛ *http://www.standard.no*

16.2.2 European Standards (CEN)

The European Committee for Standardisation (*CEN*) is in the process of issuing a number of *European Standards*, of which the following is most relevant within this field (see also Chapter 7):

- Eurocode 7: Geotechnical design - Part 1: General rules (ENV 1997-1:1994).

More information on the European Standards and how to obtain them may be found on the following web-address:
☛ *http://www.cenorm.be*

16.2.3 Other National Standards

Many countries have issued national standards within the fields of engineering geology and rock engineering. Most detailed and commonly used internationally are the American ASTM-Standards, the British BSI-Standards and the German DIN-Standards.

Further information on such standards and on how to obtain them, may be found on the respective web-sites:
☛ American: *http://www.astm.org*
☛ British: *http://www.bsi.org.uk*
☛ German: *http://www.din.de*

16.3 National organisations

16.3.1 The Norwegian Tunnelling Society (NFF)

Norsk Forening for Fjellsprengningsteknikk (NFF) or The Norwegian Tunnelling Society was established in 1964 in the wake of the establishment of The Office (later Institute) of Rock Blasting Techniques and the first Technical Conference on Rock Blasting which both took place in 1963.

Today NFF has grown to number 750 individual members and a further 50 corporate group or company memberships. NFF includes all walks of technical life as far as surface and underground rock excavation are concerned. Membership is open to the public - anyone interested in the subject may participate. NFF works closely with industry, governmental agencies, private sector professionals, R&D and scientific institutes, and the Norwegian educational system. It is affiliated to the Norwegian Society for Chartered Engineers. The Society is also a member of the International Tunnelling Association ITA and The European Federation of Explosives Engineers EFEE.

NFF has an Annual Meeting, a Board of Directors, Permanent Committees covering R&D, Technical issues, Conferences, International Activities and a number of ad hoc project groups.

The NFF agenda, as stated in its strategy document, aims to continuously improve and support:

- Innovation
- Technical and theoretical skills
- The exchange of experience
- The use of the underground
- International relations within ITA and EFEE
- Friendship and conviviality among its members.

16.3.2 The Norwegian Group for Rock Mechanics (NBG)

The organisation (Norsk bergmekanikkgruppe) was founded on 21 November 1968 and has approximately 200 individual members and 20 company memberships.

The organisation's purpose is to collect all Norwegian rock mechanics and engineering geology interests in common efforts to ease the exchange of knowledge and experience within the field of rock mechanics and engineering geology and to represent Norway internationally in these fields.

The organisation is affiliated with the Norwegian Soil and Rock Engineering Association (Jord- og Fjellteknisk Forbund) which is the umbrella organisation for several Norwegian organisations that work in the field of rock and soil mechanics. Internationally, the organisation is affiliated with the International Society of Rock Mechanics (ISRM) and the International Association of Engineering Geology and the Environment (IAEG).

Postal address:

Norwegian Group for Rock Mechanics	Telephone: +47 22 94 75 00
P.O. Box 2313, Solli	Telefax: +47 22 94 75 02
N-0201 OSLO NORWAY	✒ E-mail: knut.berg@nif.no

Internet address:
✒ *http://www.bergmekanikk.com*

16.4 Publications in the Norwegian Rock Excavations Series

		Pages	
Publication No. 1	Norwegian hard Rock Tunnelling	104	
Publication No. 2	Norwegian Tunnelling Technology	84	
Publication No. 3	Norwegian Hydropower Tunnelling	119	
Publication No. 4	Norwegian Road Tunnelling	172	
Publication No. 5	Norwegian Tunnelling Today	135	
Publication No. 6	Geology of Norway	4	& geological map
Publication No. 7	Norwegian Tunnels & Tunnelling	130	
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Publication No. 11	Norwegian TBM Tunnelling	118	
Gjøvik Olympic Mountain Hall Design & Construction		32	

Gjøvik Olympic Mountain Hall Video (13 min.) 1. PAL-VHS, 2. NTSC, 3. SECAM
Tunnelling in Norway '98 Video (10 min.) PAL-VHS

16.5 Norwegian business enterprises involved in rock excavation and rock engineering

The following Norwegian firms are active in the areas of rock excavation, planning and design. They have also generously contributed to the existence of this book.