Chapter 2

Classification and Index Properties of Rocks

2.1 Geological Classification of Rocks

Although they were not developed to satisfy the needs of civil engineers, the names geologists are able to attach to rock specimens on the basis of limited observations with a hand lens, or with the eye alone, do often reveal something about rock properties. If you are unfamiliar with the common rock names and how to assign them to an unknown rock, a review of geology is highly recommended. A good way to begin is to study Appendix 3, which explains simplified schemes for classifying and naming the principal rocks and minerals. Appendix 3 also lists the periods of the earth's history, the names of which indicate the age of a rock. A rock's age often, but not infallibly, correlates with its hardness, strength, durability, and other properties.

From a genetic point of view, rocks are usually divided into the three groups: *igneous, metamorphic,* and *sedimentary*. Yet these names are the *results*, not the starting point of classification. Since we are interested in behavioral rather than genetic attributes of rocks, it makes more sense to divide the rocks into the following classes and subclasses:

I. Crystalline Texture

	Examples
A. Soluble carbonates and salts	Limestone, dolomite, marble, rock salt, trona, 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

	Examples
A. Stably cemented	Silica-cemented sandstone 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 sandstones, tuff
E. Uncemented	Clay-bound sandstones

III. Very Fine-Grained Rocks

	Examples
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

	Examples
A. Soft coal	Lignite and bituminous coal
B. Hard coal	
C. "Oil shale"	
D. Bituminous shale	
E. Tar sand	

Crystalline rocks are constructed of tightly interlocked crystals of silicate minerals or carbonate, sulfate, or other salts (Figure 2.1*a*). Unweathered crystalline silicates like fresh granite are usually elastic and strong with brittle failure characteristics at pressures throughout the usual range for civil engineering works. However, if the crystals are separated by grain boundary cracks (fissures), such rocks may deform nonlinearly and "plastically" (irreversibly). Carbonates and crystalline salt rocks may also be strong and brittle but will become plastic at modest confining pressures due to intracrystalline gliding. Also, they are soluble in water. Mica and other sheet minerals like serpentine, talc, chlorite, and graphite reduce the strength of rocks due to easy sliding along the cleavage surfaces. Mica schists and related rocks are highly anisotropic rocks with low strength in directions along the schistosity (Figure 2.1*b*) except when the schistosity has been deformed through refolding. Volcanic rocks like basalts may present numerous small holes (vugs); otherwise, they behave similarly to granitic rocks (Figure 2.2*c*). Serpentinites, because they tend to be pervasively sheared on hidden surfaces within almost any hand specimen, are highly variable and often poor in their engineering properties.

The clastic rocks, composed of pieces of various rock types and assorted mineral grains, owe their properties chiefly to the cement or binder that holds the fragments together. Some are stably and tightly cemented and behave in a brittle, elastic manner. Others are reduced to sediment upon more soaking in water. In the clastic rock group, the geological names are not very useful for rock mechanics because the name doesn't indicate the nature of the cement. However, a full geological description can often suggest the properties of the cement; for example, a *friable* sandstone, where grains can be liberated by rubbing, is obviously incompletely or weakly cemented at best.

Shales are a group of rocks primarily composed of silt and clay that vary widely in durability, strength, deformability, and toughness. Cemented shales can be hard and strong. Many so-called "compaction shales" and "mudstones," however, are just compacted clay soils without durable binder, and have the attributes of hard soils rather than of rocks: they may exhibit volume change upon wetting or drying together with extreme variation in properties with variations in moisture content. Unlike soils, which quickly lose strength when kept moist at their natural water content, compaction shales remain

Figure 2.1 Photomicrographs of thin sections of rocks, viewed in polarized, transmitted light (courtesy of Professor H. R. Wenk). (*a*) Tightly interlocked fabric of a crystalline rock—diabase (×27).

Figure 2.1 Photomicrographs of thin sections or rocks, viewed in polarized, transmitted light (courtesy of Professor H. R. Wenk). (*b*) Highly anisotrophic fabric of a quartz mylonite (×20).

Figure 2.2 Photomicrographs of thin sections of fissured rocks, photographed in transmitted, polarized light (courtesy of H. R. Wenk). (*a*) Anorthosite with many intracrystalline and some intercrystalline fractures (×6.5).

Figure 2.2 Photomicrographs of thin sections of fissured rocks, photographed in transmitted, polarized light (courtesy of H. R. Wenk). (*b*) Gabbro with regular fissures oriented across the cleavage (×7).

Figure 2.2 Photomicrographs of thin sections of fissured rocks, photographed in transmitted, polarized light (courtesy of H. R. Wenk). *(c)* Volcanic rock (trachyte) with fissured sanidine phenocrysts (×30).

intact for some time. However, when dried and then immersed in water, they gradually decrease in density and strength over days, weeks, or longer. Chalk is a highly porous clastic carbonate rock that is elastic and brittle at low pressures, but plastic at moderate pressures.

Organic rocks include viscous, plastic, and elastic types. Hard coal and oil shale are strong, elastic rocks; however, the former may be fissured. Soft coal is highly fissued and may contain hydrocarbon gases under pressure in the pores. Tar sand may behave like a viscous liquid at high pressure or temperature; it also may contain gas under pressure.

We see that the rock family is large and "nonexclusive." Some of the simple laboratory tests and measurements enumerated below will help to decide what kind of material you are dealing with in any specific case.

2.2 Index Properties of Rock Systems

Because of the vast range in properties of rocks, which reflects varieties of structures, fabrics, and components, we rely on a number of basic measurements to describe rocks quantitatively. Certain properties that are relatively easy to measure are valuable in this regard and may be designated *index properties* for rock specimens. *Porosity* identifies the relative proportion of solids and voids; *density* adds information about the mineralogic or grain constituents. The *sonic velocity* together with a petrographic description evaluate the degree of fissuring. *Permeability* evaluates the relative interconnection of the pores; *durability* indicates the tendency for eventual breakdown of components or structures, with degradation of rock quality. Finally, *strength* determines the present competency of the rock fabric to bind the components together. These attributes need to be evaluated for engineering classification of rock, and together they permit one to draw useful correlations with experience for practical applications. However, the behavior of rock specimens under changing stress, temperature, fluid pressure, and time includes many other facets that are not represented by the above list of index properties. Therefore, characterization of a series of indexes in the laboratory is not a substitute for careful and detailed testing in other areas of special concern.

A list of index properties related to laboratory specimens of rock can help classify it for applications related primarily to the behavior of the *rock* itself as opposed to the *rock mass* with the interactions among its system of discontinuities. A little reflection on the spectrum of applications of rock mechanics will yield some that do involve mainly rock specimen characteristics, for example, drillability, cuttability, aggregate selection, and rip-rap evaluation. Most applications involving excavation at the surface or underground, on the other hand, test the system of discontinuities as much as or more than the nature of the rock itself. In these instances, the classification of the rock mass for engineering purposes reflects not only laboratory tests but structural and environmental characteristics of the rock mass in the field. We consider engineering classification of rock masses later in this chapter.

2.3 Porosity

The *porosity* of a rock, indicated by the dimensionless quantity n, is a fraction expressing the proportion of void space to total space in the rock.

$$n = \frac{v_p}{v_t} \tag{2.1}$$

where v_p is the volume of pores in total volume v_t. In sedimentary rocks, formed by the accumulation of grains, rock fragments, or shells, the porosity varies from close to 0 to as much as 90% ($n = 0.90$) with 15% as a typical value for an average sandstone. In these rocks, porosity generally decreases with age, and with depth below the surface, other things being equal. Table 2.1 illustrates these tendencies for a number of sedimentary rocks: a typical Cambrian sandstone had a porosity of 11% while a Cretaceous sandstone contained 34% pores. The effect of depth is most striking in the rocks derived from compaction[1] of clay as shown in Table 2.1. A Pennsylvanian age shale from Oklahoma encountered at depth of 1000, 3000, and 5000 feet had porosities of 16%, 7%, and 4%, respectively. Chalk is among the most porous of all rocks with porosities in some instances of more than 50%. These rocks are formed of the hollow skeletons of microscopic animals—coccoliths. Some volcanic rocks (e.g., pumice) can also present very high porosity due to the preservation of the sites of volcanic gas bubbles; in volcanic rocks, the system of pores is not always well connected.

In crystalline limestones and evaporites, and most igneous and metamorphic rocks, a large proportion of the pore space belongs to planar cracks termed *fissures* (Figure 2.2). A relatively small porosity due to fissures affects the properties of the rock to the same degree as a much larger percentage of subspherical pore space and, as noted in the previous chapter, creates stress dependency in a number of physical properties. In the igneous rocks, porosity is usually less than 1 or 2% unless weathering has taken hold. As weathering

[1] *Compaction* is a term used by geologists and petroleum engineers to describe processes by which a sediment is densified. Soils engineers reserve this term for processes of densification involving the expulsion of air from the voids. *Consolidation* refers to the expulsion of water from the voids of a clay, in soil mechanics usage, whereas geologists and petroleum engineers use *consolidation* for processes of lithification.

Table 2.1 Porosities of Some Typical Rocks Showing Effects of Age and Depth[a]

Rock	Age	Depth	Porosity (%)
Mount Simon sandstone	Cambrian	13,000 ft	0.7
Nugget sandstone (Utah)	Jurassic		1.9
Potsdam sandstone	Cambrian	Surface	11.0
Pottsville sandstone	Pennsylvanian		2.9
Berea sandstone	Mississippian	0–2000 ft	14.0
Keuper sandstone (England)	Triassic	Surface	22.0
Navajo sandstone	Jurassic	Surface	15.5
Sandstone, Montana	Cretaceous	Surface	34.0
Beekmantown dolomite	Ordovician	10,500 ft	0.4
Black River limestone	Ordovician	Surface	0.46
Niagara dolomite	Silurian	Surface	2.9
Limestone, Great Britain	Carboniferous	Surface	5.7
Chalk, Great Britain	Cretaceous	Surface	28.8
Solenhofen limestone		Surface	4.8
Salem limestone	Mississippian	Surface	13.2
Bedford limestone	Mississippian	Surface	12.0
Bermuda limestone	Recent	Surface	43.0
Shale	Pre-Cambrian	Surface	1.6
Shale, Oklahoma	Pennsylvanian	1000 ft	17.0
Shale, Oklahoma	Pennsylvanian	3000 ft	7.0
Shale, Oklahoma	Pennsylvanian	5000 ft	4.0
Shale	Cretaceous	600 ft	33.5
Shale	Cretaceous	2500 ft	25.4
Shale	Cretaceous	3500 ft	21.1
Shale	Cretaceous	6100 ft	7.6
Mudstone, Japan	Upper Tertiary	Near surface	22–32
Granite, fresh		Surface	0 to 1
Granite, weathered			1–5
Decomposed granite (Saprolyte)			20.0
Marble			0.3
Marble			1.1
Bedded tuff			40.0
Welded tuff			14.0
Cedar City tonalite			7.0
Frederick diabase			0.1
San Marcos gabbro			0.2

[a] Data selected from Clark (1966) and Brace and Riley (1972).

progresses, the porosity tends to increase to 20% or more. As a result, measurement of porosity can serve as an accurate index to rock quality in such rocks. In several projects in granitic rocks the National Civil Engineering Laboratory of Portugal was able to classify the rock for the purposes of engineering design mainly on the basis of a quick porosity measurement, obtained from the water content of the rock after immersion for 24 hours at a standard temperature and pressure (Hamrol, 1961). Among unweathered rocks, there is also a general correlation between porosity and mechanical properties such as unconfined compressive strength and modulus of elasticity; but such relationships are usually marked by enormous scatter. In the case of weak sandstones (having saturated compressive strength less than 20 MPa) Dobereiner and de Freitas (1986) have demonstrated good correlations of density, modulus of elasticity, and compressive strength with the saturated moisture content. The moisture content of a saturated specimen is linked with its porosity by Equation 2.5. Saturation can be approached by soaking a specimen in water while it is subjected to a laboratory vacuum.

Porosity can be measured in rock specimens by a variety of techniques. Since it is the pore space that governs the quantity of oil contained in a saturated petroleum reservoir, accurate methods for porosity determination in sandstones have been developed by the oil industry. However, these methods are not always suitable for measurements in hard rocks with porosities of less than several percent. Porosities can be determined from the following calculations.

1. Measured density.
2. Measured water content after saturation in water.
3. Mercury content after saturation with mercury using a pressure injector.
4. Measured solid volume and pore air volume using Boyle's law.

These are considered further below.

2.4 Density

The density or "unit weight" of a rock, γ, is its specific weight (FL^{-3}),[2] for example, pounds per cubic foot or kilonewtons per cubic meter. The *specific gravity* of a solid, G, is the ratio between its density and the unit weight of water γ_w; the latter is approximately equal to 1 g-force/cm³ (9.8 kN/m³ or approximately 0.01 MN/m³).[3] Rock with a specific gravity of 2.6 has a density

[2] The terms in parenthesis indicate the dimensions of the preceding quantity. F, L, T indicate force, length, and time, respectively.

[3] At 20°C, the unit weight of water is 0.998 g/cm³ × 980 cm/s² = 978 dynes/cm³ or = 0.998 g-force/cm³.

of approximately 26 kN/m³. In the English system, the density of water is 62.4 pounds per cubic foot. (Mass density ρ equals γ/g.)

It was stated previously that the porosity of a rock can be calculated from knowledge of its weight density. This assumes that the specific gravity of the grains or crystals is known; grain specific gravity can be determined by grinding the rock and adapting methods used in soils laboratories. If the percentages of different minerals can be estimated under a binocular microscope, or from a thin section, the specific gravity of the solid part of a rock can then be calculated as the weighted average of the specific gravities of the component grains and crystals:

$$G = \sum_{i=1}^{n} G_i V_i \tag{2.2}$$

where G_i is the specific gravity of component i, and V_i is its volume percentage in the solid part of the rock. The specific gravities of a number of common rock-forming minerals are listed in Table 2.2. The relation between porosity and dry density γ_{dry} is

$$\gamma_{dry} = G\gamma_w(1 - n) \tag{2.3}$$

Table 2.2 *Specific Gravities of Common Minerals*[a]

Mineral	G
Halite	2.1–2.6
Gypsum	2.3–2.4
Serpentine	2.3–2.6
Orthoclase	2.5–2.6
Chalcedony	2.6–2.64
Quartz	2.65
Plagioclase	2.6–2.8
Chlorite and illite	2.6–3.0
Calcite	2.7
Muscovite	2.7–3.0
Biotite	2.8–3.1
Dolomite	2.8–3.1
Anhydrite	2.9–3.0
Pyroxene	3.2–3.6
Olivine	3.2–3.6
Barite	4.3–4.6
Magnetite	4.4–5.2
Pyrite	4.9–5.2
Galena	7.4–7.6

[a] A. N. Winchell (1942).

The dry density is related to the wet density by the relationship

$$\gamma_{dry} = \frac{\gamma_{wet}}{1 + w} \tag{2.4}$$

where w is the water content of the rock (dry weight basis).

Water content and porosity are related by

$$n = \frac{w \cdot G}{1 + w \cdot G} \tag{2.5}$$

If the pores of the rock are filled with mercury, and the mercury content is determined to be w_{Hg} (as a proportion of the dry weight of the rock before mercury injection), the porosity can be calculated more accurately as follows:

$$n = \frac{w_{Hg} \cdot G/G_{Hg}}{1 + (w_{Hg} \cdot G/G_{Hg})} \tag{2.6}$$

The specific gravity of mercury (G_{Hg}) equals 13.546.

The densities of some common rocks are given in Table 2.3. These figures are only sample values, of course, since special factors can cause wide variations in individual formations.

Rocks exhibit a far greater range in density values than do soils. Knowledge of rock density can be important to engineering and mining practice. For example, the density of a rock governs the stresses it will experience when acting as a beam spanning an underground opening; unusually high density in a roof rock implies a shortened limiting safe span. A concrete aggregate with higher than average density can mean a smaller volume of concrete required for a gravity retaining wall or dam. Lighter than average aggregate can mean lower stresses in a concrete roof structure. In oil shale deposits, the density indicates the value of the mineral commodity because the oil yield correlates directly with the unit weight; this is true because oil shale is a mixture of a relatively light constituent (kerogen) and a relatively heavy constituent (dolomite). In coal deposits, the density correlates with the ash content and with the previous depth of cover, accordingly with the strength and elasticity of the rock. It is easy to measure the density of a rock; simply saw off the ends of a dried drill core, calculate its volume from the dimensions, and weight it. In view of the possible significance of variations from the norm, density should therefore be measured routinely in rock investigations.

2.5 Hydraulic Permeability and Conductivity

Measurement of the permeability of a rock sample may have direct bearing on a practical problem, for example, pumping water, oil, or gas into or out of a

Table 2.3 *Dry Densities of Some Typical Rocks*[a]

Rock	Dry (g/cm³)	Dry (kN/m³)	Dry (lb/ft³)
Nepheline syenite	2.7	26.5	169
Syenite	2.6	25.5	162
Granite	2.65	26.0	165
Diorite	2.85	27.9	178
Gabbro	3.0	29.4	187
Gypsum	2.3	22.5	144
Rock salt	2.1	20.6	131
Coal	0.7–2.0		
(density varies with the ash content)			
Oil shale	1.6–2.7		
(density varies with the kerogen content, and therefore with the oil yield in gallons per ton)			
30 gal/ton rock	2.13	21.0	133
Dense limestone	2.7	20.9	168
Marble	2.75	27.0	172
Shale, Oklahoma[b]			
1000 ft depth	2.25	22.1	140
3000 ft depth	2.52	24.7	157
5000 ft depth	2.62	25.7	163
Quartz, mica schist	2.82	27.6	176
Amphibolite	2.99	29.3	187
Rhyolite	2.37	23.2	148
Basalt	2.77	27.1	173

[a] Data from Clark (1966), Davis and De Weist (1966), and other sources.
[b] This is the Pennsylvanian age shale listed in Table 2.1.

porous formation, disposing of brine wastes in porous formations, storing fluids in mined caverns for energy conversion, assessing the water tightness of a reservoir, dewatering a deep chamber, or predicting water inflows into a tunnel. In many instances the system of discontinuities will radically modify the permeability values of the rock in the field as compared to that in the lab, so that some sort of in situ pumping test will be required for an acceptable forecast of formation permeabilities. Our motivation for selecting permeability as an index property of rock is that it conveys information about the degree of interconnection between the pores or fissures—a basic part of the rock framework. Furthermore, the variation of permeability with change in normal stress, especially as the sense of the stress is varied from compression to tension, evaluates the degree of fissuring of the rock, since flat cracks are greatly affected by normal stress whereas spherical pores are not. Also, the degree to which the permeability changes by changing the permeant from air to water expresses interaction between the water and the minerals or binder of the rock and can detect subtle but fundamental flaws in the integrity of the rock; this promising aspect of permeability as an index has not been fully researched.

Most rocks obey Darcy's law. For many applications in civil engineering practice, which may involve water at about 20°C, it is common to write Darcy's law in the form

$$q_x = k\frac{dh}{dx}A \tag{2.7}$$

where q_x is the flow rate (L^3T^{-1}) in the x direction
h is the hydraulic head with dimension L
A is the cross-sectional area normal to x (dimension L^2)

The coefficient k is termed *the hydraulic conductivity*; it has dimensions of velocity (e.g., centimeters per second or feet per minute). When temperature will vary considerably from 20°C or when other fluids are to be considered, a more useful form of Darcy's law is

$$q_x = \frac{K}{\mu}\frac{dp}{dx}A \tag{2.8}$$

in which p is the fluid pressure (equal to $\gamma_w h$) with dimensions of FL^{-2} and μ is the viscosity of the permeant with dimensions $FL^{-2}T$. For water at 20°C, $\mu = 2.098 \times 10^{-5}$ lb s/ft^2 = 1.005×10^{-3} N s/m^2 and $\gamma = 62.4$ lb/ft^3 = 9.80 kN/m^3.

When Darcy's law is written this way, the coefficient K is independent of the properties of the fluid. Its dimensions are those of area (e.g., square centimeters). K is termed *the hydraulic permeability*.

A common permeability unit is the darcy: 1 darcy equals 9.86×10^{-9} cm^2. Table 2.4 gives typical values of conductivities calculated for the properties of water at 20°C; 1 darcy corresponds approximately to a conductivity value of 10^{-3} cm/s.

Permeability can be determined in the laboratory by measuring the time for a calibrated volume of fluid to pass through the specimen when a constant air pressure acts over the surface of the fluid. An alternative method is to generate radial flow in a hollow cylindrical specimen, prepared by drilling a coaxial central hole in a drill core. When the flow is from the outer circumference toward the center, a compressive body force is generated, whereas when the flow is from the central hole toward the outside, a tensile body force is set up. Consequently, rocks that owe their permeability partly to the presence of a network of fissures demonstrate a profound difference in permeability values according to the direction of flow. A radial permeability test was devised by Bernaix (1969) in testing the foundation rock of the Malpasset Dam after the failure. The permeability of the mica schist from that site varied over as much as 50,000 times as the conditions were changed from radially outward flow with

Table 2.4 Conductivities of Typical Rocks[a]

Rock	k (cm/s) for Rock with Water (20°C) as Permeant: Lab	Field
Sandstone	3×10^{-3} to 8×10^{-8}	1×10^{-3} to 3×10^{-8}
Navajo sandstone	2×10^{-3}	
Berea sandstone	4×10^{-5}	
Greywacke	3.2×10^{-8}	
Shale	10^{-9} to 5×10^{-13}	10^{-8} to 10^{-11}
Pierre shale	5×10^{-12}	2×10^{-9} to 5×10^{-11}
Limestone, dolomite	10^{-5} to 10^{-13}	10^{-3} to 10^{-7}
Salem limestone	2×10^{-6}	
Basalt	10^{-12}	10^{-2} to 10^{-7}
Granite	10^{-7} to 10^{-11}	10^{-4} to 10^{-9}
Schist	10^{-8}	2×10^{-7}
Fissured schist	1×10^{-4} to 3×10^{-4}	

[a] Data from Brace (1978), Davis and De Wiest (1966), and Serafim (1968).

ΔP of 1 bar, to radially inward flow with ΔP of 50 bars. The hydraulic conductivity (velocity units) from a radial flow test can be approximated by

$$k = \frac{q\ln(R_2/R_1)}{2\pi L\Delta h} \tag{2.9}$$

where q is the volume rate of flow
L is the length of the specimen
R_2 and R_1 are the outer and inner radii of the specimen
Δh is the head difference across the flow region corresponding to ΔP

An advantage of the radial permeability test, in addition to its capability to distinguish flow in fissures from flow in pores, is the fact that very large flow gradients can be generated, allowing permeability measurement in the millidarcy region. For rocks considerably less permeable than that, for example, granites with permeability in the region 10^{-9} darcy and below, Brace et al. (1968) devised a transient flow test.

Dense rocks like granite, basalt, schist, and crystalline limestone usually exhibit very small permeability as laboratory specimens, yet field tests in such rocks may show significant permeability as observed in Table 2.4. The reason for this discrepancy is usually attributed to regular sets of open joints and fractures throughout the rock mass. Snow (1965) showed that it is useful to idealize the rock mass as a system of parallel smooth plates, all flow running between the plates. When there are three mutually perpendicular sets of fractures with parallel walls, all with identical aperture and spacing and ideally smooth, the conductivity of the rock mass is theoretically expressed by

$$k = \frac{\gamma_w}{6\mu}\left(\frac{e^3}{S}\right) \tag{2.10}$$

where S is the *spacing* between fractures and e is the fracture *aperture* (interwall separation). It is seldom feasible to calculate the rock permeability from a description of the fractures, although Rocha and Franciss (1977) have shown how this can be done by using oriented, continuous core samples and correcting the data with results from a few pumping tests. Equation 2.10 is useful, however, for calculating the hypothetical fracture aperture e, that gives the same permeability value as measured in the field (corresponding to an assigned fracture spacing S). The aperture and spacing of the fractures then provide quantitative indexes of rock mass quality.

2.6 Strength

The value of having an index to rock strength is self-evident. The problem is that strength determinations on rock usually require careful test setup and specimen preparation, and the results are highly sensitive to the method and style of loading. An index is useful only if the properties are reproducible from one laboratory to another and can be measured inexpensively. Such a strength index is now available using the point load test, described by Broch and Franklin (1972). In this test, a rock is loaded between hardened steel cones, causing failure by the development of tensile cracks parallel to the axis of loading. The test is an outgrowth of experiments with compression of irregular pieces of rock in which it was found that the shape and size effects were relatively small and could be accounted for, and in which the failure was usually by induced tension. In the Broch and Franklin apparatus, which is commercially available, the *point load strength* is

$$I_s = \frac{P}{D^2} \tag{2.11}$$

where P is the load at rupture, and D is the distance between the point loads. Tests are done on pieces of drill core at least 1.4 times as long as the diameter. In practice there is a strength/size effect so a correction must be made to reduce results to a common size. Point load strength is found to fall by a factor of 2 to 3 as one proceeds from cores with diameter of 10 mm to diameters of 70 mm; therefore, size standardization is required. The *point load index* is reported as the point load strength of a 50-mm core. (Size correction charts are

Table 2.5 *Typical Point Load Index Values*[a]

Material	Point Load Strength Index (MPa)
Tertiary sandstone and claystone	0.05–1
Coal	0.2–2
Limestone	0.25–8
Mudstone, shale	0.2–8
Volcanic flow rocks	3.0–15
Dolomite	6.0–11

[a] Data from Broch and Franklin (1972) and other sources.

given by Broch and Franklin.) A frequently cited correlation between point load index and unconfined compression strength is

$$q_u = 24 I_{s(50)} \tag{2.12}$$

where q_u is the unconfined compressive strength of cylinders with a length to diameter ratio of 2 to 1, and $I_{s(50)}$ is the point load strength corrected to a diameter of 50 mm. However, as shown in Table 3.1, this relationship can be severely inaccurate for weak rocks and it should be checked by special calibration studies wherever such a correlation is important in practice.

The point load strength test is quick and simple, and it can be done in the field at the site of drilling. The cores are broken but not destroyed, since the fractures produced tend to be clean, single breaks that can be distinguished from preexisting fractures sampled by the drilling operation. Point load test results can be shown on the drill log, along with other geotechnical information, and repetition of tests after the core has dried out can establish the effect of natural water conditions on strength. Values of the point load index are given for a number of typical rocks in Table 2.5.

2.7 Slaking and Durability

Durability of rocks is fundamentally important for all applications. Changes in the properties of rocks are produced by exfoliation, hydration, decrepitation (slaking), solution, oxidation, abrasion, and other processes. In some shales and some volcanic rocks, radical deterioration in rock quality occurs rapidly after a new surface is uncovered. Fortunately, such changes usually act imperceptibly through the body of the rock and only the immediate surface is degraded in tens of years. At any rate, some index to the degree of alterability of rock is required. Since the paths to rock destruction devised by nature are many and varied, no test can reproduce expectable service conditions for more than a few special situations. Thus an index to alteration is useful mainly in offering a relative ranking of rock durability.

One good index test is the *slake durability* test proposed by Franklin and Chandra (1972). The apparatus consists of a drum 140 mm in diameter and 100 mm long with sieve mesh forming the cylindrical walls (2 mm opening); about 500 g of rock is broken into 10 lumps and loaded inside the drum, which is turned at 20 revolutions per minute in a water bath. After 10 min of this slow rotation, the percentage of rock retained inside the drum, on a dry weight basis, is reported as the *slake durability index* (I_d). Gamble (1971) proposed using a second 10-min cycle after drying. Values of the slake durability index for representative shales and claystones tested by Gamble varied over the whole range from 0 to 100%. There was no discernible connection between durability and geological age but durability increased linearly with density and inversely with natural water content. Based upon his results, Gamble proposed a classification of slake durability (Table 2.6).

Morgenstern and Eigenbrod (1974) expressed the durability of shales and claystones in terms of the rate and amount of strength reduction resulting from soaking. They showed that noncemented claystone or shale immersed in water tends to absorb water and soften until it reaches its *liquid limit*. The latter can be determined by a standard procedure described in ASTM designation D423-54T after disaggregating the rock by shaving it with a knife and mixing the shavings with water in a food blender. Materials with high liquid limits are more severely disrupted by slaking than those with low liquid limits. Classes of amounts of slaking were therefore defined in terms of the value of the liquid limit as presented in Table 2.7. The *rate* at which slaking occurs is independent of the liquid limit but can be indexed by the rate of water content change following soaking. The rate of slaking was classified in terms of the *change in liquidity index* (ΔI_L) following immersion in water for 2 h; ΔI_L is defined as

$$\Delta I_L = \frac{\Delta w}{w_L - w_P} \tag{2.13}$$

Table 2.6 *Gamble's Slake Durability Classification*

Group Name	% Retained after One 10-min Cycle (Dry Weight Basis)	% Retained after Two 10-min Cycles (Dry Weight Basis)
Very high durability	>99	>98
High durability	98–99	95–98
Medium high durability	95–98	85–95
Medium durability	85–95	60–85
Low durability	60–85	30–60
Very low durability	<60	<30

Table 2.7 *Description of Rate and Amount of Slaking*[a]

Amount of Slaking	*Liquid Limit (%)*
Very low	<20
Low	20–50
Medium	50–90
High	90–140
Very high	>140
Rate of Slaking	***Change in Liquidity Index after Soaking 2 h***
Slow	<0.75
Fast	0.75–1.25
Very fast	>1.25

[a] After Morgenstern and Eigenbrod (1974).

where Δw is the change in water content of the rock or soil after soaking for 2 h on filter paper in a funnel
w_P is the water content at the plastic limit
w_L is the water content at the liquid limit

All the water contents are expressed as a percentage of the dry weight. These indexes and procedures for determining them are described in most textbooks on soil mechanics (e.g., Sowers and Sowers, cited in Chapter 9).

2.8 Sonic Velocity as an Index to Degree of Fissuring

Measurement of the velocity of sound waves in a core specimen is relatively simple and apparatus is available for this purpose. The most popular method pulses one end of the rock with a piezoelectric crystal and receives the vibrations with a second crystal at the other end. The travel time is determined by measuring the phase difference with an oscilloscope equipped with a variable delay line. It is also possible to resonate the rock with a vibrator and then calculate its sonic velocity from the resonant frequency, known dimensions, and density. Both longitudinal and transverse shear wave velocities can be determined. However, the index test described here requires the determination of only the longitudinal velocity V_l, which proves the easier to measure. ASTM Designation D2845-69 (1976) describes laboratory determination of pulse velocities and ultrasonic elastic constants of rock.

Theoretically, the velocity with which stress waves are transmitted through rock depends exclusively upon their elastic properties and their density (as explored in Chapter 6). In practice, a network of fissures in the specimen superimposes an overriding effect. This being the case, the sonic velocity can serve to index the degree of fissuring within rock specimens.

Fourmaintraux (1976) proposed the following procedure. First calculate the longitudinal wave velocity (V_l^*) that the speciment would have if it lacked pores or fissures. If the mineral composition is known, V_l^* can be calculated from

$$\frac{1}{V_l^*} = \sum_i \frac{C_i}{V_{l,i}} \tag{2.14}$$

where $V_{l,i}$ is the longitudinal wave velocity in mineral constituent i, which has volume proportion C_i in the rock. Average velocities of longitudinal waves in rock-forming minerals are given in Table 2.8. Table 2.9 lists typical values of V_l^* for a few rock types.

Now measure the actual velocity of longitudinal waves in the rock specimen and form the ratio V_l/V_l^*. As a quality index define

$$\text{IQ\%} = \frac{V_l}{V_l^*} \times 100\% \tag{2.15}$$

Experiments by Fourmaintraux established that IQ is affected by pores (spherical holes) according to

$$\text{IQ\%} = 100 - 1.6n_p\% \tag{2.16}$$

where $n_p\%$ is the porosity of nonfissured rock expressed as a percentage. However, if there is even a small fraction of flat cracks (fissures), Equation 2.16 breaks down.

Table 2.8 *Longitudinal Velocities of Minerals*

Mineral	V_l (m/s)
Quartz	6050
Olivine	8400
Augite	7200
Amphibole	7200
Muscovite	5800
Orthoclase	5800
Plagioclase	6250
Calcite	6600
Dolomite	7500
Magnetite	7400
Gypsum	5200
Epidote	7450
Pyrite	8000

From Fourmaintraux (1976).

Table 2.9 *Typical Values of V_l^* for Rocks*

Rock	V_l^* (m/s)
Gabbro	7000
Basalt	6500–7000
Limestone	6000–6500
Dolomite	6500–7000
Sandstone and quartzite	6000
Granitic rocks	5500–6000

[a] From Fourmaintraux (1976).

For example, a sandstone with n_p equals 10% had IQ equal to 84%. After heating the rock to a high temperature that produced an additional increment of flat cracks amounting to 2% pore space (n_p = 10%, n = 12%), IQ fell to 52%. (Heating opens grain boundary cracks in minerals with different coefficients of thermal expansion in different directions, in this case quartz.)

Because of this extreme sensitivity of IQ to fissuring and based upon laboratory measurements and microscopic observations of fissures, Fourmaintraux proposed plotting IQ versus porosity (Figure 2.3) as a basis for describing the degree of fissuring of a rock specimen. Entering the figure with known porosity

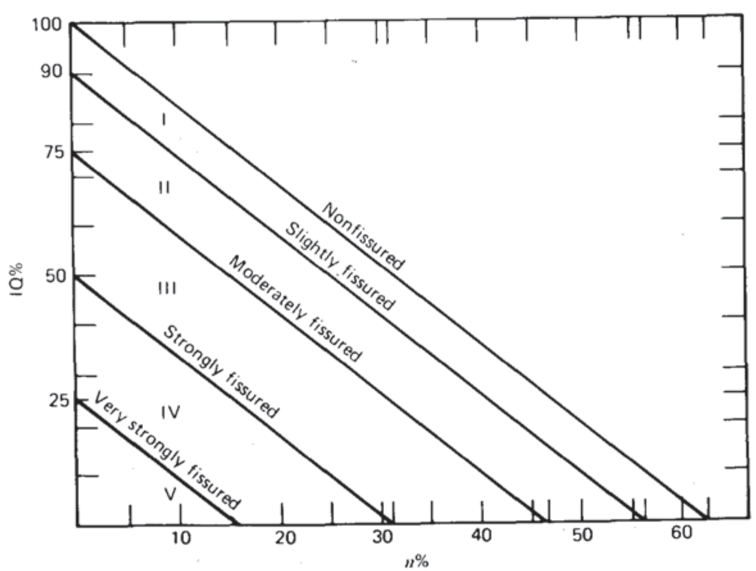

Figure 2.3 Classification scheme for fissuring in rock specimens. (After Fourmaintraux 1976.)

and calculated IQ defines a point in one of the five fields: (I) nonfissured to slightly fissured, (II) slightly to moderately fissured, (III) moderately to strongly fissured, (IV) strongly to very strongly fissured, and (V) extremely fissured. Although it would be better to determine the length, distribution, and extent of fissures by direct microscopic techniques, this necessitates tools and procedures that are not generally available. On the other hand, using Figure 2.3, the degree of fissuring can be appreciated and named readily and inexpensively in almost any rock mechanics laboratory.

2.9 Other Physical Properties

Many other physical properties are important to specific engineering tasks in rock. The hardness of rock affects drillability. Elasticity and stress-strain coefficients are basic to engineering for dams and pressure tunnels. The thermal properties—heat conductivity and heat capacity and the coefficient of linear expansion—affect storage of hot and cold fluids in caverns and geothermal energy recovery. The following chapters consider some of these rock specimen attributes further. As noted previously, an overriding influence on rock behavior in many instances stems from the characteristics of the discontinuities, including joints, bedding, foliation, and fractures. This is addressed by a meaningful system of rock classification that attempts to overlay index properties of rocks and of discontinuities.

2.10 Classification of Rock Masses for Engineering Purposes

It is not always convenient to make a definitive test in support of engineering decision involving rock, and sometimes it is not even possible. Frequently, experience and judgment are strained in trying to find answers to design decisions involving rock qualities. Where there are particular and recurrent needs for quantitative values from rock, useful index tests are used routinely as in evaluating the need for continued grouting below a dam, deepening a pier shaft before filling it with concrete, or establishing the thickness of shotcrete lining in a newly excavated stretch of a rock tunnel. Thus it is not surprising that numerous schemes have been devised to guide judgment through standardized procedures and descriptions. Three especially well-received classification systems, originally advanced for tunneling, are those developed by Barton, Lien, and Lunde (1974), Bieniawski (1974, 1984), and Wickham, Tiedemann, and Skinner (1974).

Bieniawski's *Geomechanics Classification* system provides a general rock mass rating (RMR) increasing with rock quality from 0 to 100. It is based upon five universal parameters: strength of the rock, drill core quality, groundwater conditions, joint and fracture spacing, and joint characteristics. A sixth parameter, orientation of joints, is entered differently for specific application in tunneling, mining, and foundations. Increments of rock mass rating corresponding to each parameter are summed to determine RMR.

The *strength* of the rock can be evaluated using a laboratory compression test on prepared core, as discussed in the next chapter. But for rock classification purposes, it is satisfactory to determine compressive strength approximately using the point load test described previously on intact pieces of drill core. To simplify class boundaries, Bieniawski revised Equation 2.12 to $q_u = 25 I_S$. The rock mass rating increment corresponding to compressive strength values are listed in Table 2.10.

Drill core quality is rated according to the rock quality designation (RQD) introduced by Deere (1963). Although the RQD is widely used as a sole parameter for classification of rock quality, it is preferable to combine it with other parameters accounting for rock strength, joint character, and environmental factors as done here, since the RQD alone ignores these features. The RQD of a rock is evaluated by determining the percentage recovery of core in lengths greater than twice its diameter. The index was first applied solely to NX core, usually 2.125 in. in diameter, the percentage core recovery being modified to reject from the "recovered" category any fragments less than 4 in. in length. The rock mass rating increments corresponding to five bands of RQD values are given in Table 2.11.

The *spacing* of joints is also evaluated from drill core, if available. It is assumed that the rock mass contains three sets of joints in general and the spacing entered in Table 2.12 to determine the rating increment should reflect that joint set considered to be most critical for the particular application. If the

Table 2.10 *Rock Mass Rating Increments for Compressive Strength of the Rock*

Point Load Index (MPa)	OR	Unconfined Compressive Strength (MPa)	Rating
>10		>250	15
4–10		100–250	12
2–4		50–100	7
1–2		25–50	4
Don't use		10–25	2
Don't use		3–10	1
Don't use		<3	0

***Table 2.11** Rock Mass Rating Increments for Drill Core Quality*

RQD (%)	Rating
90–100	20
75–90	17
50–75	13
25–50	8
<25	3

rock mass has fewer sets of joints, the rating may be established more favorably than indicated in this table. The *condition* of joints is also examined with respect to the joint sets most likely to influence the work. In general, the descriptions of joint surface roughness and coating material should be weighted toward the smoothest and weakest joint set. Joint condition ratings are given in Table 2.13. Further discussion of the influence of joint roughness and spacing on the properties of rocks is presented in Chapter 5.

Groundwater can strongly influence rock mass behavior so the geomechanics classification attempts to include a groundwater rating term as given in Table 2.14. If an exploratory adit or pilot tunnel is available, measurements of water inflows or joint water pressures may be used to determine the rating increment directly. The drill core and drilling log can be used in lieu of such information to assign the rock to one of four categories from which the rating increment is assigned—completely dry, moist, water under moderate pressure, or severe water problems.

***Table 2.12** Increments of Rock Mass Rating for Spacing of Joints of Most Influential Set*

Joint Spacing (m)	Rating
>2.0	20
0.6–2.0	15
0.2–.6	10
0.06–0.2	8
<0.06	5

***Table 2.13** Rock Mass Rating Increments for Joint Condition*

Description	Rating
Very rough surfaces of limited extent; hard wall rock	30
Slightly rough surfaces; aperture less than 1 mm; hard wall rock	25
Slightly rough surfaces; aperture less than 1 mm; soft wall rock	20
Smooth surfaces, OR gouge filling 1–5 mm thick, OR aperture of 1–5 mm; joints extend more than several meters	10
Open joints filled with more than 5 mm of gouge, OR open more than 5 mm; joints extend more than several meters	0

Since the orientation of the joints relative to the work can have an influence on the behavior of the rock, Bieniawski recommended adjusting the sum of the first five rating numbers to account for favorable or unfavorable orientations, according to Table 2.15. No points are subtracted for very favorable orientations of joints, up to 12 points are deducted for unfavorable orientations of joints in tunnels, and up to 25 for unfavorable orientations in foundations. It is difficult to apply these corrections by universal charts because a given orienta-

***Table 2.14** Increments of Rock Mass Rating Due to Groundwater Condition*

Inflow per 10 m Tunnel Length (L/min)	OR	Joint Water Pressure Divided by Major Principal Stress	OR	General Condition	Rating
None		0		Completely dry	15
<10		0.0–0.1		Damp	10
10–25		0.1–0.2		Wet	7
25–125		0.2–0.5		Dripping	4
>125		>0.5		Flowing	0

Table 2.15 *Adjustment in RMR for Joint Orientations*

Assessment of Influence of Orientation on the Work	Rating Increment for Tunnels	Rating Increment for Foundations
Very favorable	0	0
Favorable	−2	−2
Fair	−5	−7
Unfavorable	−10	−15
Very unfavorable	−12	−25

tion may be favorable or unfavorable depending upon the groundwater and joint conditions. Thus, applying Table 2.14 requires advice from an engineering geologist familiar with the particular rock formations and the works in question. The orientation of joint sets cannot be found from normal, routine drilling of rock masses but can be determined from drill core with special tools or procedures, as reviewed by Goodman (1976) (work cited in Chapter 1). Logging of the borehole using a television or camera downhole will reveal orientations of joints, and absolute orientations will also be obtained from logging shafts and adits.

For applications in mining, involving assessments of caveability, drillability, blasting, and supports, Laubscher and Taylor (1976) modified Tables 2.10 to 2.15 and introduced factors to adjust for blasting practice, rock stress, and weathering. They also presented a table to find joint spacing ratings given the separate spacings of all joint sets. The overall RMR rating of a rock mass places the rock in one of the five categories defined in Table 2.16. Specific applications of the rock mass rating are presented in later chapters.

Table 2.16 *Geomechanics Classification of Rock Masses*

Class	Description of Rock Mass	RMR Sum of Rating Increments from Tables 2.9–2.14
I	Very good rock	81–100
II	Good rock	61–80
III	Fair rock	41–60
IV	Poor rock	21–40
V	Very poor rock	0–20

The Q system by Barton, Lien, and Lunde (1974) (also called the NGI system) combines six parameters in a multiplicative function:

$$Q = (\text{RQD}/J_n) \times (J_r/J_a) \times (J_w/\text{SRF}) \tag{2.16}$$

where RQD is the Rock Quality Designation
J_n relates to the number of joint sets
J_r relates to the roughness of the most important joints
J_a relates to the wall rock condition and/or filling material
J_w relates to the water flow characteristics of the rock
SRF relates to looseness and stress conditions.

The first term of Equation 2.16 is a measure of the sizes of joint blocks, the second factor expresses the shear strength of the block surfaces, and the last factor evaluates the important environmental conditions influencing the behavior of the rock mass. Numerical values are assigned to each parameter of the Q system according to detailed descriptions to be found in the article by Barton et al., which are abbreviated in Table 2.17. Table 2.18 assigns qualitative classes to the rock according to the overall value of Q.

The Q system and the RMR system include somewhat different parameters and therefore cannot be strictly correlated. Equation 2.17 is an approximate connecting relationship proposed by Bianiawski, based upon a study of a large number of case histories (standard deviation = 9.4).

$$\text{RMR} = 9 \log Q + 44 \tag{2.17}$$

Table 2.17 *Values of the Parameters in the Q System*

Number of Sets of Discontinuities	J_n
Massive	0.5
One set	2.0
Two sets	4.0
Three sets	9.0
Four or more sets	15.0
Crushed rock	20.0
Roughness of Discontinuities	J_r*
Noncontinuous joints	4.0
Rough, wavy	3.0
Smooth, wavy	2.0
Rough, planar	1.5
Smooth, planar	1.0
Slick, planar	0.5
"Filled" discontinuities	1.0
*Add 1.0 if mean joint spacing exceeds 3 m	

Filling and Wall Rock Alteration	J_a
Essentially unfilled	
Healed	0.75
Staining only; no alteration	1.0
Silty or sandy coatings	3.0
Clay coatings	4.0
Filled	
Sand or crushed rock filling	4.0
Stiff clay filling <5 mm thick	6.0
Soft clay filling <5 mm thick	8.0
Swelling clay filling <5 mm thick	12.0
Stiff clay filling >5 mm thick	10.0
Soft clay filling >5 mm thick	15.0
Swelling clay filling >5 mm thick	20.0
Water Conditions	J_w
Dry	1.0
Medium water inflow	0.66
Large inflow with unfilled joints	0.5
Large inflow with filled joints that wash out	0.33
High transient inflow	0.2–0.1
High continuous inflow	0.1–0.05
Stress Reduction Class	***SRF****
Loose rock with clay-filled discontinuities	10.0
Loose rock with open discontinuities	5.0
Rock at shallow depth (<50 m) with clay-filled discontinuities	2.5
Rock with tight, unfilled discontinuities under medium stress	1.0

* Barton et al. also define SRF values corresponding to degrees of bursting, squeezing, and swelling rock conditions.

The use of engineering classification systems for rock is still somewhat controversial. Proponents point to the opportunities they offer for empiricism in design of tunnels, mines, and other works in rock. Furthermore, an attempt to fill out the tables of values required by these schemes disciplines the observer and produces a careful, thorough scrutiny of the rock mass. On the other hand, these classifications tend to promote generalizations that in some cases are

Table 2.18 *After Barton, Lien, and Lunde (1974)*

Q	Rock Mass Quality for Tunneling
<0.01	Exceptionally poor
0.01– 0.1	Extremely poor
0.1 – 1.0	Very poor
1.0 – 4.0	Poor
4.0 – 10.0	Fair
10.0 – 40.0	Good
40.0 –100.0	Very good
100.0 –400.0	Extremely good
>400.0	Exceptionally good

inadequate to describe the full range of specifics of real rocks. Whichever argument prevails in a particular case, there can be no doubt that classification systems are proving valuable to many in various aspects of applied rock mechanics.

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