

Effect of Moisture on Strength of Intact Rocks and the Role of Effective Stress Principle

by

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Introduction

A knowledge of the mechanical properties of rock material is essential in solving any problem of rock mechanics. The factors affecting the strength of intact laboratory specimens are summarised in Fig. 1. It is common practice to select a consistent set of test conditions so that the combined effects of external factors do not vary and hence the measured results are indicative of the material properties. Chamberlain et. al. (1976) identified four factors which are critical in influencing the observed rock properties, viz., moisture content, orientation with respect to rock fabric, fracture occurrence and specimen tolerances. The effect of moisture and pore fluid on rock strength is the subject of the present study.

During sampling and preparation of a test specimens, it is likely that it encounters water or any other drilling oil/lubricating oil. This is a

MECHANICAL FACTORS	MINERALOGICAL FACTORS	ENVIRONMENTAL FACTORS
Sampling technique	Mineralogy	Confining conditions
Specimen geometry and size	Type of cementing material	Water content Relative humidity
End-contact conditions	Amount of cementing material	Temperature Nature of pore fluid
End restraint	Grain size	pH,
Type of testing machine	Fabric, Structure Porosity/Density	Dielectric constant

FIGURE 1 Factors Affecting Strength of Intact Rocks

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(This paper was received in June, 1985 and is open for discussion till the end of March, 1986)

phenomenon that cannot be controlled easily. Samples are usually tested either after air drying or vacuum drying or as per ISRM recommendations of curing at a relative humidity of 50 percent at $20 \pm 2^\circ\text{C}$. Although air dry testing may be convenient in the laboratory, it may not be a realistic proposition for determining the properties of rock in underground workings. With the exception of the ones with evaporite minerals, most underground rocks are comparatively wet. However, the percentage moisture content for in-situ rock is difficult to ascertain because of evaporation from or condensation on underground rock surfaces or changes in circulation of underground water caused by mining. In addition, water is often used in coring or cutting rock specimens. Further, some slates and rocks containing clay will disintegrate if allowed to dry. In fact, it has been stated in ASTM Test (D 2664-67) for Triaxial Compressive Strength of undrained rock core specimens, without pore pressure that "the field moisture condition of the specimen should be preserved until time of test.....or should be tailored to the problem at hand."

A number of studies have shown that rock is weaker if tested "wet" rather than "dry". There appears to be no generally accepted explanation for this influence of moisture or for that matter presence of any other pore fluid on rock strength, although several mechanisms are often proposed to explain the same. In this paper, strength changes in rocks due to pore fluid/moisture changes are considered in detail and are explained on the basis of a unified mechanism through an effective stress concept.

Influence of Moisture and Pore Fluid Type on Rock Strength

Effect of moisture

Most underground rocks contain moisture ranging from less than 1 percent for some of the evaporite minerals to over 35 percent for porous sandstone. In the majority of mines and underground openings, rocks are nearly saturated. In the following, the effect of this moisture variation on compressive strength, tensile strength and shear strength and modulus of deformation is studied firstly by comparing the extreme conditions of 'wet' and 'dry' and later the intermediate moisture conditions.

Several researchers compared the uniaxial compressive strength of several rock types under oven-dry, air-dry and wet conditions (Venkatappa Rao et al 1983). Analysis of their data clearly reveals that for all rock types; the strength is highest in the oven-dry conditions followed closely by air dry conditions and strength in wet condition is significantly lower. This decrease is the highest in the case of sedimentary rocks, and least in case of most metamorphic and igneous rocks.

Analysis of the available data on the Brazilian strength of different rock types obtained by various researchers (Venkatatappa Rao et al 1983) shows that in all cases the strength reduces on saturation or at a relative humidity of 100 percent. Colback and Wiid (1965) also conducted triaxial compression tests on cylindrical specimens of a quartzitic shale, the results of which are presented in Figs. 2 and 3. They clearly indicate the reduction in strength values/parameters on submergence. Results of Mann and Fatt (1960) show that the modulus of elasticity was 8 to 20 percent less for a wet sandstone. Wiid (1970) obtained reduced values of

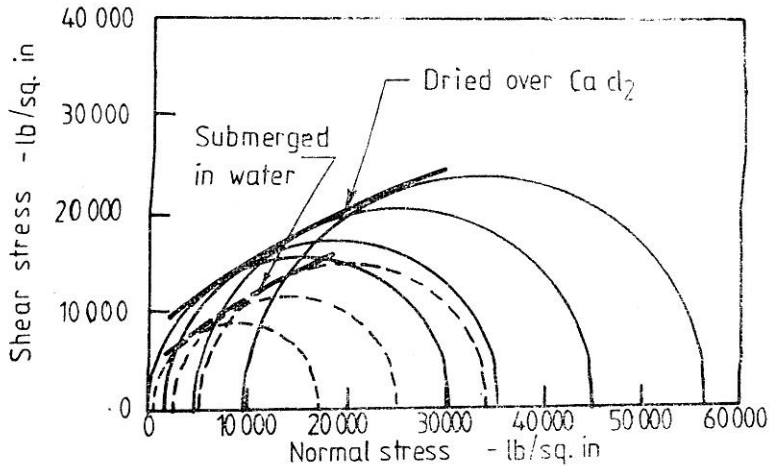


FIGURE 2 Mohr Envelopes for Quartzitic Shale at two Moisture Contents (After Colback and Wiid, 1965)

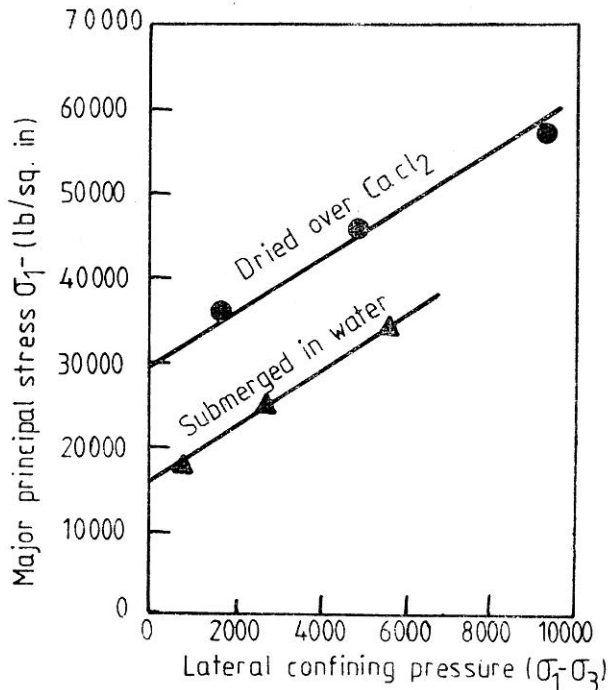


FIGURE 3 Relationship Between Axial Stress and Confining Pressure at Failure for Quartzitic Shale (After Colback and Wiid, 1965)

modulus of elasticity for two sandstones and dolerite in a saturated condition.

At a confining pressure of 1.56 kb the difference in triaxial strength for westerly granite (porosity 0.007) and Maryland diabase (porosity zero)

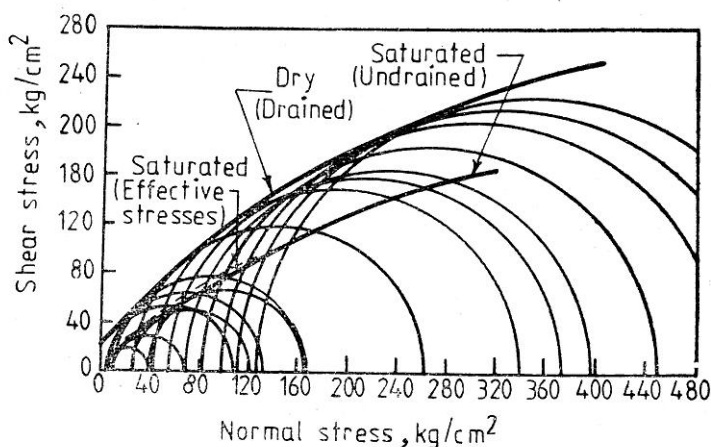


FIGURE 4 Mohr Rupture Envelopes for Dry and Saturated Siwalik Sandstone (After Ramamurthy and Goel, 1973)

between water-saturated and initially dry samples cured at 40 percent relative humidity was about 1.1, the strength of the latter samples being greater than the saturated ones at a given strain rate (Brace and Martin, 1968).

Ramamurthy and Goel (1973) studied the behaviour of a weak sandstone of Siwalik formations (India) in triaxial compression under confining pressures upto 127 kg/cm^2 on dry and saturated specimens. The deformation modulus, as shown in Fig. 4, decreased upon saturation and increased linearly with confining pressure for both dry and saturated specimens. The Mohr rupture envelopes shown in Fig. 5, clearly reveal a reduction in strength/strength parameters on saturation.

Ballivy et al (1976) have studied the effect of water saturation history on the strength of a gneiss, cemented sandstone, and a fine-grained limestone, all three rocks having porosities below 2 percent. The results reveal that proper saturation results in a decrease of triaxial and tensile strength.

Colback and Wiid (1967) were the foremost among researchers to systematically study the variation of triaxial compressive strength with moisture. Specimens of quartzitic shale and quartzitic sandstone were cured at different relative humidities for periods ranging upto 120 days, till equilibrium is attained. Their results are reproduced in Figs. 6 and 7. Wiid (1967) conducted further studies to include four sandstones and a quartzite. The quartzite had a porosity of 1 percent while the sandstones had porosities of 20 percent or more. The lowering of uniaxial compressive strength observed for these samples is presented in Fig. 8. He also found that the apparent tensile strength reduction in the Brazilian test for sandstones 1005, 1013 shown in Fig. 9 was very similar to the trends shown in Fig. 8. This seems reasonable, since a vertical splitting type failure in uniaxial compression might be similar to failure in a Brazilian test (van Eeckhout 1976).

Van Eeckhout (1976) studied the behaviour of a coal mine shale in three test modes—uniaxial compression, Brazilian and 3-point bending, at

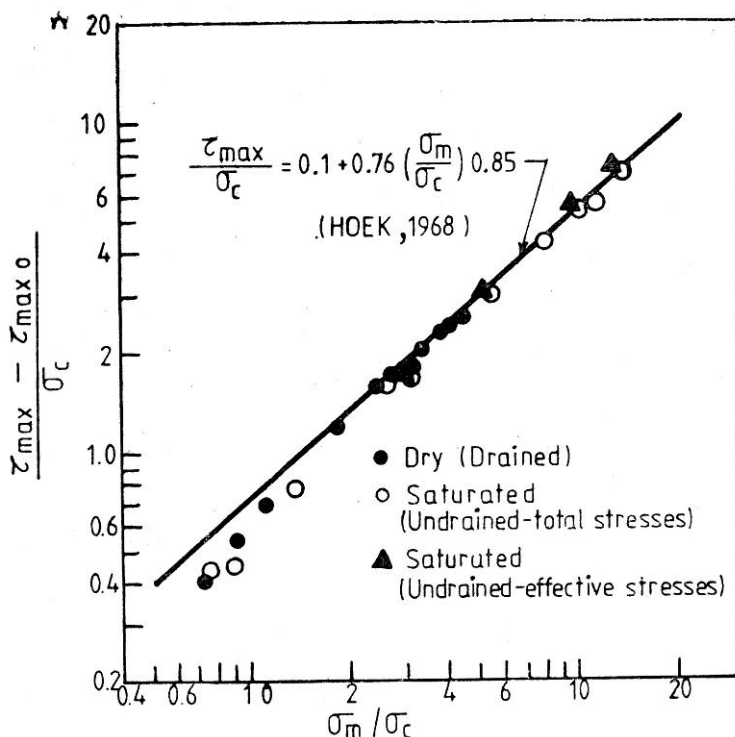


FIGURE 5 Failure Criterion for Siwalik Sandstone (After Ramamurthy and Goel, 1973)

different relative humidities. When strength lowering for all tests was compared, as shown in Fig. 10, no clear difference between test mode strength trends was noted. From this it was concluded that,

- (i) the mechanism of fracture and failure at peak load is affected equally by moisture, for all the test modes used, and
- (ii) since the compression test samples were observed to fail by vertical splitting, it appears likely that failure in all the three test modes was initiated under similar conditions, i.e. in tension.

Chamberlain et al (1976) presented van Eeckhout's (1974) data wherein the strength and modulus of elasticity of three shales are studied at three relative humidities. These data reveal that the strength as well as the modulus have registered a clear decrease with increased moisture.

Extensive tests on different rocks by Broch (1974) presented in Fig. 11 also indicate that the variation of point load strength index with relative humidity i.e., water content, is on the same lines as that of the compressive strength.

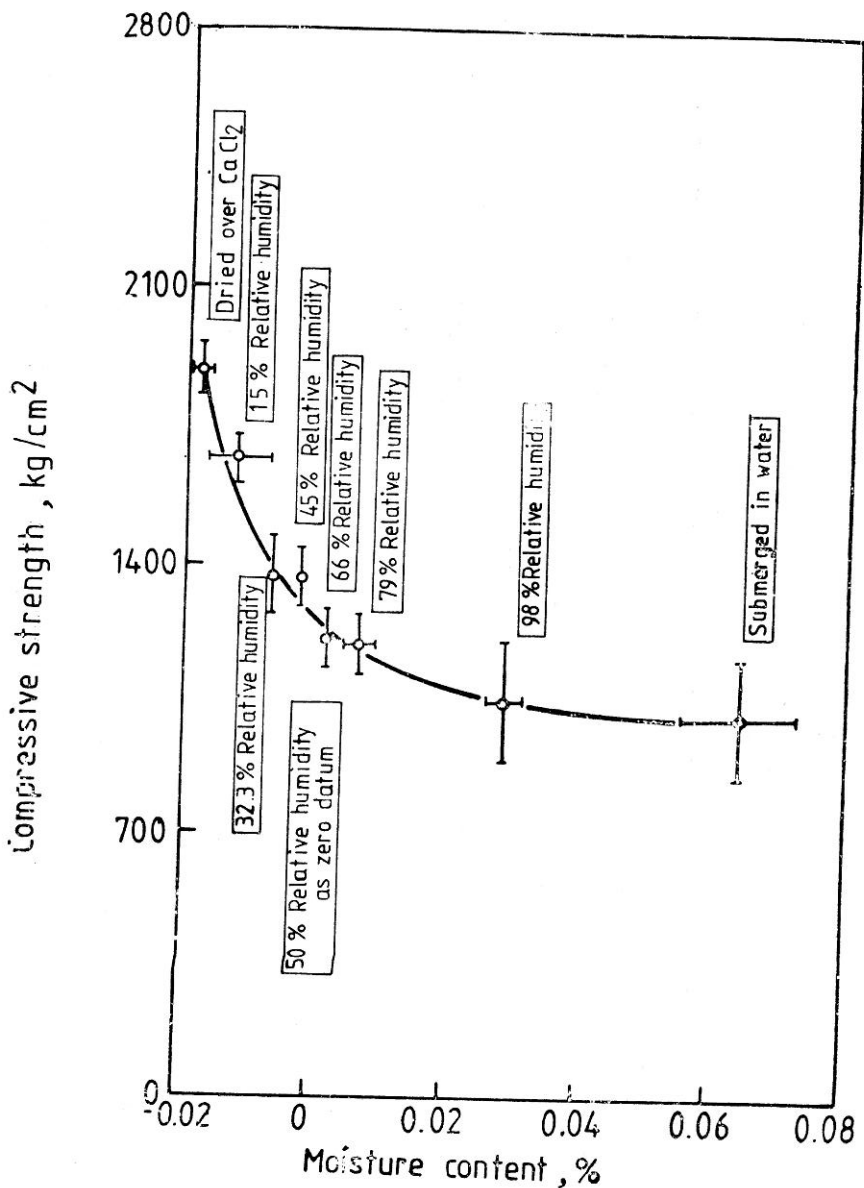


FIGURE 6 Relationship Between Compressive Strength and Moisture Content for Quartzitic Shale (After Colback and Wiid, 1965)

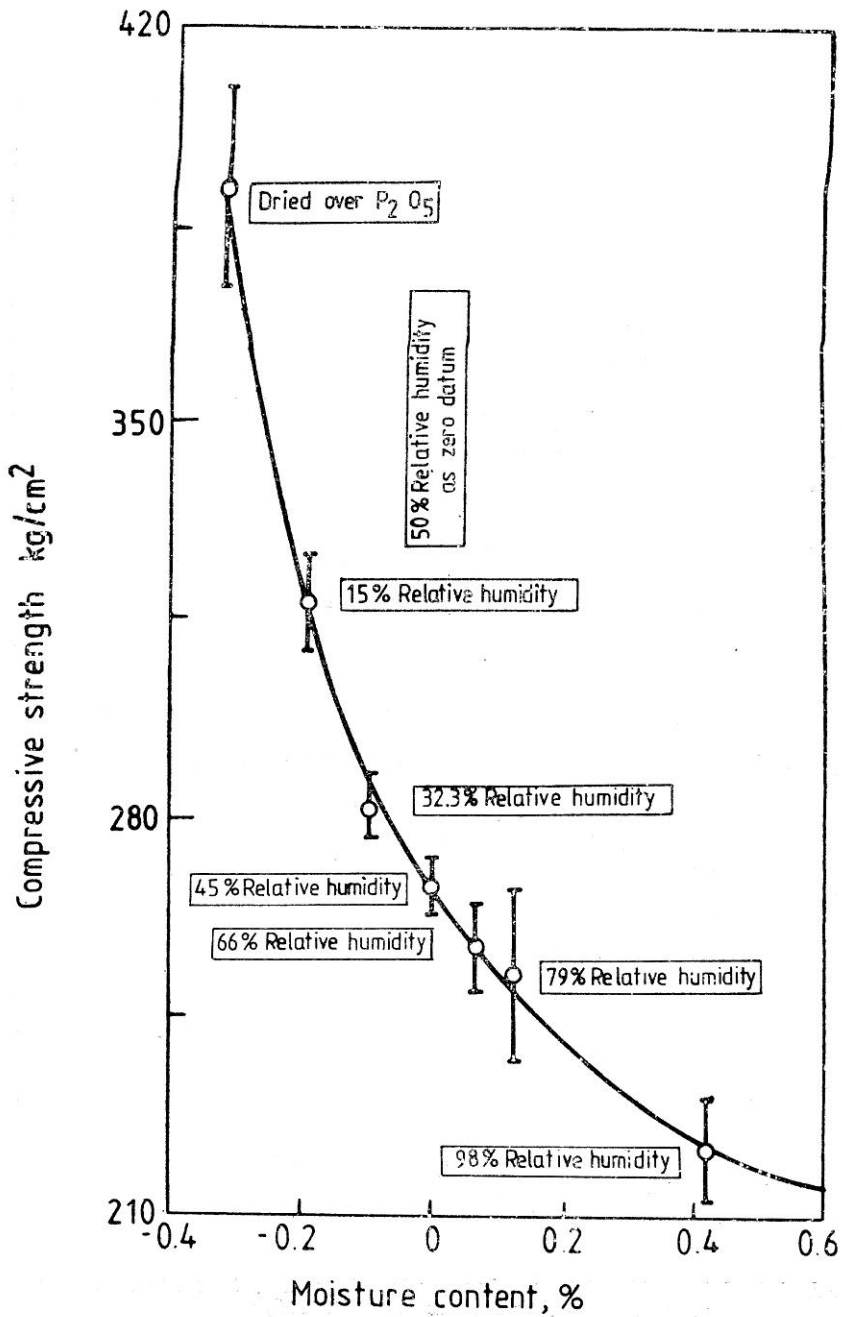


FIGURE 7 Relationship Between Compressive Strength and Moisture Content for Quartzitic Sandstone (After Colback and Wiid, 1965)

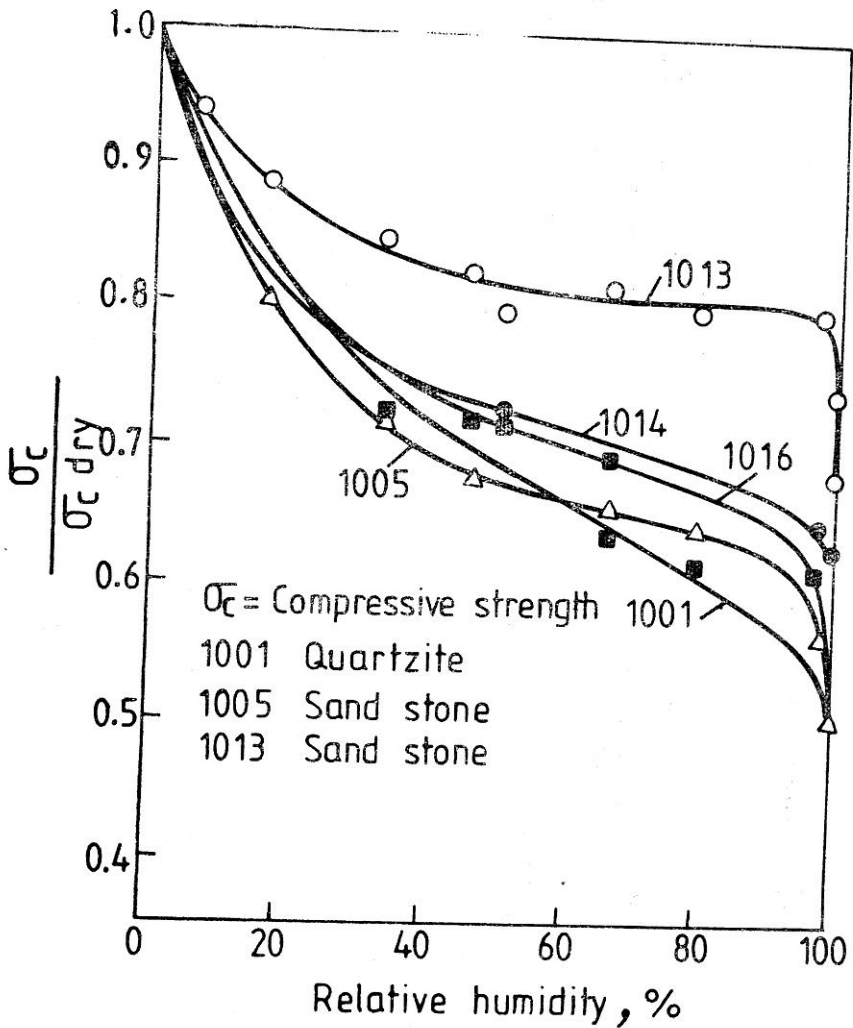


FIGURE 8 Variation of Uniaxial Compressive Strength With Relative Humidity (After Wiid, 1967)

Dube and Singh (1972) brought out the effect of moisture on Brazilian strength by conducting tests on five varieties of sandstones (with porosities ranging from 0.72—3.17) at relative humidities ranging from 0-100 percent. The results given in Fig. 12 along with those of Wiid (1967) on a sandstone show a clear reduction in Brazilian strength with relative humidity.

Effect of pore fluid type

Limited investigations have been reported on the effect of the type of pore fluid on the strength of rocks.

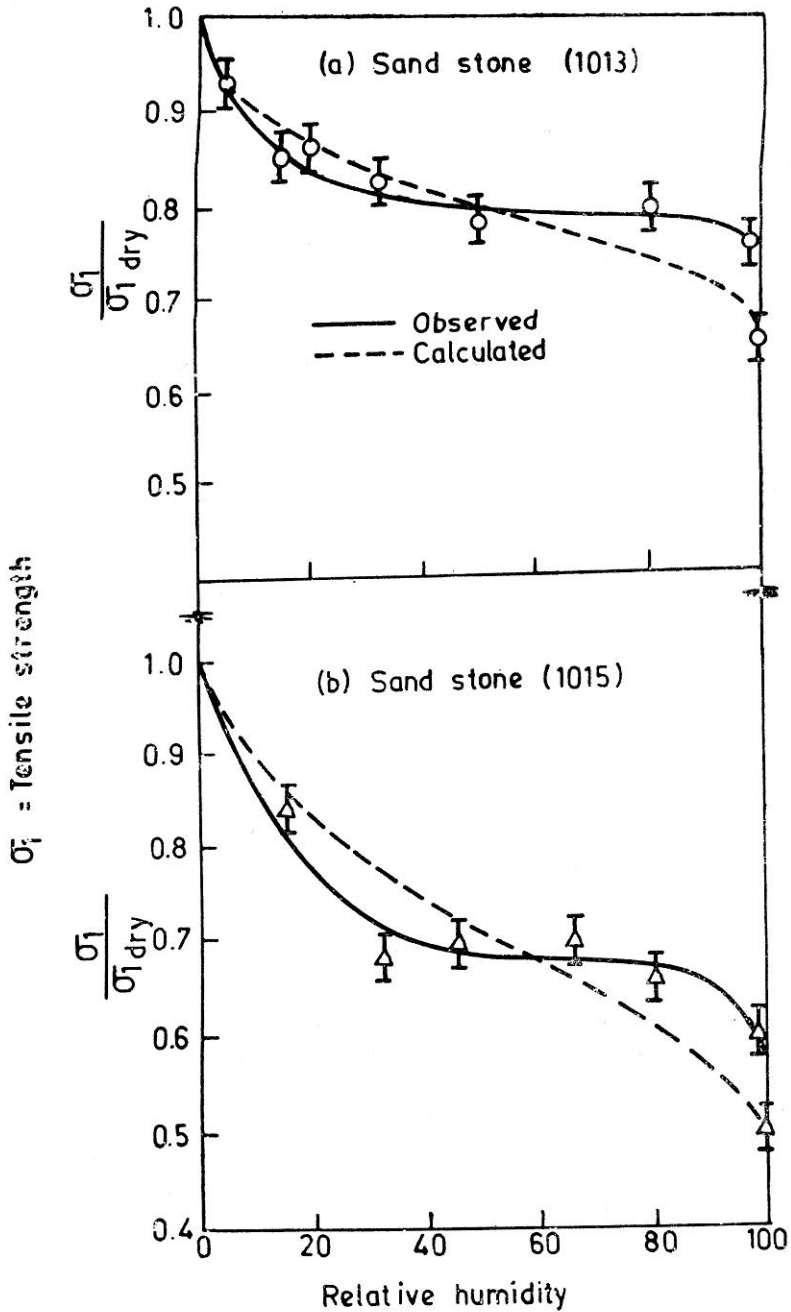


FIGURE 9 Comparison between Observed and Calculated Tensile Strength Reduction (After Wiid, 1967)

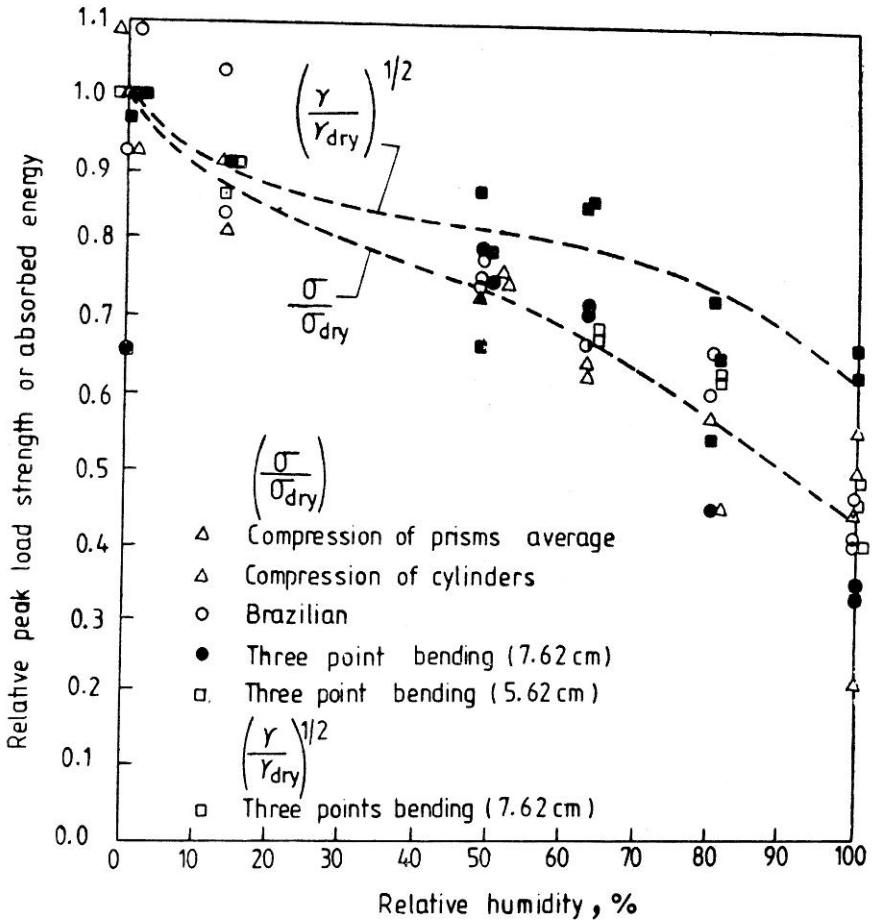


FIGURE 10 Variation of Strength and Adsorbed Energy with Relative Humidity (After van Eeckhout, 1976)

Working with different organic pore fluids, Colback and Wiid (1965) have reported (Fig. 13), the influence of surface tension of the fluid on the uniaxial compressive strength of quartzitic sandstone. They find a linear relation between compressive strength and surface tension—increasing the surface tension of the pore fluid reduces strength.

Vutukuri (1974) reported that the dielectric constant/surface tension of the pore fluid influenced the tensile strength obtained through a ring test. (Figs. 14 and 15).

Inter-particle Forces and Effective Stress Principle

During the last two decades molecular properties and intermolecular forces have been used to quantify many physical properties, even though experimental data was not available for some materials. The molecular approach gives a complete description of the bulk behaviour in terms of a

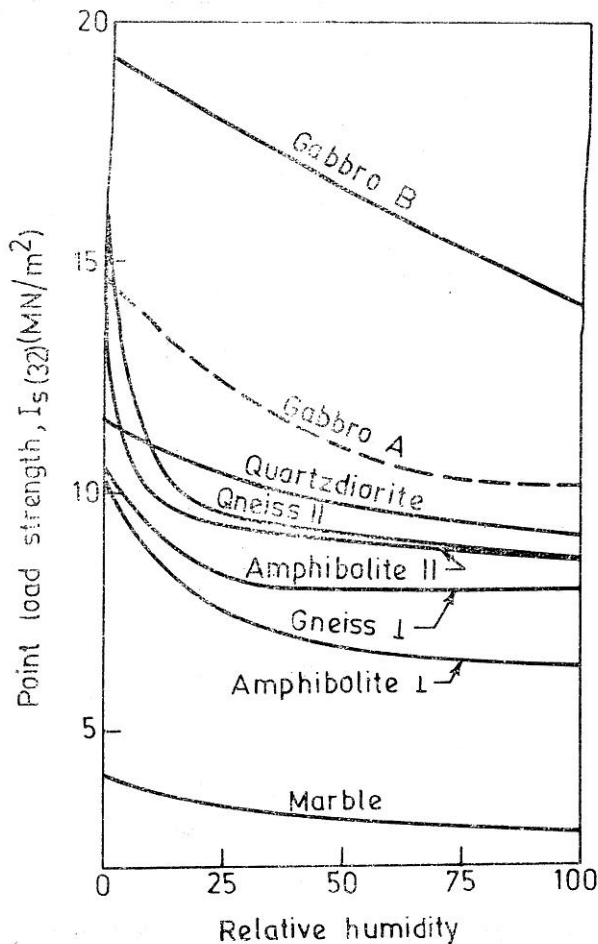


FIGURE 11 Variation of Strength With Water Content (After Broch, 1974)

small number of constants, characteristic of the material. Such an approach has been found to be quite successful in understanding the various facets of the engineering behaviour of soils, (Rosenquist 1955, Lambe 1958, 1960, Sridharan 1968, Venkatappa Rao 1972, Rekhi 1979). Lambe (1960) and Sridharan (1968) presented an effective stress concept which includes the interparticle forces of attraction and repulsion. This approach has been found to be quite valuable in understanding the complex engineering behaviour of soils (Sridharan and Venkatappa Rao, 1973, 1979, Venkatappa Rao and Rekhi 1979). Therefore, it appears worth-while to attempt to understand the engineering behaviour of rocks using similar approaches.

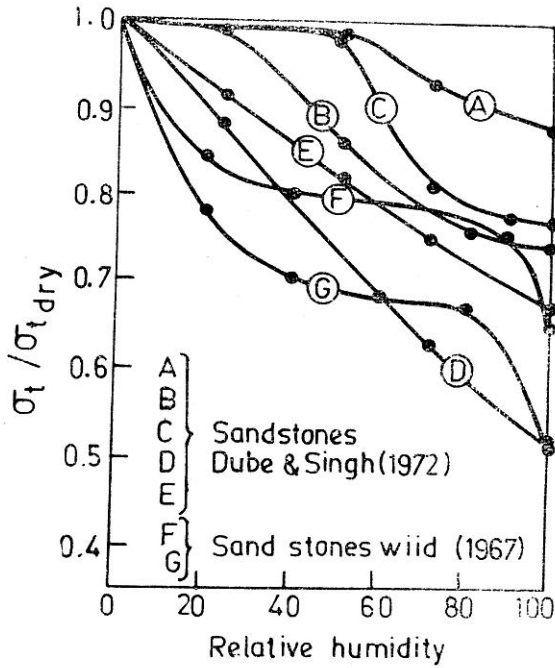


FIGURE 12 Variation of Brazilian Strength With Relative Humidity

Effective Stress Principle

The development of the concept of an effective stress in soils was by far the most significant contribution towards understanding the strength and deformation behaviour of soils. The principle as enunciated by Terzaghi (1936) for saturated soils has been generalised by various researchers to include the behaviour of partially saturated materials.

Sridharan (1968) proposed that the effective stress equation should take the following form for partially saturated soils:

$$\bar{\sigma}_c = \bar{\sigma} am = \sigma - \bar{u}_w - \bar{u}_a - R + A \quad \dots(1)$$

where

$\bar{\sigma}_c$ = effective contact stress

$\bar{\sigma}$ = mineral to mineral contact stress,

am = mineral to mineral unit contact area,

σ = externally applied pressure on a unit area,

\bar{u}_w = effective pore water pressure,

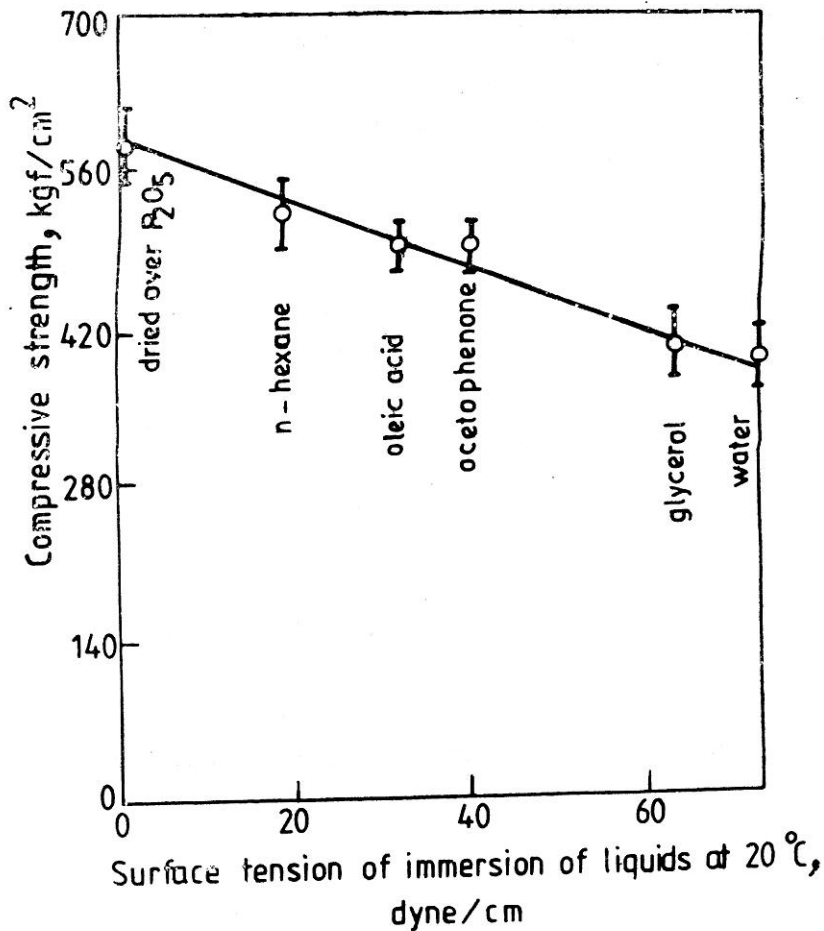


FIGURE 13 Influence of Surface Tension on Compressive Strength of Quartzitic Sandstone (After Colback and Wiid, 1965)

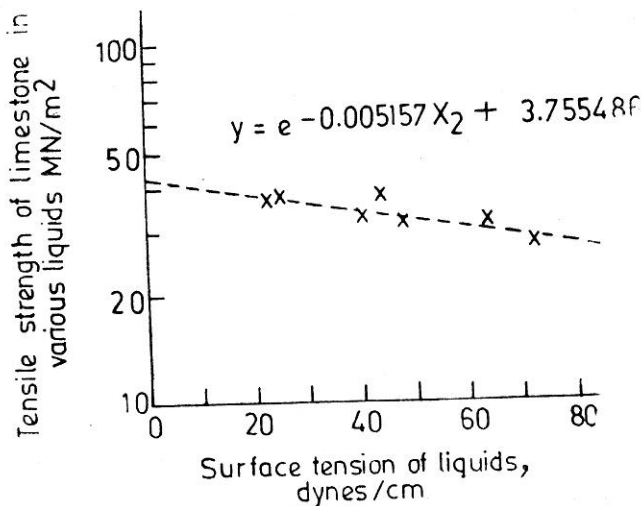


FIGURE 14 Influence of Dielectric Constant on Strength of Limestone (After Vutukuri, 1974)

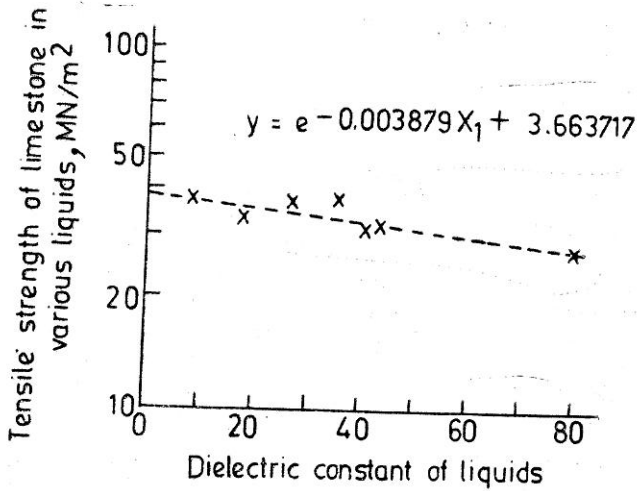


FIGURE 15 Influence of Surface Tension on Tensile Strength of Limestone (After Vutukuri, 1974)

u_a = effective pore air pressure, and

A and R = effective inter-particle repulsive and attractive pressures respectively.

The contact stress, $\bar{\sigma}_c$ has been hypothesized to be the stress controlling the engineering behaviour of soils. The success of this concept in the understanding of the strength behaviour (Sridharan and Venkatappa Rao, 1979) volume change behaviour (Sridharan and Venkatappa Rao, 1973) and shrinkage behaviour (Sridharan and Venkatappa Rao, 1971) has led the present authors to take a detailed look at the strength behaviour of rocks for a similar principle.

To enable such a study, relevant literature concerning the interparticle forces/surface energy and the pore size distributions is reviewed.

Evaluation of Attractive Forces

It is convenient, although artificial, to divide intermolecular forces into two types—long range (van der Waals) forces and short range (valence) forces. A rigorous treatment of short range forces cannot be given in terms of properties of the separated molecules. Instead, it is necessary to consider each molecular pair as a special case. Such an approach in a heterogeneous material like rock is impossible. However, it may be realised that these short range forces are the ones which are primarily responsible for the molecular structure/mineralogical nature of the rock. They in turn can be said to be responsible for the intrinsic strength of the rock and once broken cannot be reformed. Further they cannot be changed by the loads associated with engineering structures in rock. It is the long range forces which are subject to change, say, when the pore fluid varies from unsaturated condition, dry to wet or when the pore fluid is other than water. These forces can be described rigorously in terms of the physical properties of the separated molecules.

In the discussion of long range forces, it is customary to consider four types of forces:

- (i) electrostatic forces,
- (ii) induction forces
- (iii) dispersion forces (London forces) and
- (iv) resonance forces.

When two polar molecules collide, electrostatic forces come into play. The forces between two non-polar molecules, which have no resultant momentum are of the dispersion type. When a non-polar molecule interacts with a polar molecule both dispersion and induction forces occur. Resonance forces occur between two identical molecules, under certain specific conditions.

While the valence forces such as the metallic bond or hydrogen bond are a function of specific chemical nature, London dispersion forces exist in all types of matter and always impart an attractive force between adjacent atoms or molecules no matter how dissimilar their chemical natures may be (Fowkes, 1964). These forces depend on the electrical properties of the volume elements involved and the distance between them, and are independent of temperature. These forces operate over longer distances than for other intermolecular forces. Hamaker (1937) has shown that between flat plates separated by distance 'd' in vacuum, the attractive force,

$$F = \frac{A}{6\pi d^3} \quad \dots(2)$$

where A = attractive force constant.

Sridharan (1968) calculated the value 'A' for siliceous materials of different degrees of saturation as shown in Fig. 16. He has further shown that with increase in degree of saturation the dielectric constant of the pore medium increases (Fig. 17). Hence, it may be concluded that for partially saturated materials, with increase in degree of saturation, the dielectric constant of the system increases and the Hamaker's constant 'A' decreases.

Further, the senior author (1972) in his investigations carried out using various organic fluids, computed the variation of Hamaker's constant 'A' with dielectric constant of the pore fluid, which is shown in Fig. 18.

From the foregoing it may be concluded that a change in pore medium either by changing the degree of saturation or from one fluid to another, causes changes in the attractive force system, which for the particular force under consideration, can be directly linked to dielectric constant.

Evaluation of Repulsive Forces

The primary force which is responsible for repulsion between two particles floating in water is due to the interaction of diffuse double layers. According to the classical theory of Verwey and Overbeek (1948), when

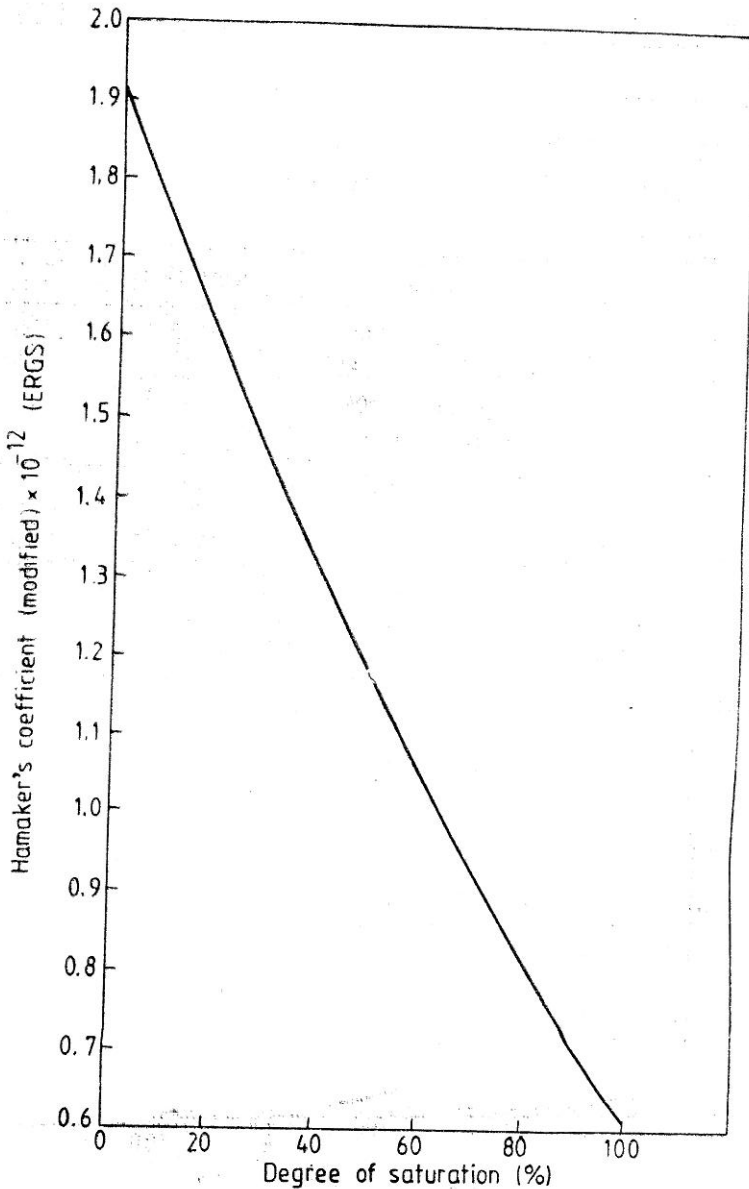


FIGURE 16 Variation of Hamaker's 'A' Coefficient with Degree of Saturation (After Sridharan, 1968)

all other factors including the distance between the particles remain constant, this repulsive force is proportional to the dielectric constant of the pore medium. Although a number of limitations exist for the application of double layer theory to soil-water or rock-water systems, this theory is extremely useful to the geotechnical engineers and vital to the qualitative understanding of the fundamental behaviour of soils and rocks, particularly for joints with clay infillings.

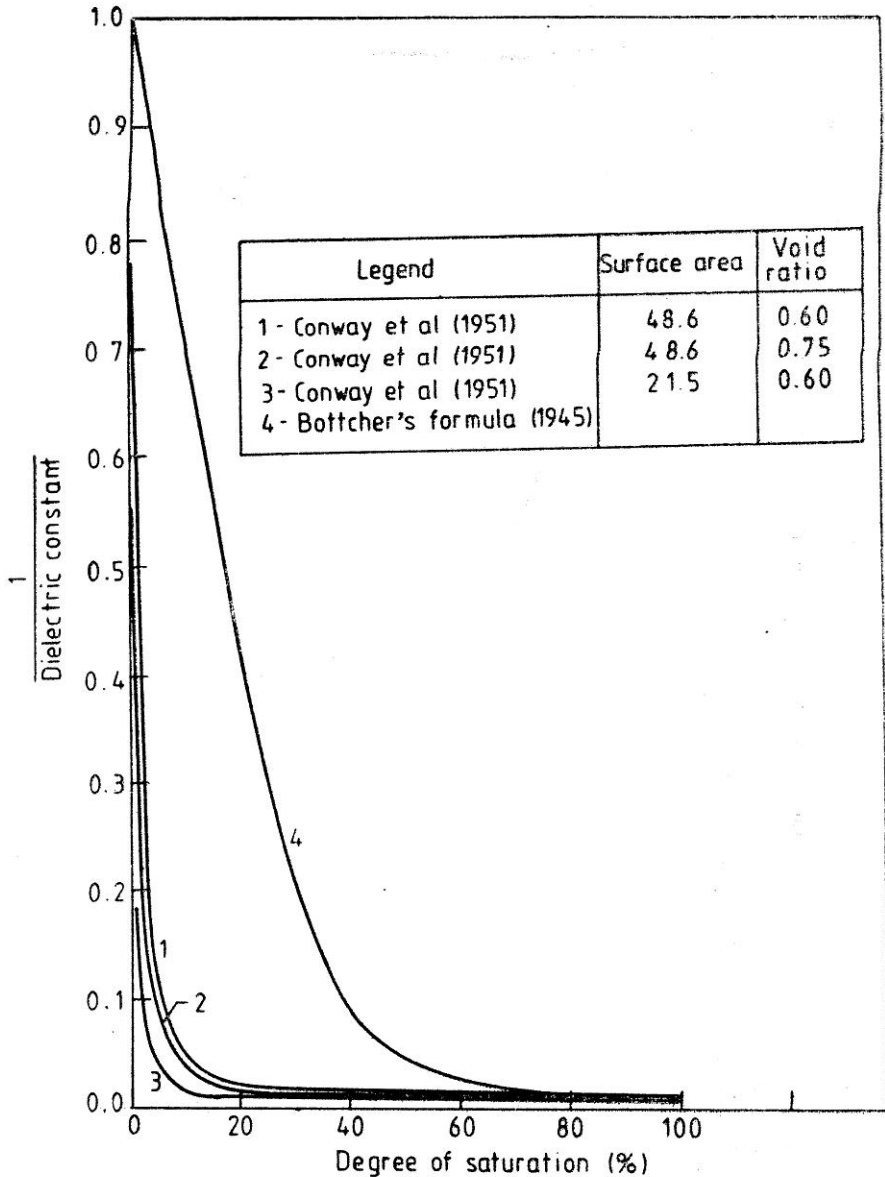


FIGURE 17 Variation of Dielectric Constant with Degree of Saturation (After Sridharan, 1968)

Pore Water Pressure Under Partial Saturation

The negative pore water pressure occurring in partially saturated soils/rocks has a direct bearing on the effective stress, thereby influencing their engineering behaviour.

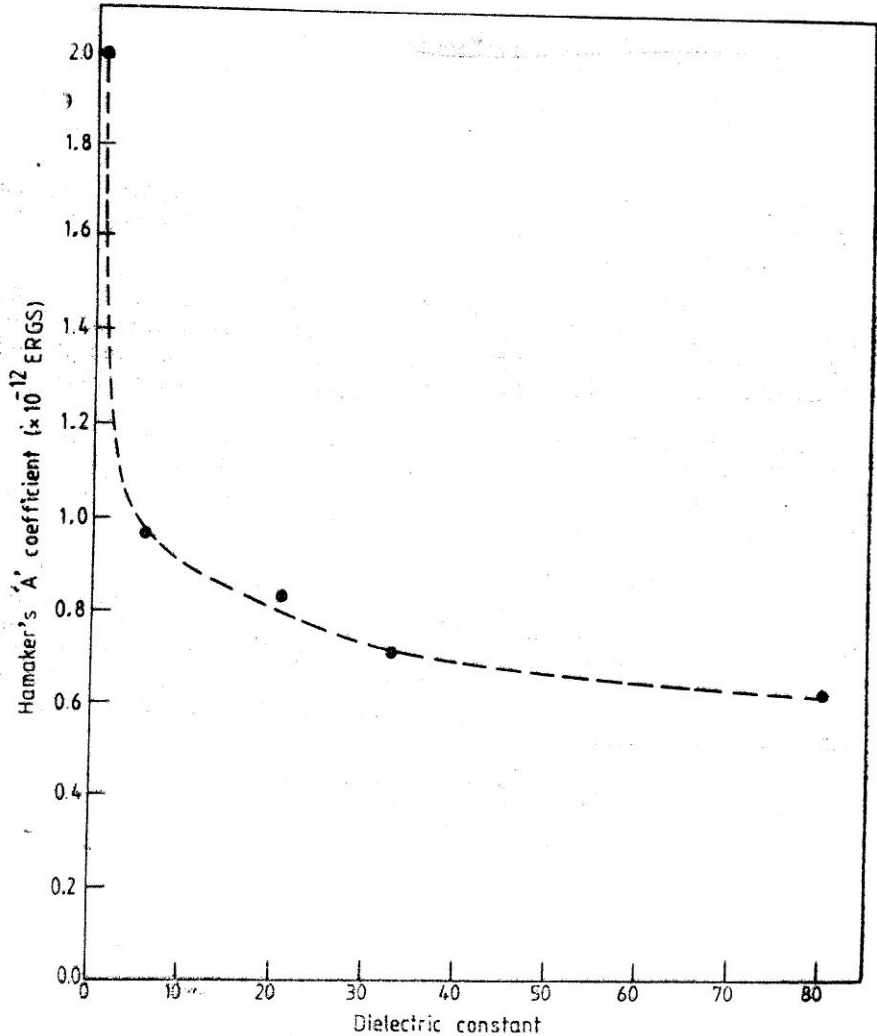


FIGURE 18 Variation of Hamaker's 'A' Coefficient with Dielectric Constant (After Venkatappa Rao, 1972)

Measurement of negative pore water pressure is very involved, and with the conventional pore pressure apparatus, negative pressures cannot be measured beyond minus one atmosphere, due to cavitation of water in the measuring system at about this pressure level. Even the more recent axis-translation technique can only be applied when there is continuity in both the air and water phase, which is only possible at high degrees of saturation.

An alternative approach for evaluating the negative pore water pressure is by the application of classical capillary equation by Kelvin :

$$P = \frac{2T_s \cos \theta}{r} \quad \dots(3)$$

where

P = pressure deficiency in pore of radius, r ,

T_s = surface tension of fluid, and

θ = contact angle.

Using this equation for materials of known pore size distributions, the effective negative pore water pressure can be computed (Venkatappa Rao and Sridharan, 1978).

Two independent methods exist for pore size distribution analysis. The first is the mercury injection method and the second is the analysis of sorption isotherms. The latter method is very simple, both in concept and in use and is described in detail by Venkatappa Rao and Rekhi (1979).

In the capillary equation the value of θ can be assumed to be equal to zero for soil-water systems (Aitchison 1961, Olson and Langfelder 1965). At constant temperature, T_s is a constant quantity for water. Hence, the capillary pressure is an inverse function of the pore radius.

The effective pore water pressure \bar{u}_w can be determined only if a_w is known; a_w being the fraction of the unit area through which the negative pore water pressure u_w is transmitted between the minerals. Hence, this should be directly proportional to the percentage of the total pore volume that is filled with water at a particular water content. Since the relation between water content and percentage of filled pore volume is known, a_w can be calculated. Thus,

$$\begin{aligned} a_w &= \frac{\text{Pore volume filled at any water content}}{\text{Total pore volume}} \\ &= \frac{\text{Total water content in c.c.—Adsorbed water in unfilled pores in c.c.}}{\text{Total pore volume}} \\ &= \frac{V_w - V_{aw}}{V_v} = \frac{V_w}{V_v} - \frac{V_{aw}}{V_v} \end{aligned}$$

Hence,

$$a_w = S - \frac{V_w}{V_v} \quad \dots(4)$$

The value of \bar{u}_w can now be obtained as the product of a_w and the capillary pressure, p .

Examination of the Effective Stress Principle

By changing the pore fluid from air (or dry state) to water (saturated state), the dielectric constant of the medium changes from 1 to 80.4. Dielectric constant of the pore medium can also be varied by use of various organic fluids. Considering a saturated system, an increase in dielectric constant of the medium, results in an increase in repulsion causing decrease

in the net attractive force. As a consequence of this, the effective contact stress decreases which brings in a decrease in macro-strength. It is worth noting that the electrical forces being referred here are definitely not predominant in sands or rocks under normal circumstances, when compared to clays. Figs. 19 and 20 (from Sridharan and Venkatappa Rao, 1973) show the variation in drained shear strength of kaolinite and montmorillonite clays with dielectric constant. When the dielectric constant changed from around unity to 80, the strength decreased by 2.5 to 4 times

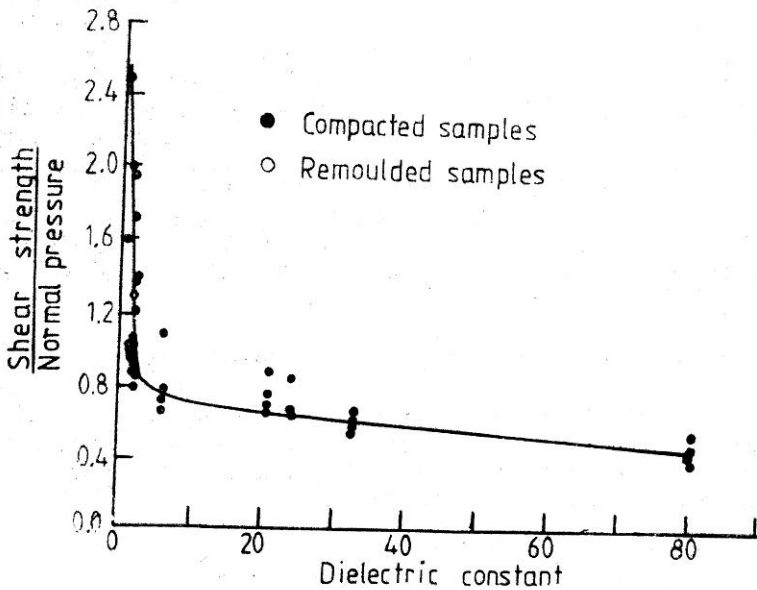


FIGURE 19 Effect of Dielectric Constant on Drained Shear Strength for Kaolinite (After Sridharan and Venkatappa Rao, 1973)

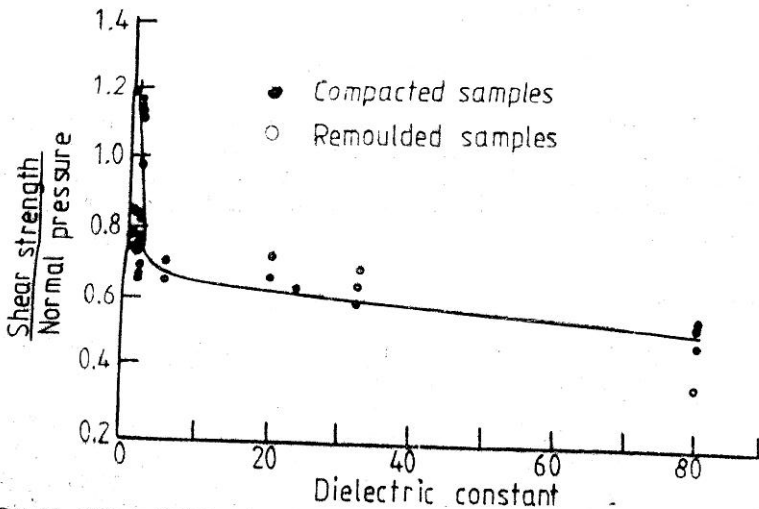


FIGURE 20 Effect of Dielectric Constant on Drained Shear Strength for Montmorillonite (After Sridharan and Venkatappa Rao, 1973)

Such a drastic reduction is of course not expected in sands/rocks because of the higher intrinsic strength caused by the strong primary valence bonds. However, there are occasions when the effect was noticed. Miura and Yamanouchi (1975) conducted a number of shear tests on a quartzitic sand with various pore fluids of different dielectric characteristics under high confining pressures. The results shown in Fig. 21 reveal that the maximum deviator stress decreases with dielectric constant, in a manner much similar to that observed earlier for clays though the relative change is much smaller. Further, if one plots the experimental results of Miura and Yamanouchi on the particle crushing in confined compression as shown in Fig. 22, it may be generally concluded that the dielectric constant of the fluid does have a significant effect on particle crushing. As is expected, the particle crushing increased with increase in dielectric constant.

If the behaviour of rocks studied by Colback and Wiid (1965) and Simpson and Eergus (1968) using different pore fluids is analysed in terms of dielectric constant as shown in Fig. 23 it is apparent that the dielectric constant is highly influential in changing the strength. The effect observed is qualitatively similar to that noticed in soils.

Mechanisms of Strength Reduction due to Moisture Changes

Several mechanisms and theories have been put forth by different workers to explain the influence of moisture on strength of rocks, but there appears to be no generally accepted explanation. van Eeckhout (1976) has summarized these different mechanisms as :

- (i) fracture energy reduction,
- (ii) capillary tension decrease,
- (iii) pore pressure increase,
- (iv) frictional reduction, and
- (v) chemical and corrosive deterioration.

Out of the above, (ii) and (iii) can be considered as a single mechanism —pore pressure change. None of these mechanism can be discounted outright, but some are likely to be more influential than others for certain rock types and loading conditions. The different mechanisms are discussed hereunder in detail.

Fracture Energy

Existence of micro-cracks within grains and at grain boundaries in rocks is common knowledge. For such brittle materials, Griffith's theory (1920) based on the initial presence of small cracks has been applied with success. As per Griffith's theory, the surface energy gained in rupturing the molecular bonds along the crack path must be equal to the net reduction in the strain energy of the system in order that the crack may propagate. Using Griffith's mathematical derivation for a plane body, or Sach's consideration for 3-D body and Orwan's atomic considerations for a plane body,

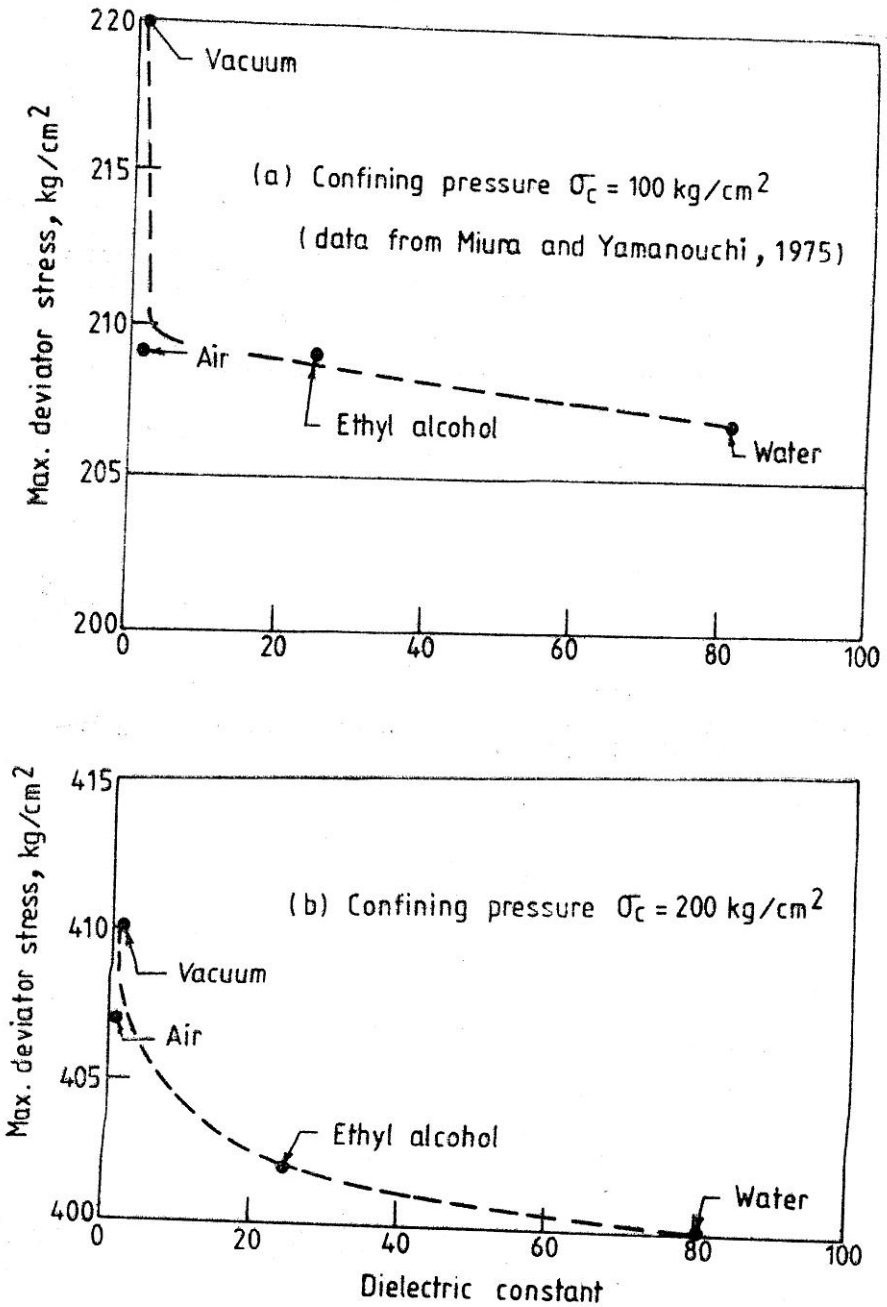


FIGURE 21 Influence of Dielectric Constant on Maximum Deviator Stress for Toyoura Sand at a) $\sigma_c = 100 \text{ kg/cm}^2$, (b) $\sigma_c = 200 \text{ kg/cm}^2$

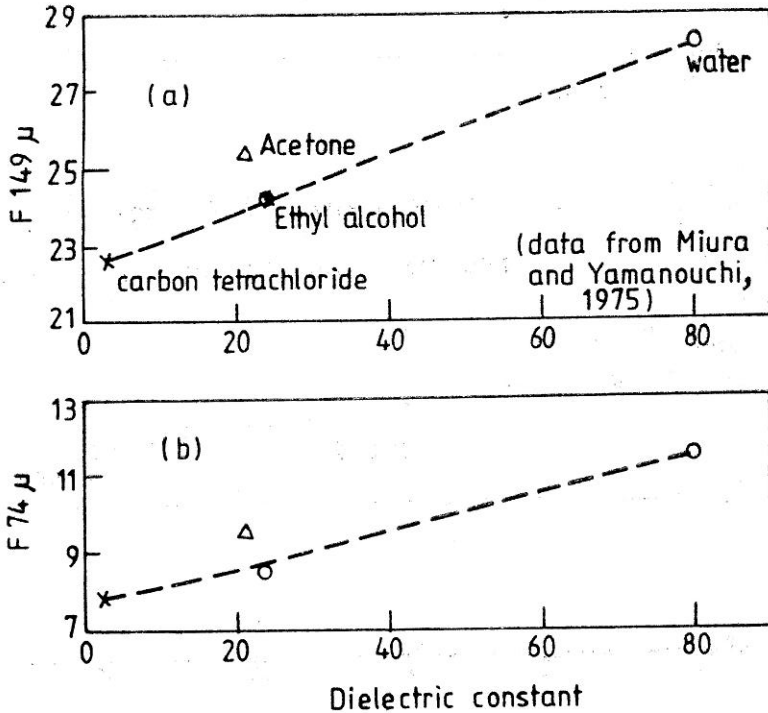


FIGURE 22 Influence of Dielectric Constant on Degree of Particle Crushing on Toyoura Sand (a) 149μ , (b) 74μ

one obtains a relation of the form,

$$\text{Strength} = K \sqrt{\frac{E\gamma}{C_0}} \quad (5)$$

where $K = \text{constant}$ (0.70 to 1.3)

$E = \text{modulus of elasticity}$

$\gamma = \text{surface energy, and}$

$C_0 = \text{half initial crack length.}$

This equation can be used for both tensile as well as compressive stress situations.

Colback and Wiid (1965) used the surface energy term of the above equation to explain the differences in strength due to different pore fluids in quartzitic sandstone. The influence of immersion in a liquid is to reduce the surface energy and hence its strength.

Brace and Walsh (1962) defined the surface energy of a solid as the work required to produce a certain amount of new surface. According to Pugh (1967), the origin of the surface energy is the work done in separating

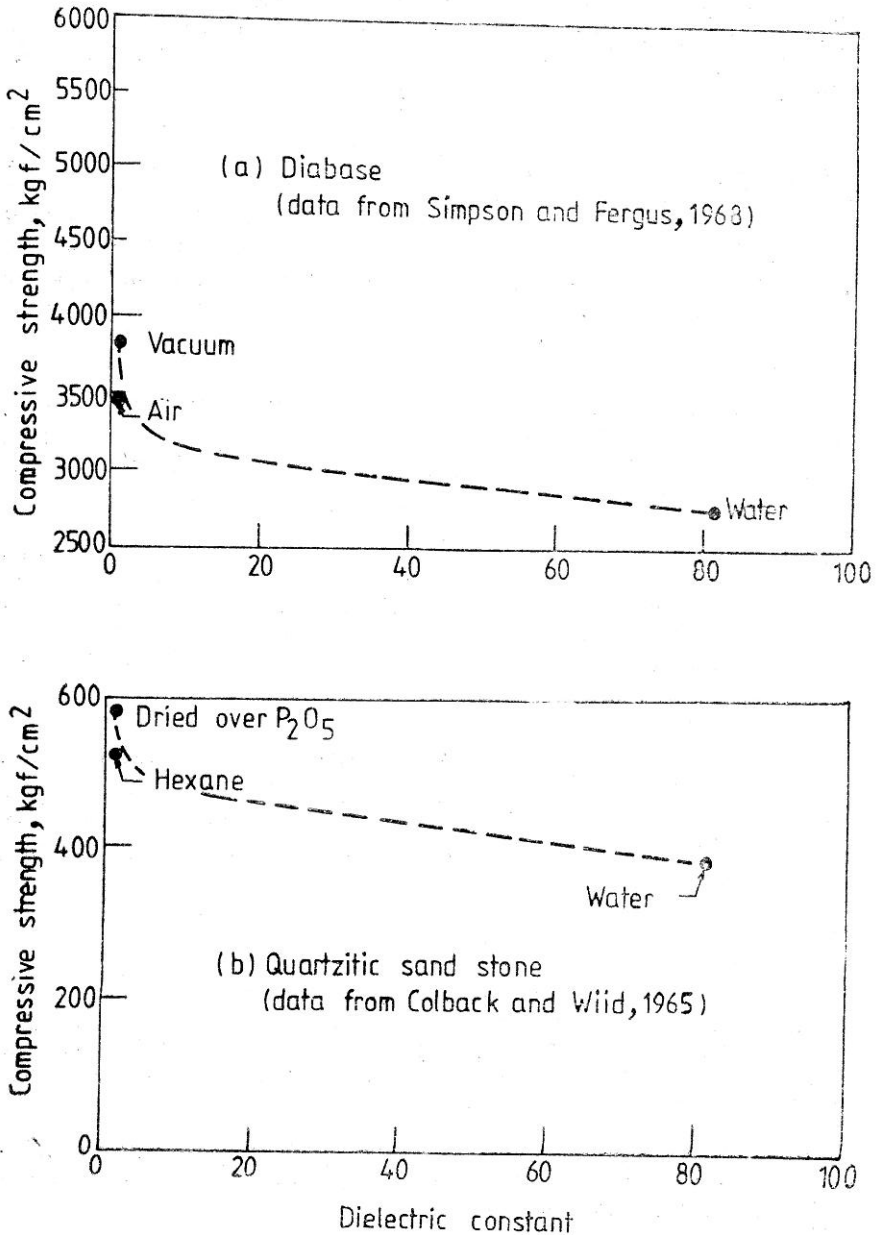


FIGURE 23 Influence of Dielectric Constant on Compressive Strength of (a) Diabase, (b) Quartzitic Sandstone

the crack surfaces against attractive interatomic forces. Thus, it may be inferred that what is customarily known as "surface energy" and which is considered to vary with pore fluid content and pore fluid type is commonly designated as "attractive and repulsive forces" which are shown to be a function of moisture and pore fluid type. An increase in pore fluid

content or degree of saturation will result in an overall decrease of net attractive force or in other words, surface energy, which when considered in terms of Griffith's criterion will reduce the fracture energy and hence its strength.

In the effective stress equation one can easily reconsider the net attractive term ($A-R$) as partly arising due to the mineralogical forces (which are constant for a given rock type) and the rest as due to the surface energy forces (which vary with moisture content). Thus with an increase in moisture content, while the mineralogical forces remain constant, the surface forces decrease, resulting in a decrease in $\bar{\sigma}_c$ causing a decrease in the strength, as observed.

Pore Pressure

While dealing with pore pressures, continuity of pores is to be assumed. If the rock is saturated, any application of stress will cause an increase in the pore water pressure, provided drainage is not allowed. This, of course, lowers the effective contact stress (Eq. 1) and hence the strength.

In partly saturated rocks, the pore water is under tension, or in other words, the pore water pressure is negative. The methodology of calculation of such negative pore water pressure is already described. Suffice it to mention here that negative pore water pressure is responsible for an increase in effective contact stress and hence an increase in strength.

Friction

Among the mechanisms suggested for strength reduction is the reduction in coefficient of friction due to moisture increase. In the present context, the coefficient of friction can be viewed as a manifestation of the surface energy. Changes in friction is another mechanistic way of viewing the strength.

Chemical and Corrosive deterioration

If the changes caused are permanent, then it is outside the scope of this investigation. On the other hand, if changes are due to expanding lattice type of clay minerals, they can be considered directly under the electrical force system. For example, absorption of water by montmorillonite results in an increase in repulsion.

It is thus seen that the effective stress principle encompasses the various mechanisms put forth by earlier workers explaining the strength reduction in rocks due to moisture changes.

Experimental Work

Rocks tested

Bunter sandstone and Bath Limestone from United Kingdom were used in this investigation. The physical properties of intact specimens are presented in Table 1.

TABLE 1

Physical Properties of Rocks Tested

Property	Bunter Sandstone		Bath Limestone	
	No. of Measurements	Mean value	No. of Measurements	Mean value
Dry Bulk Density (g/cm ³)	106	2.083	87	2.097
Saturated Bulk Density (g/cm ³)	43	2.299	87	2.322
Effective Porosity (per cent)	43	21.205	87	23.051

Specimen Preparation

It is desirable in a rock testing programme to select a set of conditions so that the combined effects of the various external factors that can influence rock behaviour are relatively small and hence the measured results are indicative of the real characteristics of the rock. Attempts have been made to minimize these effects by standardizing the procedure used in measuring the mechanical properties of the rock (as recommended by ISRM 1972).

Nominal dimensions of 50.8 mm diameter and 127 mm length were adopted for the specimens. The length to diameter ratio (2.5:1) conforms to the one generally recommended for compression tests (ISRM, 1972). The 50.8 mm diameter specification sufficiently meets the ISRM recommendation that the diameter be 10 times the maximum grain size of the specimen.

Sawn blocks with parallel ends and dimensions of 30×30×20cm were selected from a single location at each quarry to ensure material uniformity. Specimens were cored in a single direction normal to the surface face of the parent block. These cores were cut to length in a lathe. Precision grinding was required in order to obtain specimens with end faces parallel and also at right angles to the core axis, the specimens were rigidly held in V—blocks and positioned on a reference plane which was a carefully ground marble base. Both ends were then ground carefully in a precision grinding machine. After grinding, the samples were placed on a steel base and the end tolerances were checked by a dial gauge measuring to 0.0029 mm.

One hundred and seven specimens of Bunter sandstone and eighty nine specimens of Bath Limestone were prepared in this way with smooth sides (*i.e.* free of abrupt irregularities) and a maximum deviation from parallelism of 0.005 mm in the 50.8 mm diameter sample.

Moisture Content Control

Nine different levels of moisture contents were produced for these studies, ranging from a dry to fully saturated condition. All test specimens were initially air-dried in an oven for 5 days at 50°C and were weighed to the nearest 0.0001g on an electric balance at the end of this period. The

dry weight of the specimens was taken as the various treatments to obtain different moisture contents were related.

Different salt solutions produce different well defined relative humidities in a confined atmosphere over them. Intermediate moisture contents obtained by curing the samples above various salt solutions in an air-tight vessel or by placing the samples in moist saw dust for a period of several weeks.

Test Equipment and Procedure

A closed-loop fast response, programmable ESH Serve-controlled machine was used in uniaxial compression testing of the specimens. The servo-controlled testing machine enables the displacement to be maintained at a constant rate and the failure to be controlled beyond the ultimate strength of the specimen. The axial displacement of the specimen due to loading was measured by a pair of Linear Variable Differential Transducers (LVDT). Circumferential deformation was measured in two ways, firstly by a circumferentially mounted electrical resistance strain gauge, and secondly by a LVDT mounted to the wrap-around device. LVDT measured circumferential displacement was used to control the progress of the failure of the specimen.

Each data point in the graphs or table is an average of at least nine specimen test results.

Test Results and Discussion

The water vapour absorption isotherms obtained for the two rocks are presented in Fig. 24. From these isotherms the pore size distribution is computed following a method described by Venkatappa Rao and Rekhi (1979) assuming cylindrical pores and presented in Tables 2 and 3 for Bunter sandstone and Bath limestone respectively. The negative pore water pressure is then calculated by the use of Kelvin's equation and presented in Fig. 25, as a function of moisture content. From this figure it may be observed that for both the rocks after an initial peak, the negative pore water pressure decreases rapidly with moisture content. The behaviour is somewhat similar to that observed in compacted soils (Venkatappa Rao and Sridharan, 1978). Amongst the two rocks the limestone has a higher negative pore water pressure, at a given moisture content; but the rate at which it falls is also higher for limestone with a moisture content between 0.25 and 0.40 percent.

The variation in the uniaxial compressive strength with moisture content is presented in Fig. 26. In general, Bunter sandstone exhibited higher compressive strength than Bath limestone. As observed by earlier researchers, both the rocks exhibit maximum strength when dry and minimum, when saturated. With increase in moisture content upto 1 percent, the strength falls rapidly from its dry value and beyond a moisture content of 1 percent the decrease is only marginal. To depict clearly the rapid change at low moisture contents (which were achieved by curing at different relative humidities), the strength results are replotted against relative humidity and shown in Fig. 27.

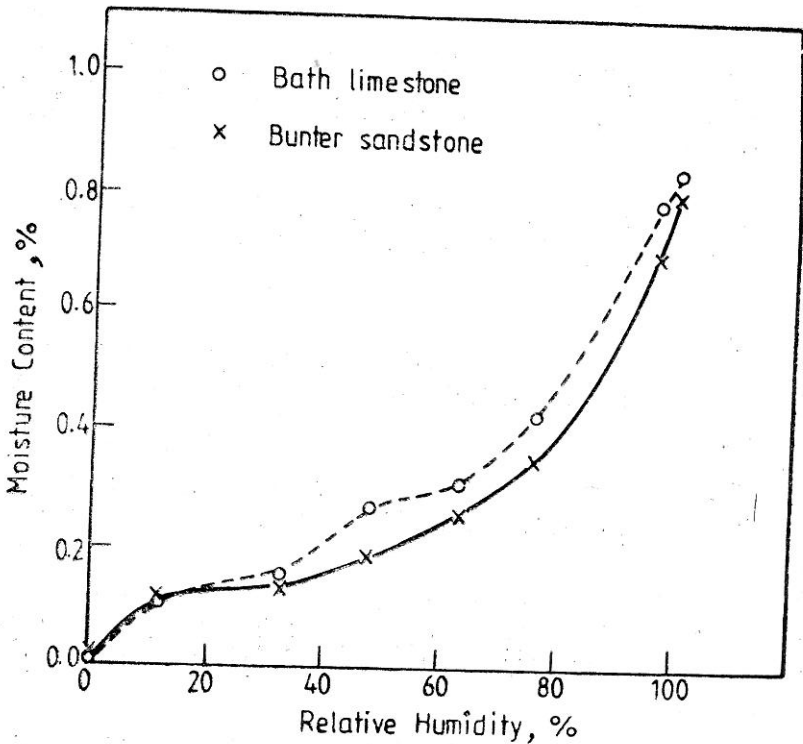


FIGURE 24 Water Vapour Adsorption Isotherms

A scrutiny of the results obtained reveals that there is a clear reduction in strength with increase in moisture. These results are in line with those of earlier researchers, as detailed in the literature review. The strength decrease can be attributed to two factors :

- (i) a change in the surface energy or electrical force system, and
- (ii) a change in the pore water pressure. (As the specimens were tested without confinement, the pore air pressure remains constant at atmospheric pressure).

It is already brought out that an increase in moisture reduces the net attractive force or in other words the surface energy. It may also be recalled that the negative pore water pressure decreased rapidly with the increase in moisture content. Thus, both these factors influence in reducing the contact stress with increase in moisture, which in turn results in a reduction in strength.

A further analysis of the results is possible by the use of Griffith's criterion. It has been mentioned that as per this criterion, the strength is expected to be proportional to $(E\gamma)^{1/2}$. The variation of deformation modulus with moisture is presented in Fig. 28. As both E and γ change with moisture content, a study can be made of the variation in $(\sigma_a/\sigma_c \text{ dry})$, $(\gamma/\gamma_{dry})^{1/2}$ and $(E\gamma/E_{dry} \gamma_{dry})^{1/2}$ with respect to moisture content. For this study, the surface energy of dry sandstone is assumed to be the same as

TABLE 2
Analysis of Water Vapour Adsorption Isotherm of Bunter Sandstone

Relative Humidity (%)	Moisture Content (%)	$V_{adsorbed}$ (uncorrected) (cc)	$V_{adsorbed}$ (corrected) (cc)	S_{uncorr} (m^2/g)	$S_{corrected}$ (m^2/g)	$I_h^{(core)}$ uncorrected (A°)	$I_h^{(core)}$ corrected (A°)	$\Gamma_A^{(core)}$ (A°)	$\Gamma_A^{(pore)}$ (A°)	Pore diameter (mm)
100.0	0.808	0.000721	0.000714	0.017357	0.017357	415.63	411.49	411.49	411.49	0.1646
97.5	0.7359	0.000553	0.000543	0.040393	0.039721	136.79	136.79	136.79	136.79	0.0547
95.0	0.6806	0.000440	0.000430	0.054304	0.053077	81.01	81.01	81.01	81.01	0.0324
92.5	0.6366	0.000440	0.000430	0.077046	0.075252	57.10	57.10	57.10	57.10	0.0228
90.0	0.5926	0.000440	0.000428	0.100421	0.097808	43.81	43.81	43.81	43.81	0.0175
87.5	0.5486	0.000440	0.000225	0.124463	0.120359	35.34	33.34	33.34	35.34	0.0141
85.0	0.5046	0.000440	0.000426	0.149212	0.144617	29.48	29.48	29.48	29.48	0.01179
82.5	0.4607	0.000440	0.000424	0.174712	0.168344	25.18	25.18	25.18	25.18	0.01007
80.0	0.4167	0.000440	0.000426	0.201008	0.194570	21.88	21.88	21.88	21.88	0.00875
77.5	0.3727	0.000365	0.000351	0.199426	0.181833	19.28	19.28	19.28	19.28	0.00771
75.0	0.3361	0.000173	0.000165	0.100889	0.095817	17.17	17.17	17.17	17.17	0.00687
72.5	0.3188	0.000173	0.000167	0.112316	0.108169	15.42	15.42	15.42	15.42	0.00617
70.0	0.3015	0.000173	0.000167	0.124151	0.119829	13.95	13.95	13.95	13.95	0.00558
67.0	0.2842	0.000173	0.000167	0.136424	0.131191	12.70	12.70	12.70	12.70	0.00508
65.5	0.2669	0.000160	0.000153	0.137365	0.131707	11.61	11.61	11.61	11.61	0.00465
62.5	0.2509	0.000105	0.000099	0.098159	0.092859	10.67	10.67	10.67	10.67	0.00427
60.0	0.2404	0.000105	0.000101	0.106503	0.102755	9.83	9.83	9.83	9.83	0.00393
57.5	0.2300	0.000105	0.000101	0.115211	0.110819	9.09	9.09	9.09	9.09	0.00363
55.0	0.2195	0.000105	0.000100	0.124314	0.118767	8.42	8.42	8.42	8.42	0.00337
52.5	0.2090	0.000105	0.000100	0.133851	0.127414	7.82	7.82	7.82	7.82	0.00313
50.0	0.1986	0.000105	0.000100	0.143866	0.137568	7.28	7.28	7.28	7.28	0.00291
47.5	0.1881	0.000105	0.000100	0.154978	0.147706	6.78	6.78	6.78	6.78	0.00271
45.0	0.1776	0.000105	0.000100	0.166300	0.160204	6.32	6.32	6.32	6.32	0.00256
42.5	0.1671	0.000105	0.000100	0.178137	0.171136	5.90	5.90	5.90	5.90	0.00233
40.0	0.1556	0.000105	0.000100							

TABLE 3
Analysis of Water Vapour Adsorption Isotherm of Both Limestone

Relative Humidity (%)	Moisture Content (%)	$V_{adsorbed}$ (uncorrected) (cc)	$V_{adsorbed}$ corrected (cc)	$S_{uncorrected}$ (m^2/g)	$S_{corrected}$ (m^2/g)	$\Gamma_{K(core)}$ uncorrected (A°)	$\Gamma_{K(core)}$ corrected (A°)	$\Gamma_{K(pore)}$ (A°)	Pore diameter (mm)
100.0	0.797	0.000007	0.000007	0.000172	0.000172	415.63	411.49	411.49	0.16460
97.5	0.7963	0.000271	0.000268	0.019821	0.019589	136.79	136.79	136.79	0.05471
95.0	0.7692	0.000447	0.000439	0.055194	0.076486	81.01	81.01	81.01	0.03240
92.5	0.7245	0.000447	0.000437	0.078309	0.054222	57.10	57.10	57.10	0.02284
90.0	0.6798	0.000447	0.000447	0.102267	0.099411	43.81	43.81	43.81	0.01752
87.5	0.6350	0.000447	0.000432	0.126503	0.146988	35.34	35.34	35.34	0.01414
85.0	0.5903	0.000447	0.000431	0.151658	0.171103	29.48	29.48	29.48	0.01179
82.5	0.5456	0.000447	0.000433	0.177576	0.197760	25.18	25.18	25.18	0.01007
80.0	0.5009	0.000447	0.000433	0.204303	0.193099	21.88	21.88	21.88	0.00875
77.5	0.4562	0.000447	0.000372	0.200981	0.136430	19.28	19.28	19.28	0.00771
75.0	0.4174	0.000388	0.000224	0.136430	0.130535	17.17	17.17	17.17	0.00687
72.5	0.3940	0.000234	0.000226	0.151882	0.146274	15.42	15.42	15.42	0.00617
70.0	0.3706	0.000234	0.000226	0.167886	0.162042	13.95	13.95	13.95	0.00558
67.5	0.3472	0.000234	0.000226	0.184483	0.177406	12.70	12.70	12.70	0.00508
65.0	0.3237	0.000234	0.000225	0.171869	0.164504	11.61	11.61	11.61	0.00465
62.5	0.3038	0.000200	0.000191	0.171869	0.151984	10.67	10.67	10.67	0.00427
60.0	0.2977	0.000061	0.000055	0.057137	0.051984	9.83	9.83	9.83	0.00393
57.5	0.2916	0.000061	0.000059	0.061994	0.061994	9.09	9.09	9.09	0.00363
55.0	0.2855	0.000061	0.000059	0.067063	0.064506	8.42	8.42	8.42	0.00337
52.5	0.2794	0.000061	0.000058	0.072362	0.069133	7.82	7.82	7.82	0.00313
50.0	0.2733	0.000061	0.000058	0.077913	0.074167	7.28	7.28	7.28	0.00291
47.5	0.2672	0.000061	0.000058	0.083742	0.080077	6.78	6.78	6.78	0.00271
45.0	0.2512	0.000160	0.000155	0.235654	0.227881	6.32	6.32	6.32	0.00253
42.5	0.2328	0.000184	0.000178	0.291707	0.281701	5.90	5.90	5.90	0.00236
40.0	0.2143	0.000184	0.000177	0.312470	0.300190				

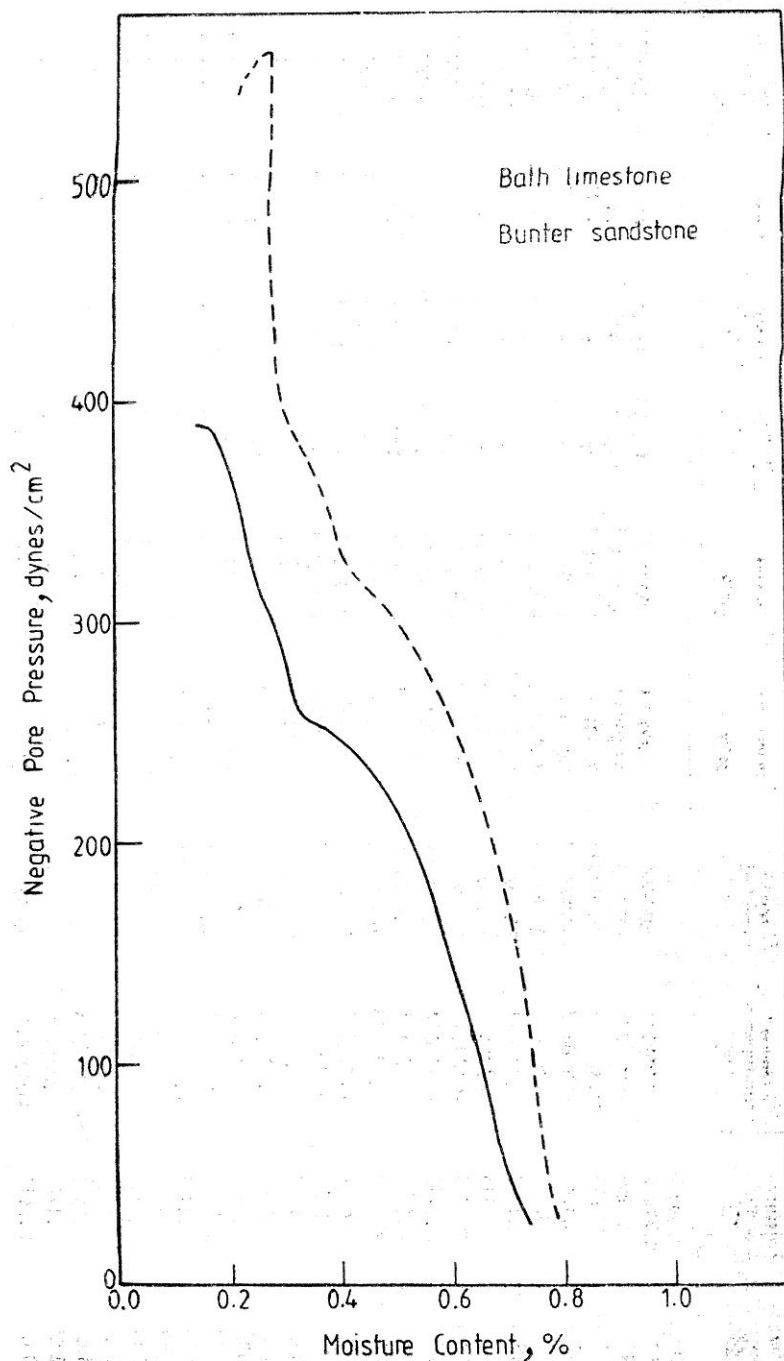


FIGURE 25 Variation in Negative Pore Water Pressure with Moisture Content

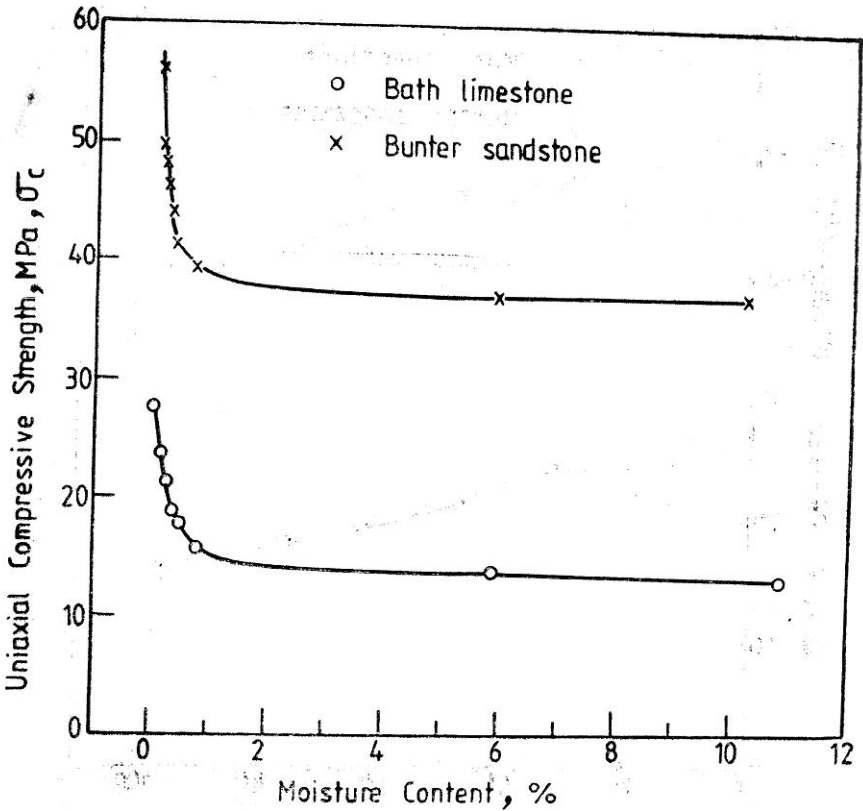


FIGURE 26 Variation in Uniaxial Compressive Strength with Moisture Content

that for quartz, equal to 640 erg/cm^2 * (Brace and Walsh, 1962) which on exposure to a relative humidity of 100 percent reduces by 240 erg/cm^2 (Boyd and Livingston, 1942). Similarly, the value chosen for limestone is that of calcite equal to 285 erg/cm^2 (Brace and Walsh, 1962) which reduces to 60 erg/cm^2 on immersion in saturated water vapour. Assuming that the reduction in surface energy is proportional to the change in relative humidity, the computation for $(\gamma/\gamma_{dry})^{1/2}$ and $(E\gamma/E_{dry}\gamma_{dry})^{1/2}$ are tabulated in Tables 4 and 5 and presented in Fig. 29. As seen in the figure with increase in moisture, the variation in $(\gamma/\gamma_{dry})^{1/2}$ and $(E\gamma/E_{dry})^{1/2}$ is similar to that of $(\sigma_c/\sigma_{c,dry})$.

With the many assumptions made in arriving at this picture, it will be futile to expect one to one correspondence. But two specific observations can be made. At low moisture contents upto 0.30 percent, limestone exhibits higher $(\sigma_c/\sigma_{c,dry})$, beyond which sandstone shows higher values. Possibly this is due to the rapid fall in negative pore water pressure of limestone in this regard. Whereas for sandstone the curves for $(\sigma_c/\sigma_{c,dry})$

* No unique values are available concerning the surface energy, however, theoretical predictions agree well with experimental observations for silicate minerals (Paterson, 1978).

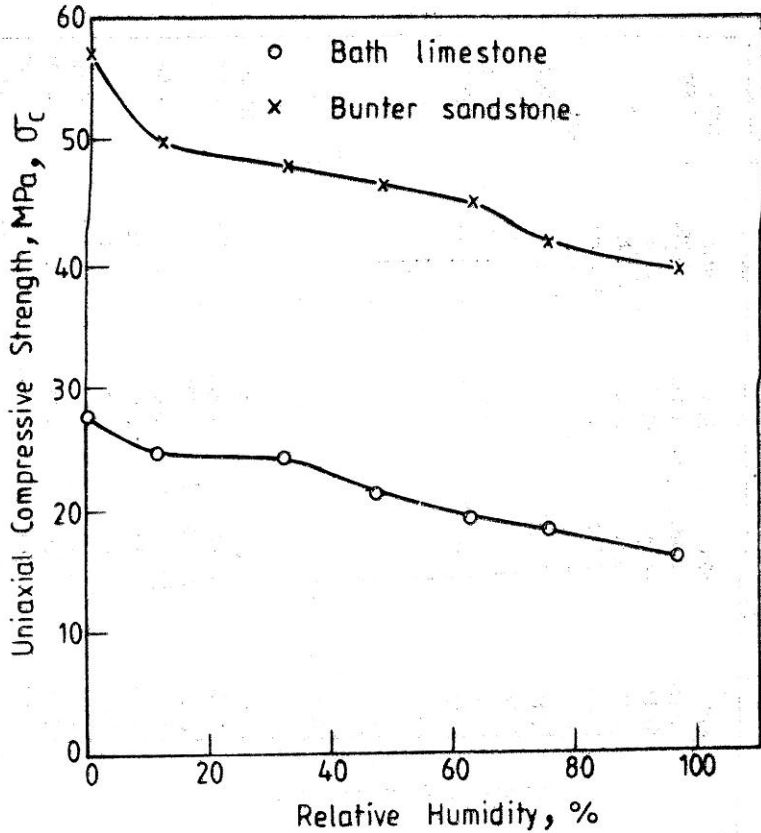


FIGURE 27 Variation of Uniaxial Compressive Strength With Relative Humidity

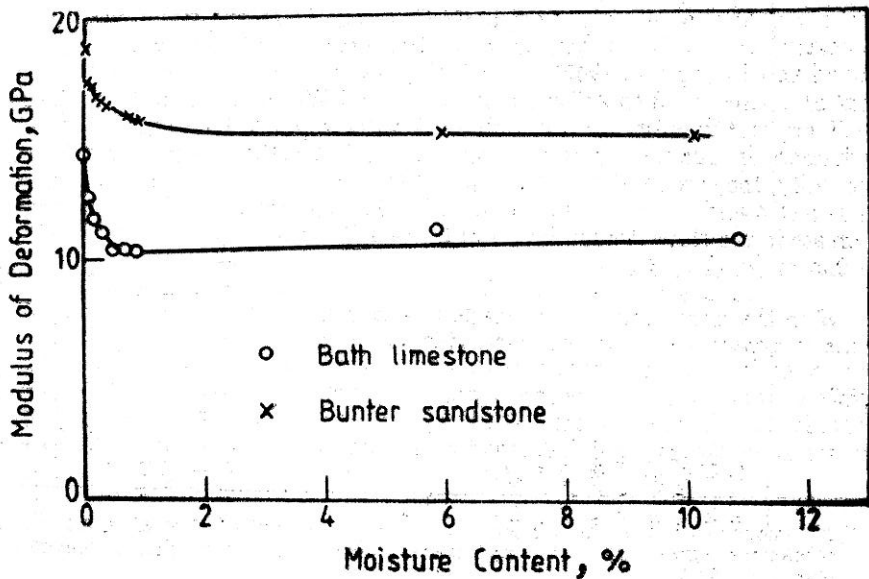


FIGURE 28 Variation in Deformation Modulus with Moisture Content

TABLE 4

Test Results for Bunter Sandstone

RH (%)	0	11.5	32.5	47.5	63.0	74.5	96.5	100.0
MC (%)	0.009	0.109	0.125	0.186	0.256	0.341	0.707	0.869
σ_c (MPa)	56.81	49.77	47.89	46.01	44.60	41.21	39.42	15.75
E (GPa)	18.64	17.35	17.03	16.61	16.50	16.43	15.83	15.75
σ_c/σ_c dry	1.0	0.876	0.843	0.810	0.785	0.725	0.694	
γ (erg/cm ²)	6.40	612.4	562	526	489	459	408.4	400
γ/γ dry	1.0	0.957	0.878	0.822	0.764	0.717	0.638	0.625
$(\gamma/\gamma$ dry) ^{1/2}	1.0	0.978	0.937	0.907	0.874	0.847	0.799	0.791
$E\gamma/E$ dry γ dry)	1.0	0.891	0.802	0.732	0.676	0.632	0.542	0.528
$(E\gamma/E$ dry γ dry) ^{1/2}	1.0	0.944	0.896	0.856	0.822	0.795	0.736	0.727

TABLE 5

Test Results for Bath Limestone

RH (%)	0	11.5	32.5	47.5	63.0	75.5	96.5	100.0
MC (%)	0.006	0.105	0.154	0.266	0.305	0.424	0.796	0.850
σ_c (MPa)	27.589	24.580	23.939	21.236	18.938	17.994	15.666	10.43
E (GPa)	14.15	12.52	12.26	12.14	11.18	10.42	10.45	10.43
σ_c/σ_c dry	1.0	0.891	0.868	0.770	0.686	0.652	0.568	
γ (erg/cm ²)	285	259	212	178	143	115	6.8	60
γ/γ dry	1.0	0.909	0.744	0.597	0.502	0.404	0.238	0.210
$(\gamma/\gamma$ dry) ^{1/2}	1.0	0.953	0.862	0.773	0.709	0.636	0.488	0.458
$E\gamma/E$ dry γ dry)	1.0	0.804	0.645	0.512	0.094	0.297	0.176	0.155
$(E\gamma/E$ dry γ dry) ^{1/2}	1.0	0.897	0.803	0.716	0.628	0.545	0.420	0.394

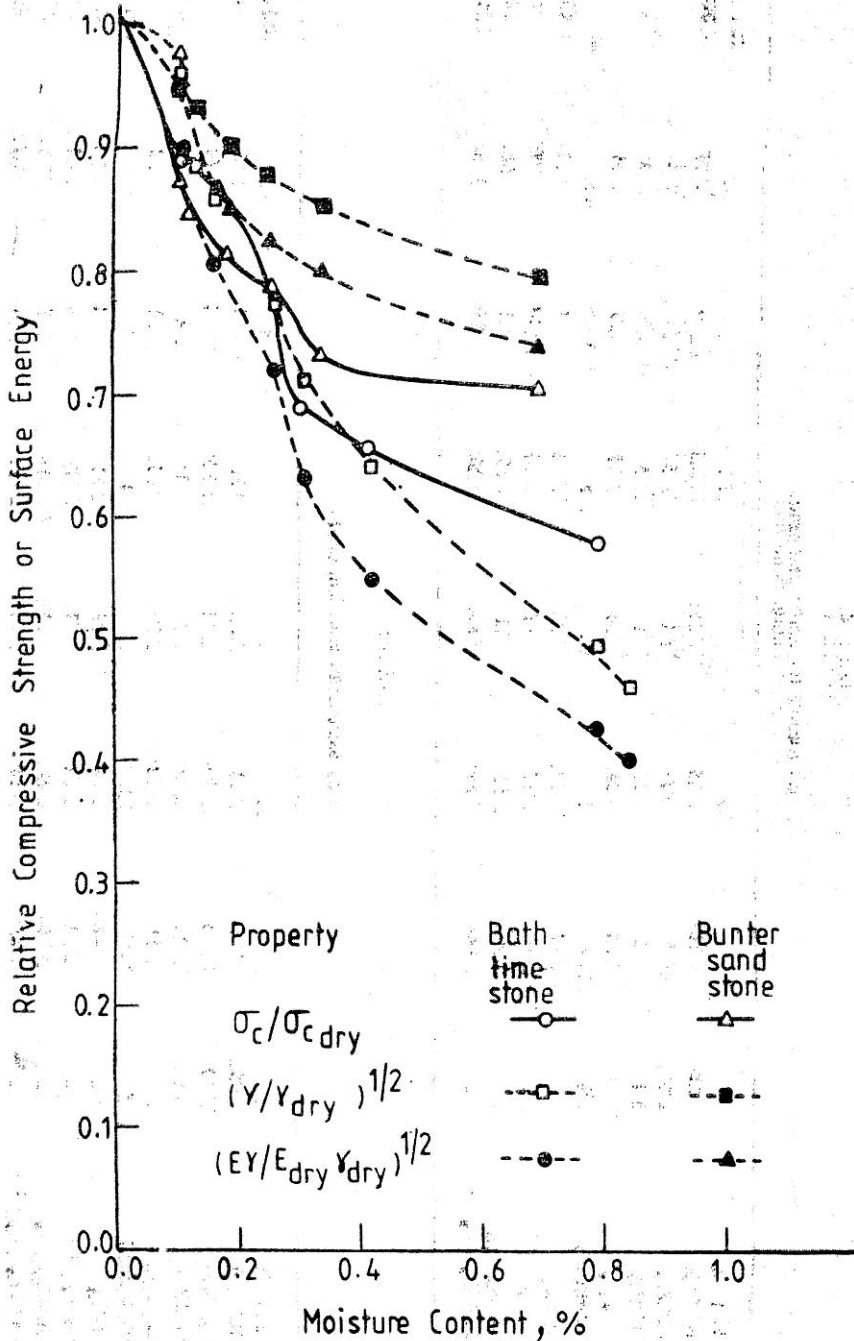


FIGURE 29 Variation of Relative Compressive Strength and Surface Energy with Moisture Content

and $(E\gamma/E_{dry}\gamma_{dry})^{1/2}$ are rather close, for limestone these curves are farther off. This difference in limestone is possibly because of the higher contribution of negative pore water pressure towards strength.

Summary and Conclusions

An extensive literature review reveals that the strength of an intact rock—compressive, tensile, point load, triaxial, as well as the modulus of deformation—is significantly reduced with an increase in moisture or change in pore fluid type. To understand these changes from the physico-chemical view point the principle of effective stress is brought into focus. This principle, successful in explaining the engineering behaviour of clays, takes into consideration the interparticle electrical attractive and repulsive forces, pore water pressure and pore air pressure. The existing mechanisms used to explain the strength reduction due to moisture increase are examined in the light of the effective stress principle. The results of the uniaxial compressive strength tests conducted on a limestone and a sandstone with varying moisture contents obtained by curing the specimens at different relative humidities, are compared with changes in surface energy and changes in negative pore water pressure. Whereas the comparisons are clearly qualitative in nature, the results bring to focus the factors responsible for the strength reduction and depict the potential of the effective stress principle, in understanding such behaviour.

Acknowledgement

The authors express their gratitude to the authorities of Indian Institute of Technology, Delhi and Imperial College of Science and Technology, London for extending the necessary help and to Prof. E.T. Brown for providing necessary facilities at the Rock Mechanics Division at ICST and also for interest shown. Grateful thanks are also extended to Prof. T. Ramamurthy for his continuous encouragement and also for meticulously going through the manuscript and making valuable suggestions.

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