# PHYSICAL PROPERTIES OF ROCK

## 1. INTRODUCTION

Information collected by geologists and engineering geologists is in general not sufficient to predict the engineering behavior of rocks and rock masses. Tests need to be conducted to assess the response of rocks under a wide variety of disturbances such as static and dynamic loading, seepage and gravity and the effect of atmospheric conditions and applied temperatures. In general, rock and rock mass properties can be divided into five groups:

- physical properties (durability, hardness, porosity, etc.),
- mechanical properties (deformability, strength),
- hydraulic properties (permeability, storativity),
- thermal properties (thermal expansion, conductivity), and
- *in situ* stresses.

This second set of lecture notes focuses on physical properties such as weathering potential, slaking potential, swelling potential, hardness, abrasiveness, and other properties such as porosity, density, water content, etc. Most of those properties are intact rock properties.

## 2. WEATHERING AND SLAKING

When exposed to atmospheric conditions, rocks slowly break down. This process is called weathering and can be separated into *mechanical* (also called physical) weathering and *chemical* weathering. The principal types of mechanical and chemical weathering processes are listed in Table 1 (after Kehew, 1995).

### 2.1 Mechanical Weathering

Mechanical weathering causes disintegration of rocks into smaller pieces by exfoliation or decrepitation (slaking). The chemical composition of the parent rock is not or is only slightly altered. Mechanical weathering can result from the action of agents such as frost action, salt crystallization, temperature changes (freezing and thawing), moisture changes (cycles of wetting and drying), wind, glaciers, streams, unloading of rock masses (sheet jointing), and biogenic processes (plants, animals, etc.).

For instance, mechanical weathering is very active in high mountains with cold climates (see Figure 1). The 9% increase in volume associated with the transformation of water into ice as the temperature drops below 0°C can create pressures large enough to crack rocks. A good example of this type of process can be found in the Niagara Falls area where large blocks of dolomite detach from the rest of the rock mass in the Spring and Summer seasons.

Another example is the weathering associated with the natural unloading of massive granitic or sandstone rock masses associated with removal of overburden. As unloading takes place, discontinuities called *sheet joints* (also called *exfoliation joints* or *lift joints*) may develop

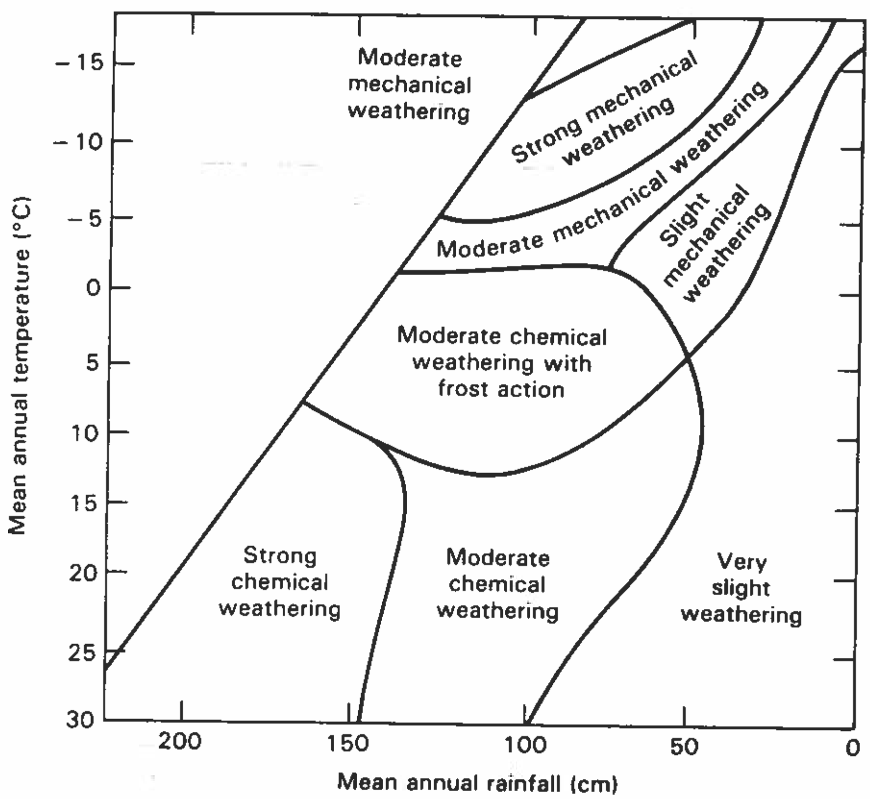



Figure 1. Climatic influences on types of weathering processes (after Kehew, 1995).

Table 1. Types of mechanical and chemical weathering processes (after Kehew, 1995).

Mechanical

- Frost action
- Salt weathering
- Temperature changes
- Moisture changes
- Unloading
- Biogenic processes

Chemical

Solution:

| | | | | | | |
|---|---|---|---|---|---|---|
| $CaCO_3$ | + | $H_2CO_3$ | $\rightarrow$ | $Ca^{2+}$ | + | $2HCO_3^-$ |
| (Calcite) | | (Carbonic acid) | | (Calcium ion) | | (Bicarbonate ion) |

Hydrolysis:

| | | | | | | |
|---|---|---|---|---|---|---|
| $2KAlSi_3O_8$ | $+ 2H^+ + 9H_2O \rightarrow$ | $Al_2Si_2O_5(OH)_4$ | + | $4H_4SiO_4$ | + | $2K^+$ |
| (Orthoclase) | | (Kaolinite) | | (Dissolved silica) | | (Dissolved potassium) |

Hydration:

| | | |
|---|---|---|
| $CaSO_4$ | $+ 2H_2O \rightarrow$ | $CaSO_4 \cdot 2H_2O$ |
| (Anhydrite) | | (Gypsum) |

Oxidation:

| | | | | |
|---|---|---|---|---|
| $4FeSiO_3$ | $+ O_2 + H_2O \rightarrow$ | $4FeO(OH)$ | + | $4SiO_2$ |
| (Pyroxene) | | (Limonite) | | (Dissolved silica) |

to the surface of rock outcrops. The rock outcrops appear to be spalling off like layers of a giant onion. The rock mass is divided onto blocks or sheets, a few centimeters thick near the ground surface and becoming thicker with depth until it fades out completely at depth of about 50 m. Stability problems can arise if these joints dip toward excavations with a potential for detachment of sheets. Sheeting tends to round the topography and create dome-shaped hills. Good examples of sheeting can be found at Yosemite, Zion and Stone Mountain National parks in the US. The "Sugarloaf" mountain near Rio de Janeiro in Brazil is another example. Unloading of rock masses in the form of rock bursts can be found in deep mines such as those in the Coeur d' Alene mining district in Idaho or in South Africa. Unloading can also be expressed as buckling of canal or quarry floors such as in the Niagara Falls area.

Shale and poorly cemented sandstones quickly disintegrate when exposed to natural conditions, and in particular moisture changes. Swelling or shrinking of the shale may occur if it contains such minerals as montmorillonite. Note that weathered shales are most susceptible to swell than non weathered types.

A last example of mechanical weathering is the one associated with the rapid cooling and heating of rocks in desert areas. Temperature gradients are large enough to crack rocks. On the Moon, meteorite impacts are also responsible for the weathering of basalt.

## 2.2 Chemical Weathering

This type of weathering creates new minerals in place of the ones it destroys in the parent rock. As rocks are exposed to atmospheric conditions at or near the ground surface, they react with components of the atmosphere to form new minerals. The most important atmospheric reactants are oxygen, carbon dioxide, and water. In polluted air, other reactants are available (acid rain problems associated with the release of sulfuric acid from coal-fired power plants, sulfur dioxide and smoke emissions, nitrogen oxides from vehicle exhaust). Table 1 gives a list of weathering reactions that have been recognized. In general, chemical weathering reaction are exothermic and cause volume increases.

*Solution* is a reaction whereby a mineral completely dissolves during weathering. This type of reaction depends on the solubility of the rock minerals. For instance, evaporite minerals (salt, gypsum) dissolve quickly in water, whereas carbonate minerals are somewhat less soluble. Limestone dissolves by meteoric water which contains dissolved carbon dioxide. This results in the formation of cavities called *dolines* or *karsts* and geologic hazards called *sink holes*. Because of impurities in the limestone, a red residual soil remains at the limestone surface called *terra rossa* (Sowers, 1975). Underground cavities can also be formed in gypsum because of its large solubility (Brune, 1965).

*Hydrolysis* is the reaction between acidic weathering solutions and many of the silicate minerals. Feldspars are transformed by hydrolysis as they react with hydrogen ions to form various products including clay minerals. This phenomenon is responsible for the degradation of granite and other plutonic rocks to a material that resembles more of a dense soil than a rock. The disintegrated granite called *grus*, *saprolite*, or *spheroidal granite* consists of rounded blocks surrounded by a mixture of detrital clays and resistant grains of quartz.

*Hydration* corresponds to the penetration of water into the lattice structure of minerals. A good example is the hydration of anhydrite into gypsum which is often accompanied with large volume increases and substantial swelling pressures (Brune, 1965).

*Oxidation* corresponds to the reaction of free oxygen with metallic elements. This reaction is familiar to everyone as rust. In an oxidation reaction, the iron atoms contained in the minerals lose one or more electrons each and then precipitate as different minerals or amorphous substances. An example of oxidation reaction is the transformation of pyrite ($FeS_2$) into iron hydroxides that liberates sulfuric acid. The sulfuric acid can also attack calcium carbonate to produce gypsum. This production of gypsum creates a local volume increase and possible attack of concrete (Grattan-Bellew and Eden, 1975). Oxidation of pyrite in mudstone can transform chlorite into smectite along the oxidation front. The increase in smectite is expected to be closely related to landslides.

**2.3 Importance of Weathering in Rock Engineering**

The two types of weathering mentioned above can take place simultaneously or one can be more important than the other depending on the climate, temperature variations and elevation. The composition of the weathered material depends of course on that of the parent rock. For instance, granitic rocks weather to a mixture of (kaolinite) clay, silt and sand whereas basic igneous rocks such as basalt give rise to (montmorillonite) clay soils only. In all cases, weathering gives rise to either transported or residual soils. The rate of weathering depends on the rock type and composition, the climate, the temperature and the elevation (see Goodman, 1993). Figure 1 shows the effect of temperature and rainfall on weathering.

The degree and pattern of weathering are among the most important factors to be determined in an on-site exploration. In general, as weathering takes place, the engineering properties of a rock change. Its porosity, permeability and deformability increase whereas its strength decreases. This detrimental changes can be critical to the suitability of a site for structures such as dams which require maximum strength and elasticity. Also, geologic hazards can be created in weathered areas such as block movement, or landslides. Table 2 gives a descriptive scheme for grading the degree of weathering of a rock mass.

Most rock engineering works involve rocks in various levels within the weathered zone. Engineers need to know the elevations and locations of structures, selecting the types of foundations and locating the materials with which to build them. Weathering controls the depth to the bedrock and the uniformity of this contact. An uneven bedrock and paleovalleys can create several foundation and underground engineering problems. Weathering may also lead to cracking in concrete due to alkali-aggregate reactions (Gillott, 1975) such as *alkali-silica* reactions

Table 2. Different degrees of rock weathering (from Johnson and DeGraff, 1988 )

| Term | Description | Grade |
|---|---|---|
| fresh | no visible sign of rock material; perhaps slight discoloration on major discontinuity surfaces | I |
| slightly weathered | discoloration indicates weathering of rock material and discontinuity surfaces; all the rock material may be discolored by weathering. | II |
| moderately weathered | less than half of the rock is decomposed and /or disintegrated to a soil; fresh or discolored rock is present either as a continuous framework or as corestones. | III |
| highly weathered | more than half of the rock is decomposed and/or disintegrated to a soil; fresh or discolored rock is present either as a discontinuous framework or as corestones. | IV |
| completely weathered | all rock material is decomposed and/or disintegrated to soil; the original mass structure is still largely intact. | V |
| residual soil | all rock material is converted to soil; the mass structure and material fabric are destroyed; there is a large change in volume, but the soil has not been significantly transported. | VI |

(reactions between cement alkalies and minerals such as opaline silica, chert, chalcedony, volcanic glasses) or *alkali-carbonate* reactions (reactions with certain types of argillaceous dolomitic limestones). The formation of karsts may create substantial field problems that require special treatments such as grouting.

The value of the compressional wave velocity can serve as an indicator of the degree of weathering. For instance, Dearman et al. (1978) have tabulated ranges of velocity for various degrees of weathering in granites and gneisses: fresh, 3050-5500 m/s; slightly weathered, 2500-4000 m/s; moderately weathered, 1500-3000 m/s; highly weathered, 1000-2000 m/s; completely weathered to residual soil, 500-1000 m/s. Note that an empirical upper limit for the velocity of 2000 m/s is often used in practice to define geologic materials that can be ripped without difficulty.

### 2.4 Slaking

Since rocks change properties with time, a problem of interest is to assess their weatherability or its inverse their durability. From an engineering stand point, we are interested in an index to describe the degree of rock alterability and relate the properties of the rock to that index. Such an index has been developed for clay-bearing rocks (shales, claystones, mudstones...) and is called the *slake durability index*.

The *slake durability test,* first proposed by Franklin and Chandra (1972), is a test intended to assess the resistance offered by a rock sample to weakening and disintegration when subject to one (or several) cycle(s) of drying and wetting. It is a standardized measurement of the weight loss of rock lumps when repeatedly rotated through an air water interface. The procedure has been standardized by the ISRM (Franklin, 1979) and the ASTM (ASTM D4644-87).

The slake durability test apparatus is shown in Figure 2. It consists of two drums 100 mm long and 140 mm in diameter, containing about 500g of rocks (10 lumps) in each drum. Sieve mesh forms the walls of the drums with openings of 2 mm. The drums rotate at a speed of 20 rpm for a period of 10 minutes in a water bath. The rock in the drums are subject to different cycles of wetting in the bath and drying in the oven.

Let D be the mass of the empty dry drum. The initial dry mass of rock plus drum is defined as A. After one cycle of wetting and drying, the new dry mass of the drum and the rock is B. The slake durability index $I_{d1}$ is the percent of rock retained and is equal to

$$I_{d1} = \frac{(B-D)}{(A-D)} . 100\% \tag{1}$$

The test is repeated a second time and C is the final dry mass of the drum and remaining rock. The slake durability index $I_{d2}$ is then equal to

Table 3. Slake Durability Classification (after Franklin and Chandra, 1972)

| Classification | Slake durability (%) |
|---|---|
| Very low | 0-25 |
| Low | 25-50 |
| Medium | 50-75 |
| High | 75-90 |
| Very high | 90-95 |
| Extremely high | 95-100 |

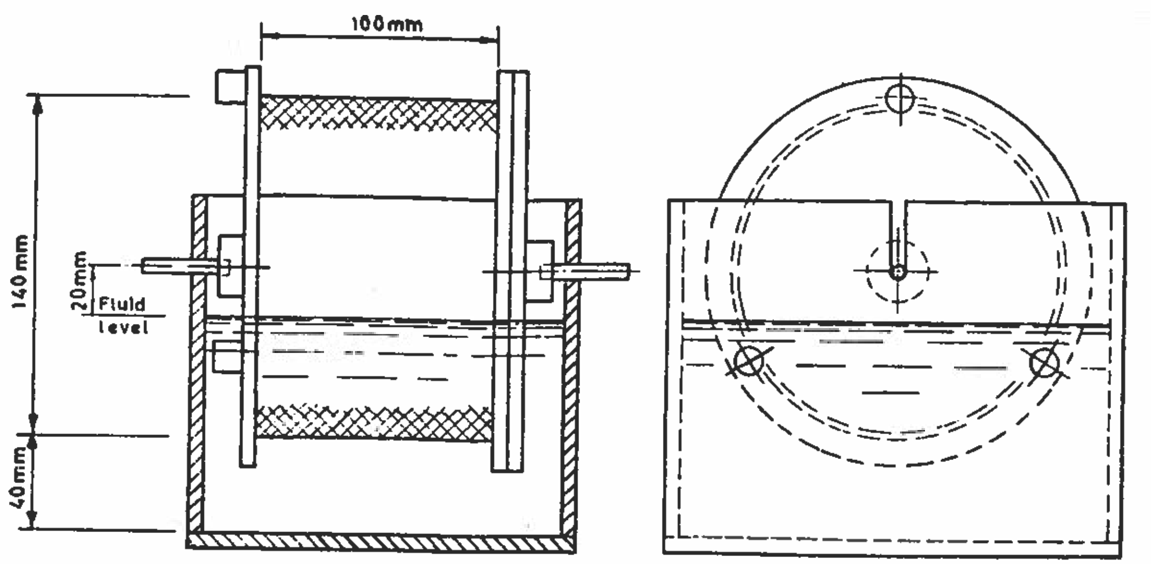



Figure 2. Slake Durability Test Equipment (after Franklin, 1979).

$$I_{d2} = \frac{(C-D)}{(A-D)}.100\% \tag{2}$$

Table 3 gives the slake durability classification suggested by Franklin and Chandra (1972) based on the value of $I_{d2}$. It is also recommended that the value of $I_{d1}$ be used whenever the values of $I_{d2}$ range between 0 and 10%,

Rocks giving low slake durability results should be subjected to soils classification tests such as Atterberg limits (Gamble, 1971). Morgenstern and Eigenbrod (1974) have shown that the liquid limit test in soil mechanics can be used to predict the maximum amount of slaking that can be expected for argillaceous rocks.

Note that several versions of the original slake durability test have been proposed in the literature. For instance, more cycles of drying and wetting may be necessary especially for rocks with higher durability. A comprehensive review and discussion of the different methods can be found in Richardson and Long (1987). They also proposed a test called the *Sieved Slake Durability Test* whereby the rock is passed through a series of graduated sieves after the conventional cycles of drying and wetting.

From a practical point of view, slaking of clay-bearing rocks requires protection of all outcrops. Shotcrete or any other form of protective layers are usually adequate.

## 3. SWELLING POTENTIAL

Chemical weathering reactions are usually accompanied with an increase in volume such as in the transformation of anhydrite into gypsum. For this reaction, increases in volumes ranging between 30 and 58% and swelling pressures as high as 10,000 psi (70 MPa) have been reported by Brune (1965) for anhydrite deposits in Texas. Heaving of structures founded on black shale in Canada has been observed as a result of the oxidation of pyrite and the formation of secondary sulfates such as gypsum (Quigley and Vogan, 1970; Grattan-Bellew and Eden, 1975). Swelling can also take place in clay bearing rocks containing such minerals as montmorillonite (Meehan et al., 1975). In Norway, Selmer-Olsen and Palmsrom (1989) reported several examples of tunnel collapse due to swelling clay gouge in faults and other rock discontinuities intersecting the tunnels. For the Pierre Shale of Colorado, Baker (1975) reported expansion averaging 2000 psf (0.1 MPa) with a maximum of 10,000 psf (0.5 MPa). Expansive soils and rocks do at least $ 1 billion a year in damage to U.S. homes more than the combined residential damage from floods, hurricanes, earthquakes and tornadoes (Jones and Jones, 1987).

The term swelling rock (or soil) implies not only the tendency of a material to increase in volume when water is available but also to decrease in volume and shrink if water is removed. Whether a soil or rock with high swelling potential will actually exhibit swelling characteristics depends on several factors: (1) the difference between the field moisture content at the time of construction and the final equilibrium, moisture content associated with the completed strcuture (2) the degree of compaction with more compaction favor swelling as moisture becomes available, (3) the final stress to which the material will be subjected after construction is complete.

Various tests have been proposed in the literature to determine the swelling potential of rocks, a review of which can be found in Einstein (1989). In 1994, the ISRM proposed some suggested methods for rapid field identification of swelling and slaking rocks (Einstein, 1994a).

The swelling potential can be assessed by conducting a swelling test with the geometry shown in Figure 3 (Franklin, 1979). The apparatus is essentially an oedometer with a rock specimen placed in a rigid ring. An initial vertical load is applied on the specimen. As water is added, the specimen swells and the vertical load is adjusted to maintain zero specimen swell. The swelling pressure is defined as the maximum swelling vertical force recorded during the test divided by the specimen cross-sectional area. Swelling strain or displacement can also be measured on unconfined specimens after immersion in water (as long as the specimens do not slake or disintegrate upon contact with water). Franklin (1984) also proposed the ring swell test to measure swelling or shrinking. This test allows to account for axisymmetric radial confinement and axial loading on swelling. Huang et al. (1986) proposed the moisture activity index as a measure of the swelling potential of shale.

Remedial actions to reduce the swelling potential of a rock can essentially be classified into two groups: (1) treating the rock (removal or control its water content or chemical treatment) and (2) design engineering structures to account for possible swelling or shrinking (bell shape piers, caissons, piles).

Various design philosophies have been suggested regarding structures built on or in swelling rocks. The ISRM published in 1994 a set of comments and recommendations on design and analysis procedures for *tunnels* and other *underground excavations* in argillaceous swelling rocks (Einstein, 1994b). Figure 4 shows the recommended design procedure. Three approaches were proposed: accommodate swelling in the design (*passive* approach), prevent it (*active* approach), or find an intermediate solution between passive and active.

- Swelling can be accommodated by passive (flexible) design which can be done in three ways. The rock is allowed to swell freely and is removed on a regular basis such that the structure is always useable. Another approach is to leave a void between the rock surface and an internal rigid structure. The third approach is to shape the excavated opening in such a way that the stress redistribution minimizes the effect of swelling pressures.

- Active design prevents or limits swelling and often results in a stress build-up on structures. Swelling can be reduced by counter-stresses (thick-walled curved liners, bolting, prestressing) and/or by limiting the access of water by drainage, sealing of exposed rock surfaces and grouting

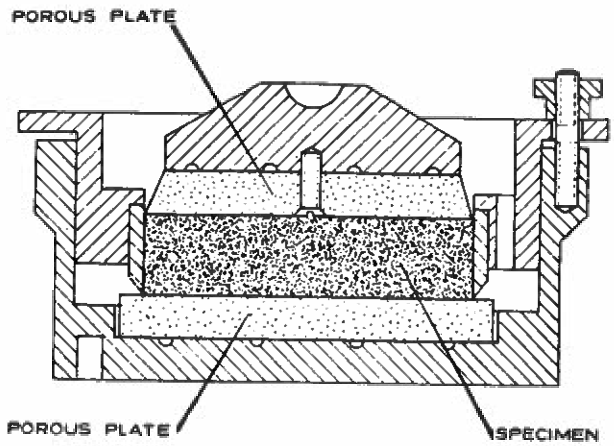
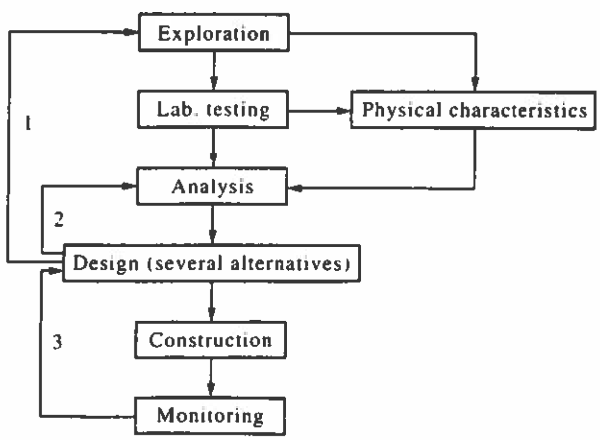



Figure 3. Confined Swelling Test Assembly (after Franklin, 1979).




Feedback cycles

1. Conduct additional exploration, laboratory testing and analysis based on design.
2. "Classic" cycle starting with conceptual design, and proceeding to initial and final design with associated increasingly detailed analyses.
3. Observational approach, i.e. select proper design alternative based on *in situ* observation (monitoring); possibly modify design. This feedback cycle may also involve a test structure.

Figure 4. Integrated Design Procedure Applied to Swelling Rock (after Einstein, 1994b).

- Intermediate design solutions include compressible support systems or support systems that can deform to a certain extent.

According to Chen (1988), three methods are available for reducing or avoiding the effect of swelling of soils and rocks on foundations. These include isolating the structure from the swelling materials, designing a structure that will remain undamaged in spite of swelling (rare approach), and elimination of the swelling altogether.

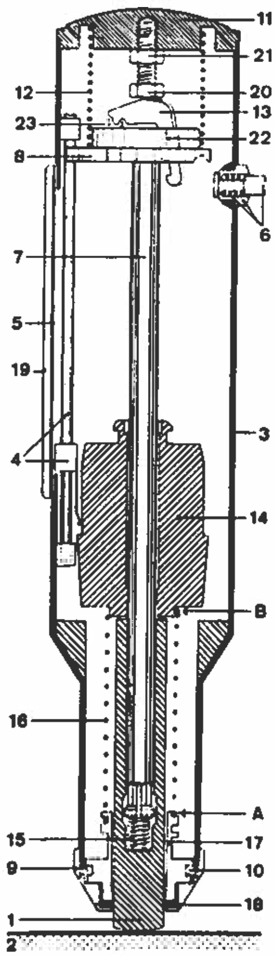
## 4. HARDNESS AND ABRASIVENESS

Knowledge of the hardness and abrasiveness of rock is very important when predicting rock drillability, cuttability, borability and tunnel boring machine advance rates. These two physical properties depend to a great extent on the mineralogical composition of the rock and the type and the degree of cementation of the mineral grains. Examples showing the importance of these properties in excavation engineering can be found in Selmer-Olsen and Blindheim (1970), Lachel (1973), Hansen and Lachel (1980), Aleman (1983), Nelson et al. (1983), Nelson et al. (1984), Howarth et al. (1986), Howarth (1987a), and West (1989) among others.

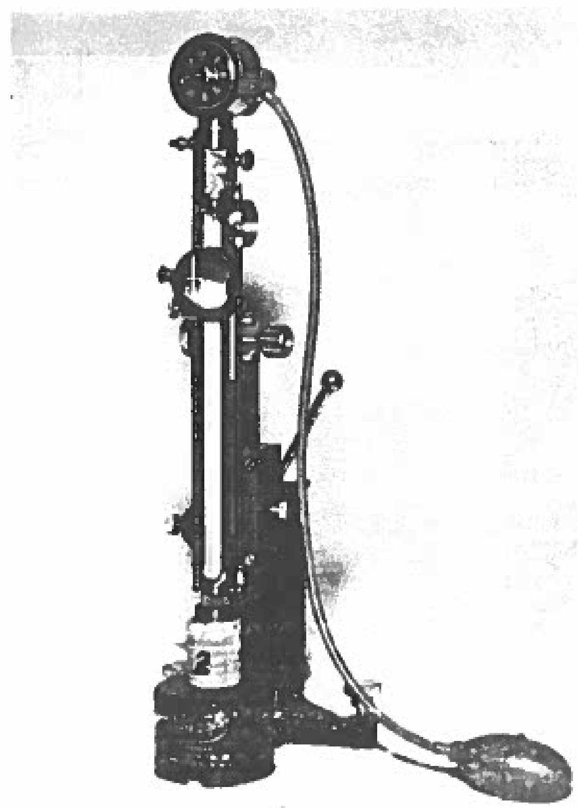
Rock hardness can be expressed using the Mohs scale used for minerals or can be measured (in a non-destructive way) using the *Schmidt Rebound Hammer* or the *Shore Scleroscope*. Suggested procedures for measuring intact rock hardness with those two devices can be found in Atkinson (1978). The techniques are simple, and the tests can be done rapidly and inexpensively.

The Schmidt Rebound Hammer, shown in Figure 5a, is used in rock mechanics (L-type) and is similar to that used to determine the strength of concrete (N-type). After calibration (on a material supplied by the manufacturer), the plunger (1) of the hammer is pressed against the rock surface (2). A spring driven mass (14) within the housing of the hammer (3) is released, strikes the plunger and rebounds, a pointer (4) on a scale (5) recording the amount of rebound as a percentage of the initial spring compression. The rebound number, also known as the *Schmidt Rebound Index*, R, is read by pressing a locking mechanism (6). In general, 10 readings are made. The value of R is higher for harder and stronger rocks which absorb less of the impact energy. Tests can be conducted in the laboratory on rock specimens or in the field on rock surfaces away from major discontinuities. In all cases, measurements must be made at right angles to the surfaces. The Schmidt hammer is calibrated for horizontal impact direction, i.e for testing vertical surfaces. When using it on horizontal or inclined surfaces, correction factors must be applied. Empirical equations have been suggested to relate R to the unconfined compressive strength. Figure 6a shows the relationship between R and the unconfined compressive strength for shale proposed by Hucka (1965). Figure 6b is a chart proposed by Deere and Miller (1966) where the strength is determined by multiplying the value of R by the dry density of the rock. Other empirical equations have been proposed by Aufmuth (1974), and by Irfan and Dearman (1978).

The Shore Scleroscope, shown in Figure 5b, is used to determine dynamic hardness where

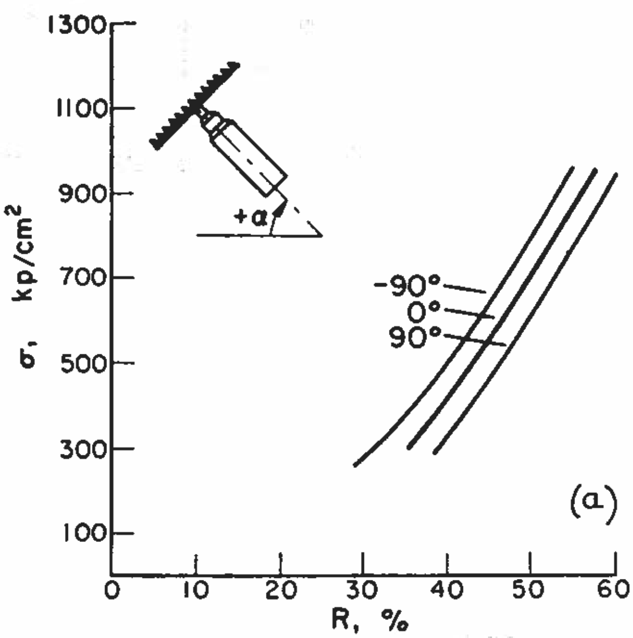



(a)



(b)

Figure 5 (a) Schmidt Rebound Hammer, (b) Shore Scleroscope.
(after Hucka, 1965).



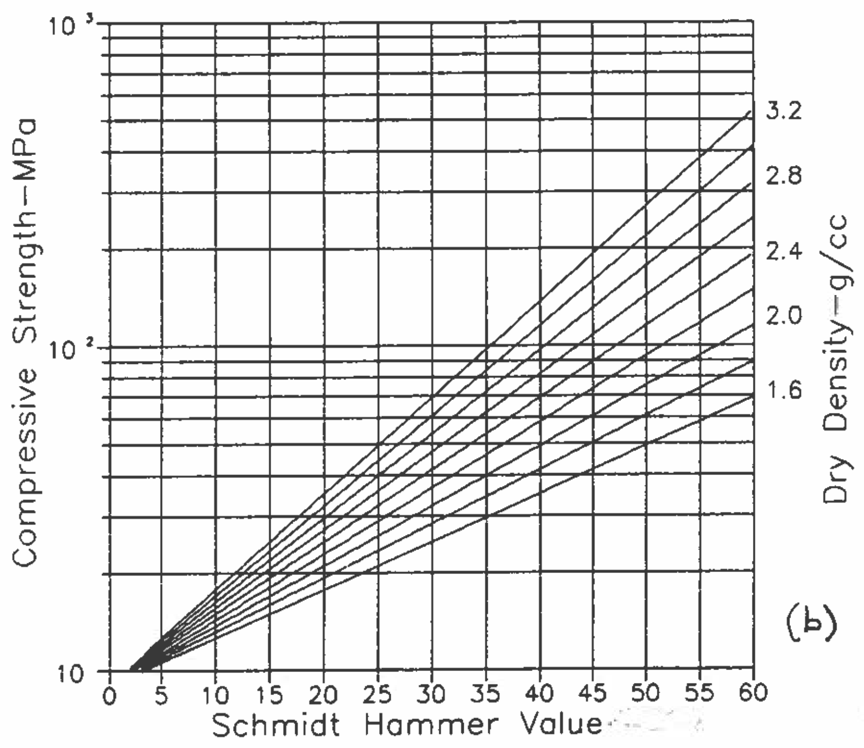




Figure 6. Relationship between R and unconfined compressive strength (a) for argillaceous shale (after Hucka, 1965), (b) as a function of dry density (after Deere and Miller, 1966).

indentation is done by a rapidly moving indenter. It measures the height of rebound of a diamond-tipping hammer falling freely on a horizontal planar surface from a height of about 10 inches. The hammer is contained within a close-bore glass tube. Air pressure, supplied by hand compression of a rubber bulb, operates a catch which releases the hammer. The bulb is connected by a rubber tube to a cylinder, containing a piston, at the top of the instrument. The vertical scent of the hammer after a test is effected by squeezing the bulb, the hammer again being suspended by the catch at the top. The height of rebound is read from the attached scale. The average of 10 readings is used to determine the value of the *Shore Scleroscope Index*, S. Empirical equations have been suggested to relate S to the unconfined compressive strength

Dietl and Tarkoy (1973) studied the relationship between rock hardness and the advance rate of a TBM in Manhattan schist. Several hardness indices were introduced. They found that the advance rate could be predicted by using a total hardness, $H_T$, which depends on the Schmidt Hammer Rebound Index $R=H_R$ and a so-called rock abrasion hardness $H_A$ (see Table 4). Figure 7 shows the variation of the TBM advance rate with $H_T$. The advance rate increases as the rock hardness decreases as expected.

The resistance to abrasion of aggregates can be obtained using the Los Angeles abrasion testing machine (Atkinson, 1978). A steel drum is loaded with about 5 kg of rock samples and a specified number of iron balls. The drum is rotated at 30-33 rpm for 500 revolutions. Abrasion, A, is expressed as a percentage of the original weight, or

$$A = \frac{W_o - W_r}{W_o} . 100\% \tag{3}$$

Selmer-Olsen and Blindheim (1970) also proposed a bit wear index to assess the abrasion capacity of different rock types on wolfram carbide bits. This index is obtained by using the wolfram carbide abrasion laboratory test. West (1981) discussed various testing methods to evaluate rock abrasiveness for tunneling applications and recommended the quartz content and the uniaxial compressive strength as useful rock properties. Tarkoy and Hendron (1975) also proposed a rock abrasiveness index for tunneling purpose.

## 5. DEGREE OF FISSURING

The degree of intact rock fissuring can be characterized through direct observation using the microscope. It can also be characterized through simple tests such as measurement of sonic velocity or permeability. Permeability will be discussed in another set of lecture notes.

The sonic velocity method (or pulse method) consists of propagating waves in intact samples of rock. Transmitters and receivers transducers and an oscilloscope are used to measure the time that longitudinal and transverse elastic waves propagate through an intact rock sample (see attached

| Hardness Test | Description | Typical Values | Example | Remarks |
|---|---|---|---|---|
| $H_R$ Schmidt (L-type) Hammer Rebound Hardness | 10 readings are taken; average of 5 highest values are used. | 10<br>~40<br>~63 | Friable standstone, Clay shale<br>Siltstone, Sandstone<br>Quartz-mica schist, Granite | Best for mass property measurements because contact point is large [about 1/2-in. (1.3-cm) diameter]. |
| $H_S$ Shore Scleroscope (C-2 type) Hardness | 20 readings are taken; average of 10 highest values are used. | 0<br>40<br>92 | Soft shales<br>Siltstone, Sandstone<br>Quartz-mica schist, Granite | Contact point is fine, therefore measurements are more accurate for individual grains and crystals, but statistical sampling must be taken and averaged for mass properties. |
| $H_A$ Rock Abrasion Hardness | 2 NX-size discs [1/4-in. (.6-cm) thick] abraded for 400 revolutions on each side; determine weight loss; use average values of 2 discs. $H_A$ = 1/wt loss in grams. | .4<br>.1<br>13 | Friable sandstone<br>Siltstone, Sandstone<br>Quartz-mica schist, Granite | This test is sensitive to factors that influence small-scale strength. |
| $H_T$ "Total" Hardness | Use appropriate average values to compute. $H_T = H_R\sqrt{H_A}$ | 1<br>40<br>230 | Friable sandstone<br>Siltstone, Sandstone<br>Fine-grained quartz-mica schist, Granite pegmatite | This combination of hardness has been successfully correlated with advance rates. |
| $H_C$ "Combined" Hardness | Use appropriate average values to compute. $H_C = H_S\sqrt{H_A}$ | 0 to 300 | Generally similar to the above | $H_C$ is yet to be evaluated for a range of rock types. |

Table 4. Outline of Hardness-Test Methods (after Dietl and Tarkoy, 1973).

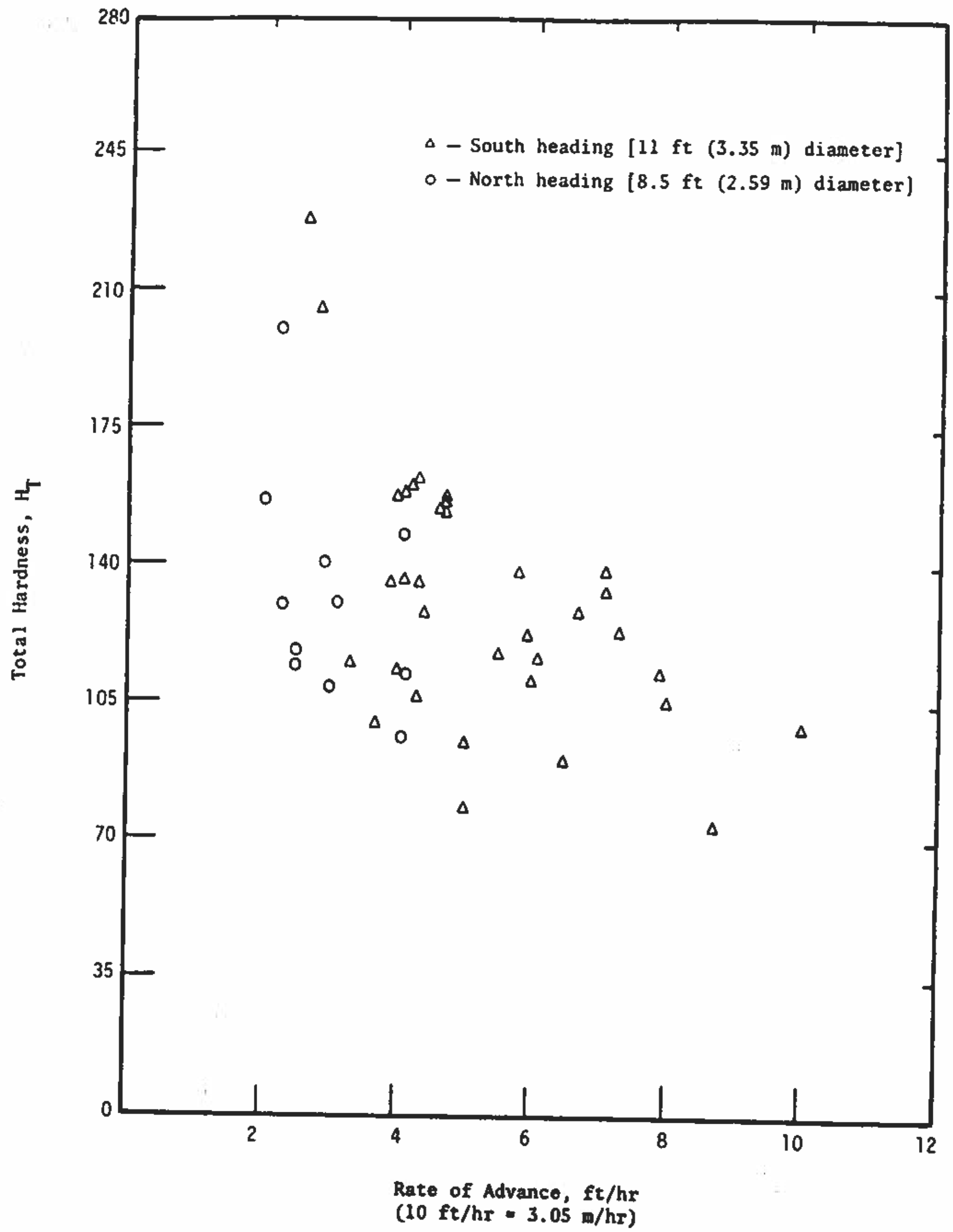



Figure 7. Rate of Advance of TBM in Manhattan Schist as a Function of Hardness (after Dietl and Tarkoy, 1973).

technical documentation, ASTM D2845-90 and Rummel and Van Heerden, 1978). As discussed by Goodman (section 2.8, 1989), if we know the mineral composition of the rock, the theoretical longitudinal velocity $V_l^*$ that the sample would have without fissures and pores can be written as

$$\frac{1}{V_l^*} = \sum_{i=1}^{N} \frac{C_i}{V_{il}} \tag{4}$$

where N is the number of mineral constituents in the rock, $V_{il}$ and $C_i$ are the theoretical longitudinal velocity and concentration of the ith mineral. Values of $V_{il}$ for some common minerals are given in Table 2.8 in Goodman (1989) and Christensen (1989).

The ratio between the measured value $V_l$ and the theoretical value $V_l^*$ can serve as an index to describe the degree of rock fissuring (Fourmaintraux, 1976), i.e

$$I_f = \frac{V_l}{V_l^*}.\ 100\% \tag{5}$$

The degree of fissuring is affected by the temperature (Houpert and Homand-Etienne, 1989).

## 6. PHASE RELATIONSHIPS

Rocks like soils are *three* phase materials. They consist of solid particles such as grains and crystals with void space in between. The void space can be occupied by air or water or both. As for soil, the components of a rock can be represented by a phase diagram (see Figure 8) and several parameters can be defined such as porosity, specific gravity, water content,degree of saturation and density. Tests to measure these properties are discussed in Franklin (1979).

### 6.1 Porosity

The porosity, n, (expressed in percent) is defined as follows

$$n = \frac{V_v}{V}.100\% \tag{6}$$

It represents the relative proportion of solid grains and voids in the rock. It is also a measure of the interconnected pore space. Note that the pore phase may not be completely continuous in a rock and fluid may not permeate to all the pores. The *apparent porosity* is the measure of the

- **High voltage pulse generator**
- **High sensitivity sensing heads**
- **Simultaneous measurement of compression and shear wave velocities**
- **Simple rapid test procedures**
- **Low cost**

Terrametrics sonic velocity equipment is designed to provide a simple low cost means for measuring compression and shear wave velocities in laboratory rock and soil samples. The equipment consists of a high powered pulse generator and sets of pulsing and sensing heads.

The measurement of sonic velocities in laboratory samples of rock, soil and concrete provides a simple means of calculating the dynamic elastic properties of these materials. These parameters can be correlated with static elastic constants, and strengths. If a good correlation can be demonstrated, which is often the case, the value of the sonic velocity measurements is further enhanced.

A complete sonic velocity equipment array is shown in figure 1. Practically any oscilloscope will do the job, the main requirement being a means of measuring along the time base so that time can be measured to an accuracy of 1%.

In use, the pulsing and sensing heads (which are identical and interchangeable) are attached to the flat ends of the specimen by means of molten phenol salicylate. A phenol salicylate dispensing kit is available as an accessory.

The heads are connected to the pulse generator and to the oscilloscope by means of connecting cables supplied with the equipment. The trigger pulse on the pulse generator is connected to the oscilloscope.

**TERRAMETRICS**
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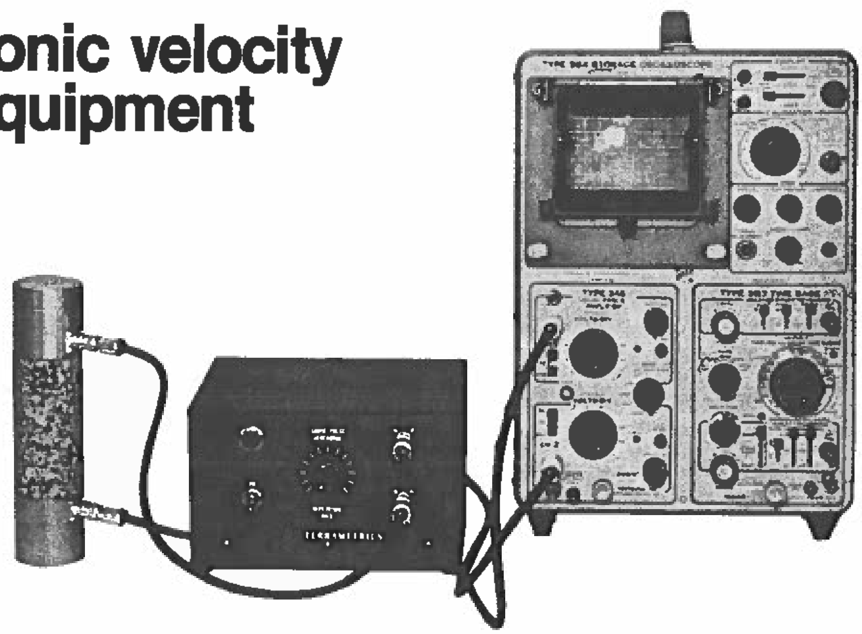
# sonic velocity equipment

## sonic velocity equipment

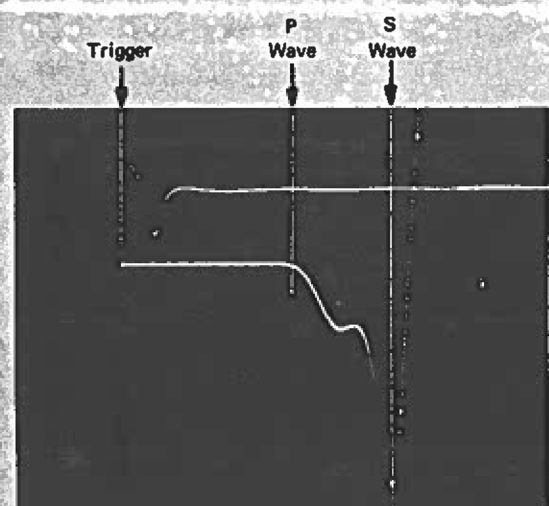


Figures 2 and 3 show typical oscilloscope traces from materials as diverse as granite ($E = 10 \times 10^6$) and oil shale ($E = 1 \times 10^6$). Note that both the shear wave velocity and the compression wave velocity can be obtained using only the shear wave heads. Thus, the compression wave arrival is marked by the first deviation of the trace, and the shear wave arrival by the first violent upward deflection of the trace.

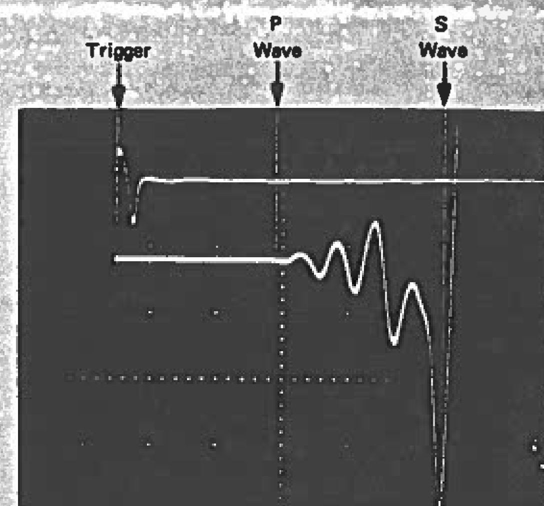
The pulse generator develops a 1000 volt pulse in the form of a rapid rise with exponential decay. The pulse repetition rate is adjustable between 20 to 150 repetitions per second. The pulse generator also generates a trigger pulse which is used to trigger the oscilloscope. Since the output voltage of the pulse generator is high, there is no need for amplifying either the input pulse to, or output pulses from, the transducer heads.

The pulsing and sensing heads contain piezoelectric crystals to generate either compressional or shear waves. The transducers are housed in magnesium, which protects them from stray electromagnectic pick-up, and mechanical damage. The use of magnesium improves the energy transmission across the face plate. The energy is further enhanced by pressurizing the crystal and by the use of phenol salicylate as a bonding agent between the head and the surface of the test sample.

Sonic velocity heads made of high strength steel, are available for use as platens to load the test specimen for investigation of the effects of uniaxial stress on the sonic velocities.



Actual Oscilloscope Traces of the Shear Wave Pulse (Granite)



Actual Oscilloscope Traces of the Shear Wave Pusle (Oil Shale)

## SPECIFICATIONS

| | |
|---|---|
| **Pulse Generator** | |
| Weight | 5-1/2 lbs |
| Size | 8 x 6-1/2 x 5 inches |
| Output Voltage | 1000V |
| Repetition Rate | 20 to 100 pulses per second |
| Power Requirement | 110–115 Vac, 50–60 Hz |
| **Transducer Heads** | |
| Size | 2" long x 2-1/8" dia. (NX) (other sizes available on request) |
| Compression Crystal | 800 KHz resonant frequency |
| Shear Wave Crystal | 630 KHz resonant frequency |
| **Connecting Cables** | Co-axial cables with MHV connectors |
| **Accessories** | Phenyl Salicylate dispensing kit |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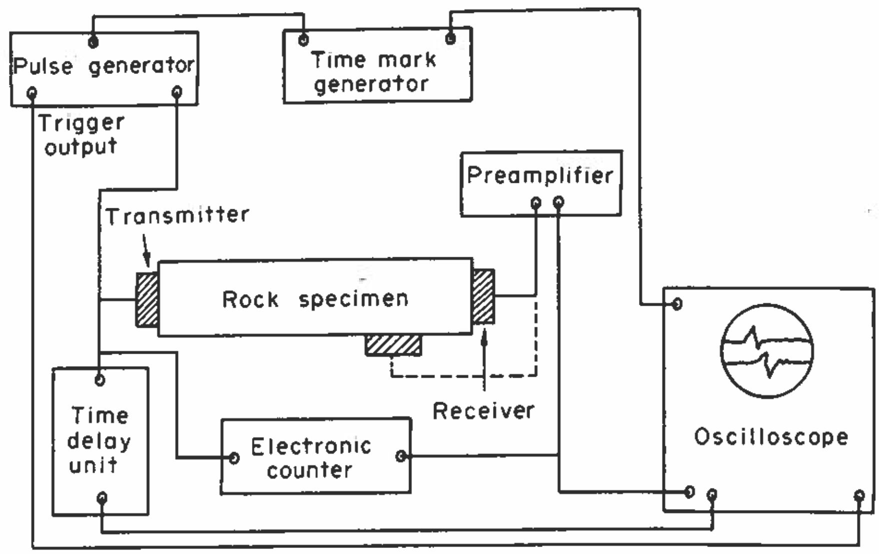
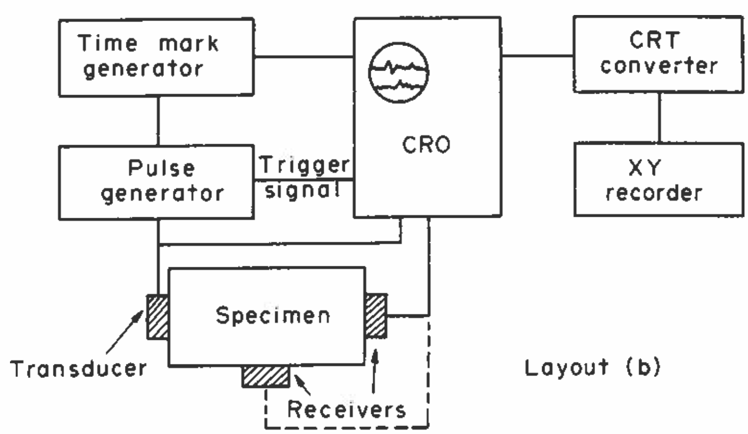



Layout (a)




Layout (b)

Two possible layouts of electronic components for
pulse generator method (after Rummel and van Heerden, 1978)

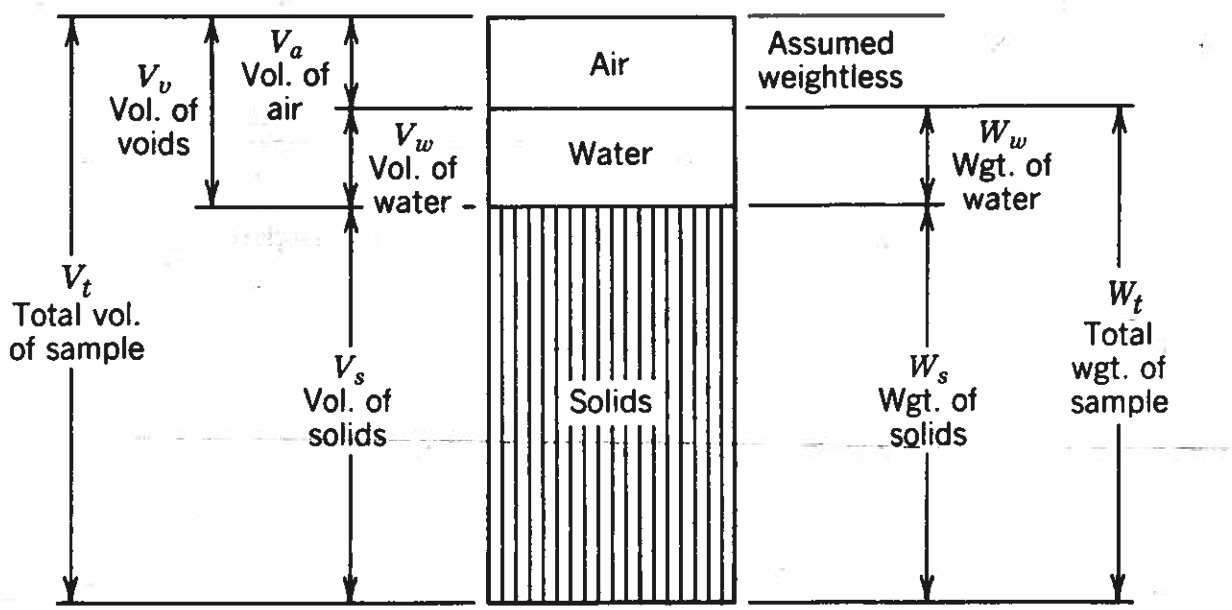



Figure 8. Phase Diagram Representing the Different Phases in a Rock. (after Johnson and DeGraff, 1988)

Table 5. Porosities for Different Rock Types (after Costa and Baker, 1981).

| Rock Type | Porosity % |
|---|---|
| Granite | 0.4-4.0 |
| Andesite | 0.1-11 |
| Gabbro, Diorite, Diabase | 0.1-1.0 |
| Basalt | 0.2-22 |
| Limestone | 0.2-4.4 |
| Sandstone | 1.6-26 |
| Chert | 4 |
| Gneiss | 0.3-2.2 |
| Marble | 0.3-2.1 |
| Quartzite | 0.3-0.5 |
| Slate | 0.1-1.0 |

volume of interconnected pores and cracks linked to the external surface of the rock. On the other hand, the *total porosity* is a measure of the volume of all the cracks and pores and includes those interconnected to the external surface and those having no connection to the external surface of the rock. Porosity values for different rock types are given in Table 5 and in Table 2.1 in Goodman (1989).

In general, cavities in an intact rock specimen can be classified into two groups: (1) cavities with more or less equal dimensions in all directions called *pores* (example: vugs in basalt formed by exsolution of gases),and (2) cavities that are elongated called *microfissures* (example: cavities at contact between grains in igneous and metamorphic rocks due to thermal or mechanical straining). For rocks with porosity less than two percent, microfissures are dominant. On the other hand, for rocks with porosity larger than two percent, pores are dominant  Porosity is very much affected by the rock texture, its age, depth, and the *in situ* state of stress. For instance, sedimentary rocks with clastic texture will be more porous than those with crystalline texture.

In general,the presence of microcavities in the fabric of a rock will influence its engineering properties. An increase in porosity is usually accompanied with an increase in deformability and permeability and a decrease in strength. The decrease in strength with an increase in porosity was observed by Howarth (1987b). He also found that drilling rate increases with rock porosity. Porosity increases with temperature (Houpert and Homand-Etienne, 1989).

### 6.2 Specific Gravity

The specific gravity of the *solid phase* of a rock, $G_s$, is defined as follows

$$G_s = \frac{\rho_s}{\rho_w} \tag{7}$$

where $\rho_s$ and $\rho_w$ are the density of the solid particles and water (at 20°C), respectively. The specific gravity can be measured directly or estimated once the mineralogical composition of the rock is known. As shown by Goodman (1989), the specific gravity for an aggregate of mineral grains i (i=1,N) can be expressed as follows

$$G_s = \sum_{i=1}^{N} G_{si} V_i \tag{8}$$

where $G_{si}$ and $V_i$ are the specific gravity and volume percentage of the ith mineral in the rock. Values of $G_{si}$ for most common minerals are given in Table 2.2 in Goodman (1989) and Olhoeft and Johnson (1989).

## 6.3 Water Content and Saturation

As for soils, the water content, w, and the degree of saturation, $S_r$, are equal to

$$w = \frac{M_w}{M_s} \tag{9}$$

and

$$S_r = \frac{V_w}{V_v} \tag{10}$$

## 6.4 Bulk Density

The bulk density, $\rho$, is equal to

$$\rho = \frac{(M_w + M_s)}{V} \tag{11}$$

For rocks, the bulk density varies between 2.5 and 3.0 g/cm$^3$. Values for different rock types can be found in Olhoeft and Johnson (1989). In general, low density rocks are highly porous. The dry density $\rho_d$ is the value of the bulk density when the rock is dry, i.e. $M_w=0$ ($S_r=0\%$). On the other hand, when the rock is saturated ($S_r=100\%$), the bulk density is defined as $\rho_{sat}$.

The following relationships exist between densities and other physical properties introduced before:

$$\rho_d = (1-n).\rho_s = (1-n).G_s.\rho_w \tag{12}$$

and

$$\rho_{sat} = \rho_d + n.\rho_w \tag{13}$$

If both sides of equations (12) and (13) are multiplied by the acceleration due to gravity, g, both equations can be expressed in terms of unit weights with $\gamma = g\rho$, $\gamma_w = g\rho_w$, $\gamma_d = g\rho_d$, $\gamma_{sat} = g\rho_{sat}$ and $\gamma_s = g\rho_s$.

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