1966

The effect of moisture on the compressive and tensile strength on a variety of rock materials

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THE EFFECT OF MOISTURE ON THE

COMPRESSIVE AND TENSILE STRENGTH

ON A VARIETY OF ROCK MATERIALS

BY

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A

THESIS

submitted to the faculty of the

UNIVERSITY OF MISSOURI AT ROLLA

in partial fulfillment of the requirements for the

Degree of

MASTER OF SCIENCE IN MINING ENGINEERING

Rolla, Missouri

1966

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ABSTRACT

The effect of moisture on the strength characteristics of rock materials has been neglected in the study of rock mechanics. This study was undertaken to determine if there was any effect on the compressive and the tensile strength of rock by varying the moisture content from an oven-dried to a saturated condition.

It was found, by testing eight different rock materials, that the compressive strength per unit area decreased with an increasing moisture content. The tensile strengths per unit area, with the exception of the quartzite and the porphyry samples, also decreased with an increase in moisture content. The tensile strength per unit area of the porphyry and quartzite increased with an increase in moisture content. It was also found that the graphs of strength versus moisture content generally followed a log-log relationship.

ACKNOWLEDGEMENTS

The writer wishes to express his appreciation to Dr. James J. Scott and Professor Richard L. Ash of the Mining Engineering Department, University of Missouri at Rolla, who offered continued assistance and guidance throughout the investigation. Appreciation is also due to Professor John B. Heagler, of the Civil Engineering Department, for his valuable advice, and to Professor Robert F. Bruzewski, of the Mining Engineering Department, for his assistance in assembling the photographs included in this thesis. Appreciation is also extended to the White Pine Copper Company and the Meramec Mining Company, who graciously supplied the samples used in the testing program. The writer is indebted to his wife, Joyce, who willingly typed the manuscript.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

Figure | Page

LIST OF TABLES

I. INTRODUCTION

The design of mine openings and mine excavations is based partly or solely on the strength characteristics of the rock materials encountered. Competition in the world market necessitates that the most profitable and safest design be developed in the rock formations. Moisture in the rock should have an effect on the strength and, therefore, on the design of both open pit and underground mining operations.

Moisture has been shown to reduce the strength of soil material and has long been encompassed in the studies of soil mechanics and foundation design. The difference in soils and rocks is in the hardness or more specifically, in the degree of compaction, the bonding between grains and the mode of formation. The principal reason for neglecting the effect of moisture in past studies is the fact that rock has so little void volume that the moisture content is small as compared to the solid or granular portion of the rock. Since moisture is normally present in rock it is logical to investigate its effect on rock strength even though these volumes may be small.

By testing rock materials for the effects of moisture on their strengths, the doors may be opened to further studies on the feasibility of stabilizing slopes and mine openings by eliminating the moisture present in the rock. It may be possible that dewatering a rock zone to increase its strength could become a useful technique in the future.

II. REVIEW OF LITERATURE

A. Rock Mechanics Theory

In reviewing the past publications of work done on the effects of moisture on rock strength, little work was found in this area. Three major pieces of work (1, 2, 3) have been done in the line of rock mechanics studies. Related works in the field of soil mechanics were also reviewed in order to determine if the general theories in this field would apply for rock materials.

1. Unconfined Compression Tests

Results on the effect of moisture on the unconfined compressive strength (Figs. 1, 2, 3) of three types of limestone were obtained by Razvi (1) in his investigation. The specimens were prepared by submerging them for 7 to 10 days in water and then allowing them to dry in air. Tests were made, after calculating the moisture content, each day until no change in strength was observed. This occurred on about the seventh day of air drying. By completely drying the specimen in the oven, there was an additional increase in strength observed.

Tests run by Colback and Wiid (2) showed a decrease in strength of approximately 50% for specimens tested after being submerged in water for long periods of time to specimens tested after being dried over calcium chloride (Figs. 4, 5). In compressive tests made by the Bureau of Mines, Table 1, on specimens that had been submerged for 7 days,

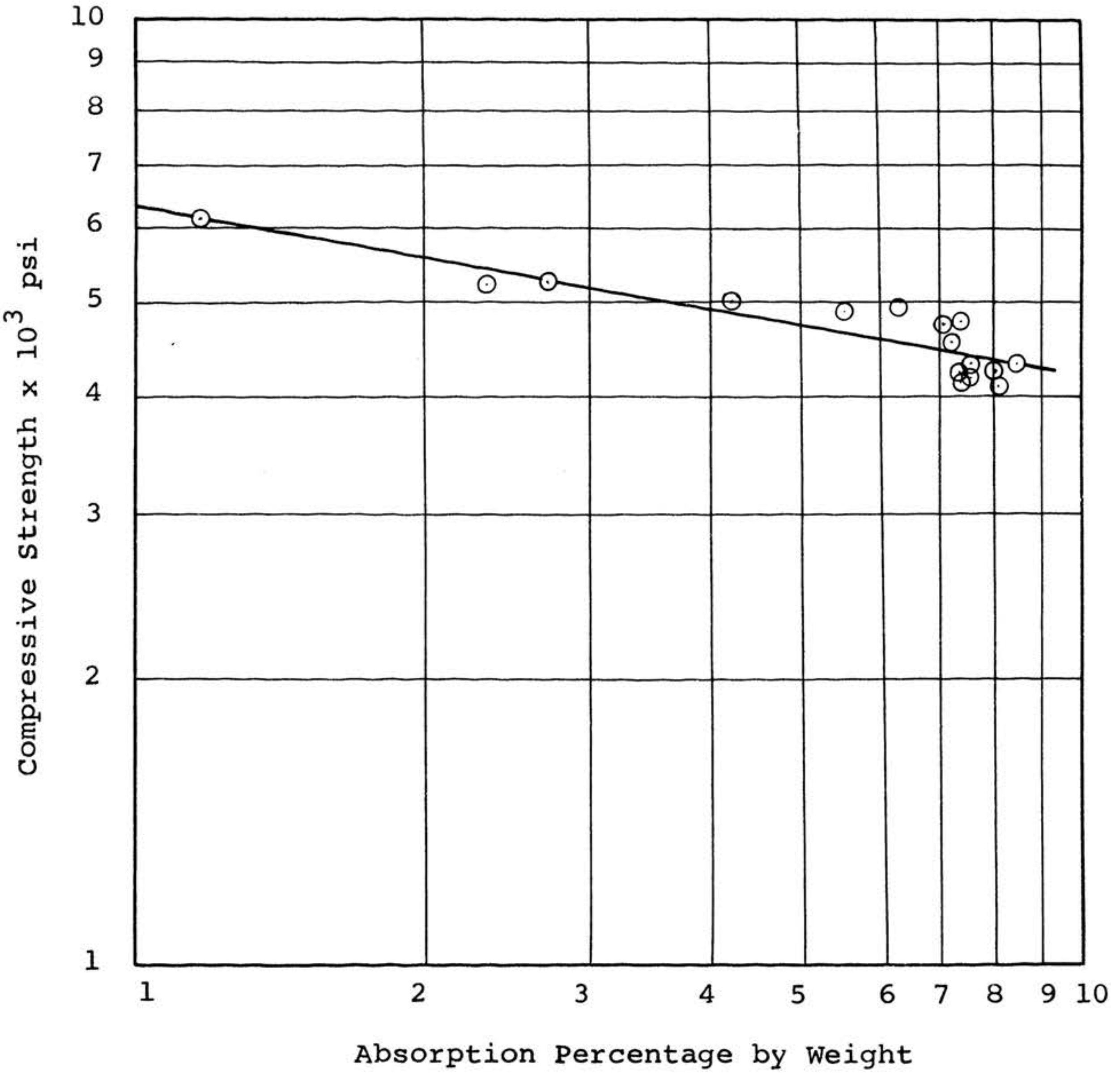


Figure 1 Absorption vs Compressive Strength for Colorado Limestone (1).

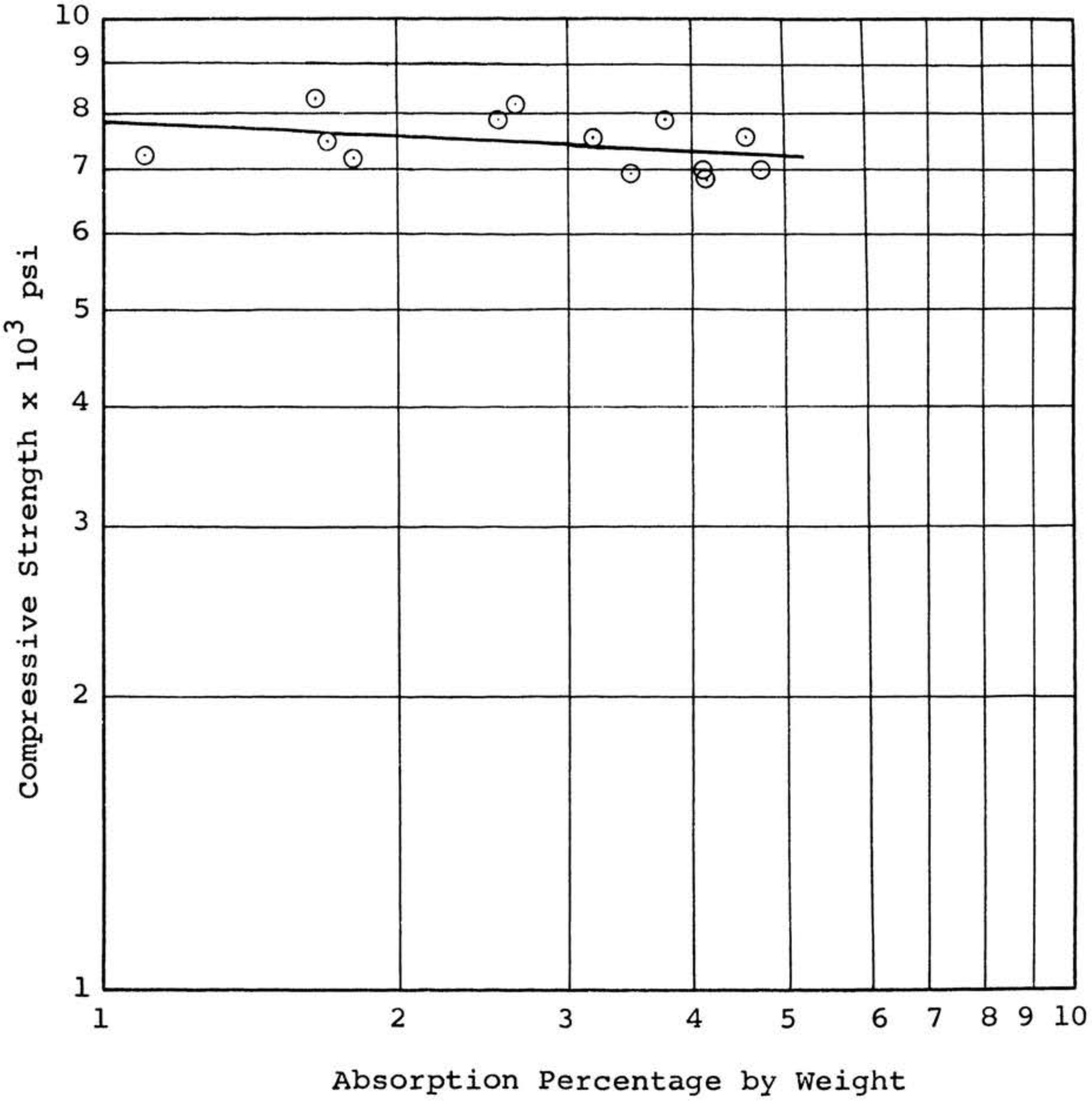


Figure 2 Absorption vs Compressive Strength for Indiana Limestone (1).

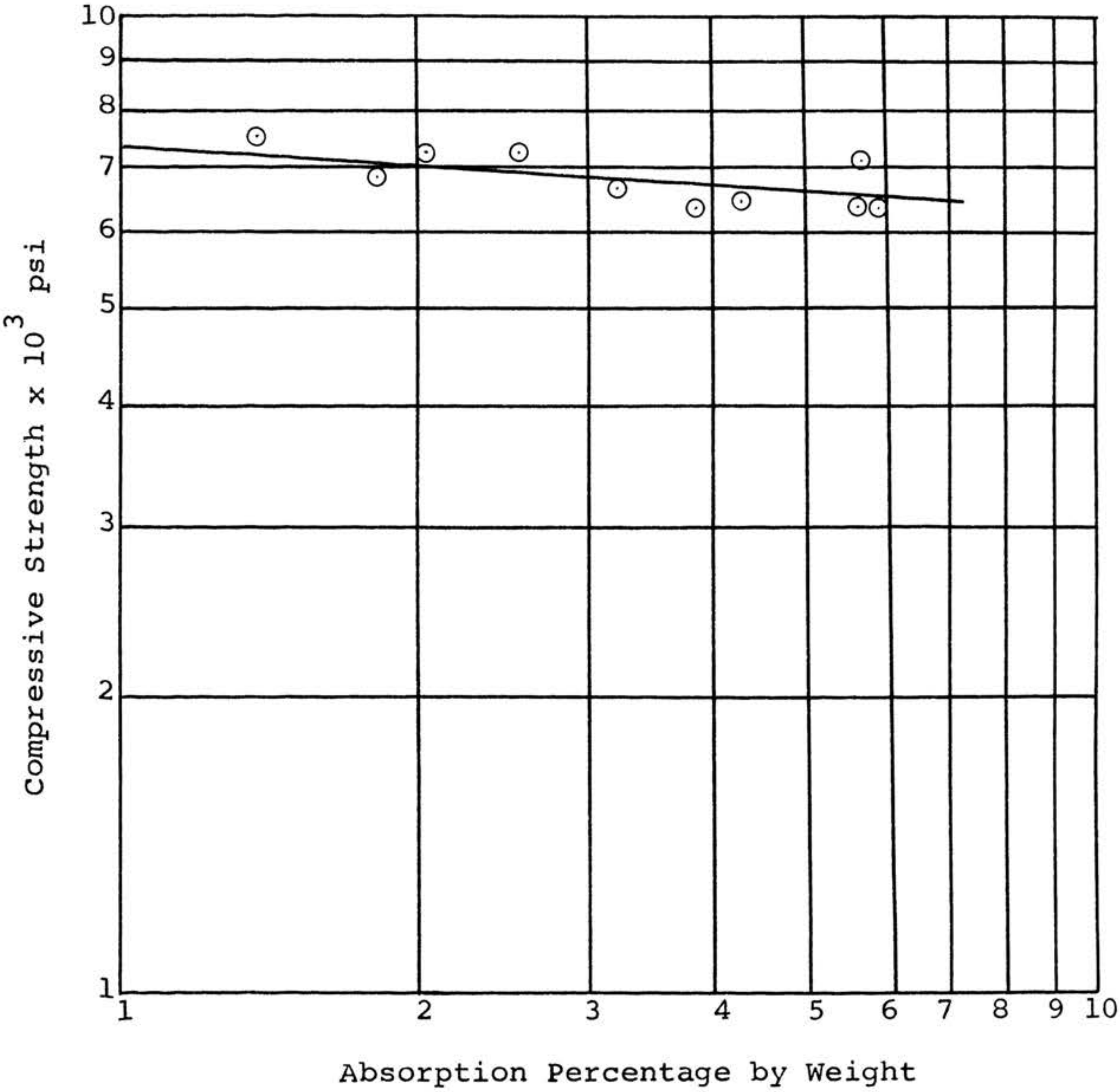


Figure 3 Absorption vs Compressive Strength for Texas Limestone (1).

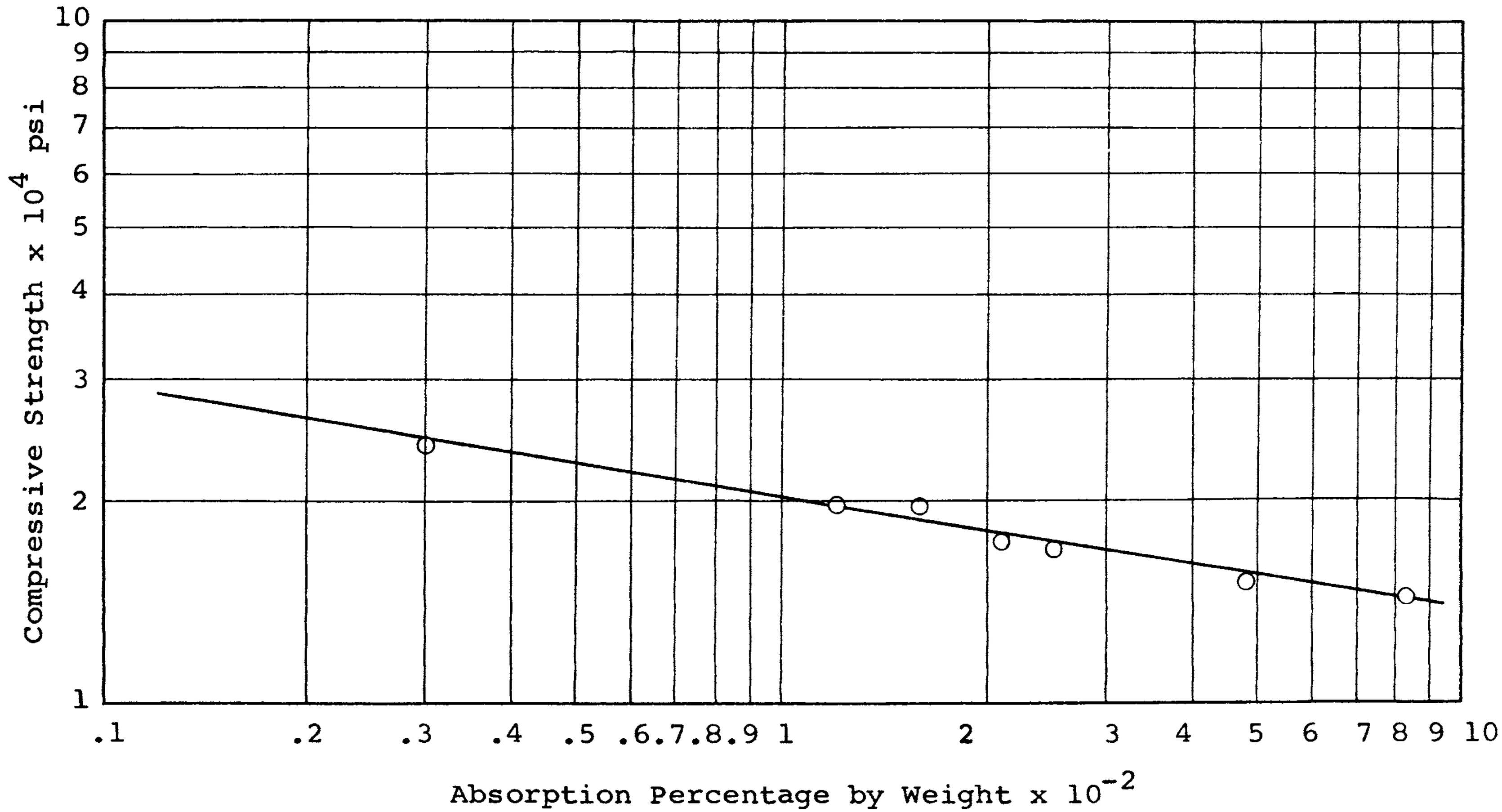


Figure 4 Absorption vs Compressive Strength for Quartzitic Shale (2).

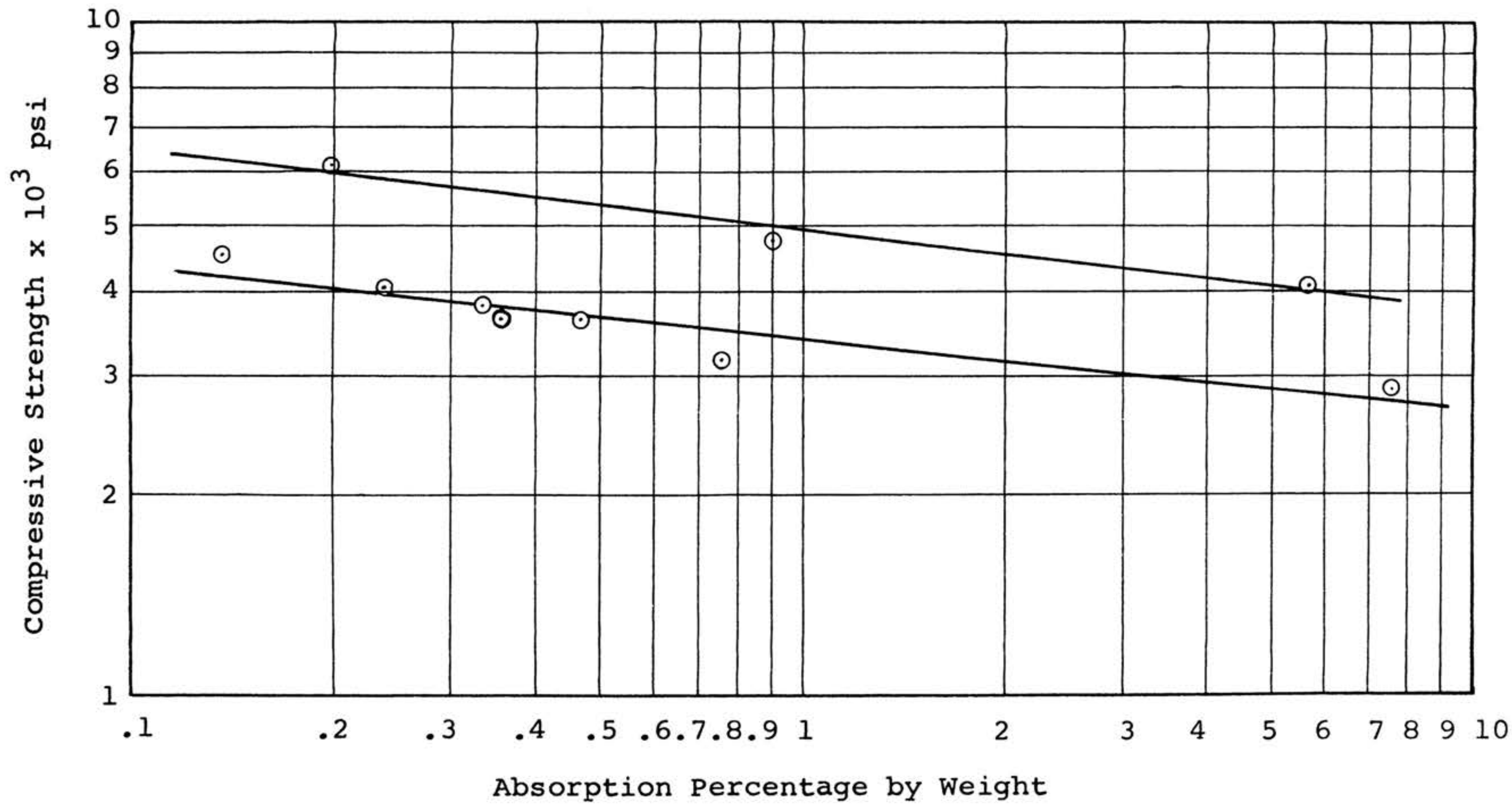


Figure 5 Absorption vs Compressive Strength for two Quartzitic Sandstones. (2)

Table 1 Effect of Moisture Content on Compressive Strength (3)

Ratio of oven-dried and saturated compressive strength to air-dried compressive strength

Moisture Condition	Marble	Limestone	Granite	Sandstone 1	Sandstone 2	Slate	Average
Oven-dried	1.01	1.03	1.07	1.01	1.18	1.06	1.06
Air-dried	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Saturated	0.96	0.85	0.92	0.90	0.80	0.85	0.88

the change in relative strengths was less pronounced (3). It appears that the specimens tested by the U. S. Bureau of Mines (3) with the lower porosity were not saturated at the time of testing and would show a greater decrease in strength had they remained in a submerged condition for a longer period of time.

2. Triaxial Compression Test

The results of triaxial compression tests on quartzitic shale (Fig. 6) and quartzitic sandstone (Fig. 7) shows a definite change in the cohesiveness of the specimens under a varied moisture content, with only a slight change in the angle of friction. It has been concluded that the increase in moisture content primarily reduces the strength of the material due to a reduction in the uniaxial tensile strength which is, in turn, a function of the molecular cohesive strength of the material (2).

3. Surface Free Energy

The molecular cohesive strength, σm, of an elastic material, according to Orowan (13), is given by:

$$\sigma m = \sqrt{\frac{2 \gamma E}{a}}$$

where: γ = the surface free energy of the material.

E = Young's Modulus.

a = spacing between neighboring atomic planes.

This may also be associated to Helmholtz's double

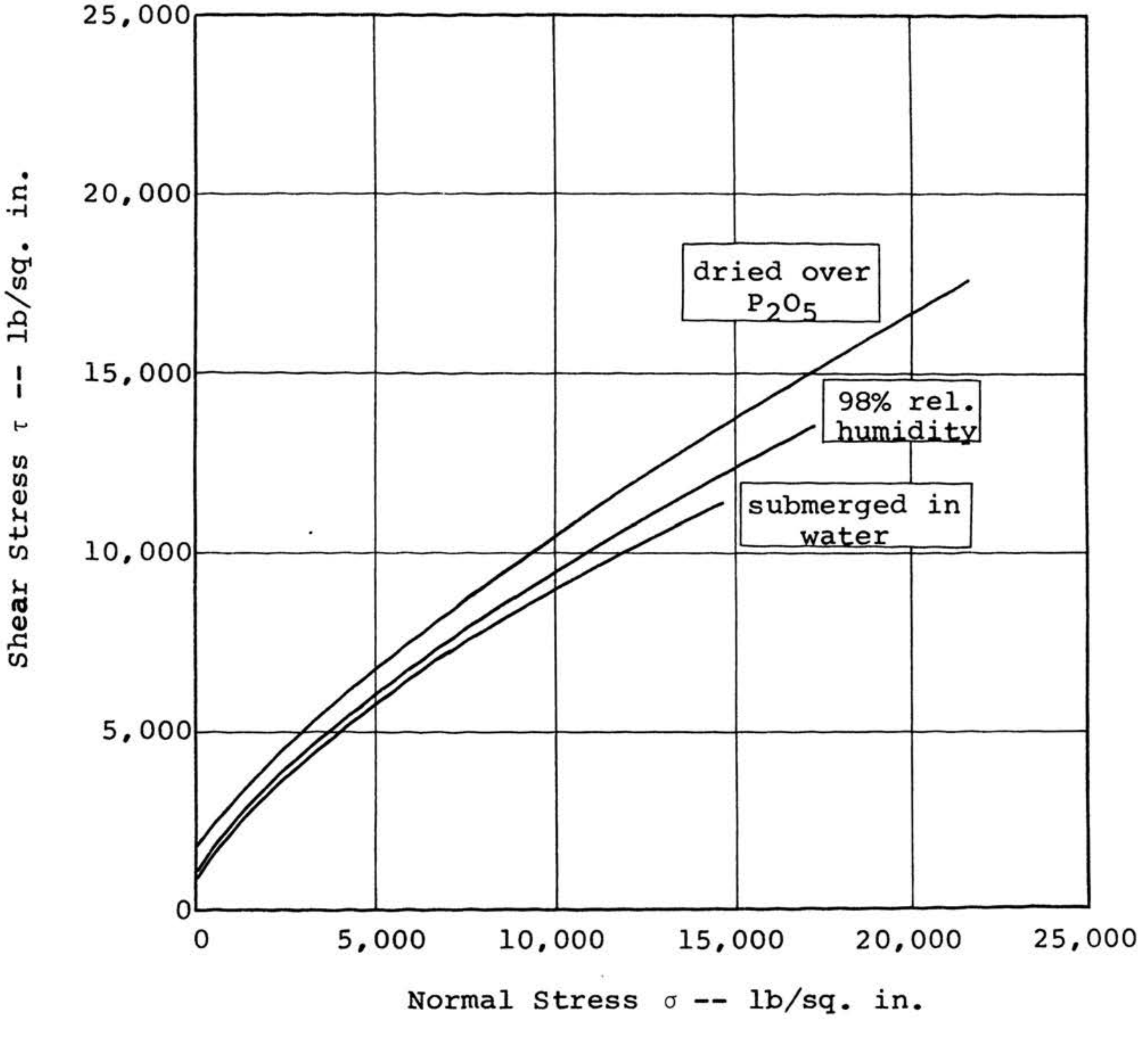


Figure 6 Mohr Fracture Envelopes for Quartzitic Sandstone at Three Moisture Contents (2).

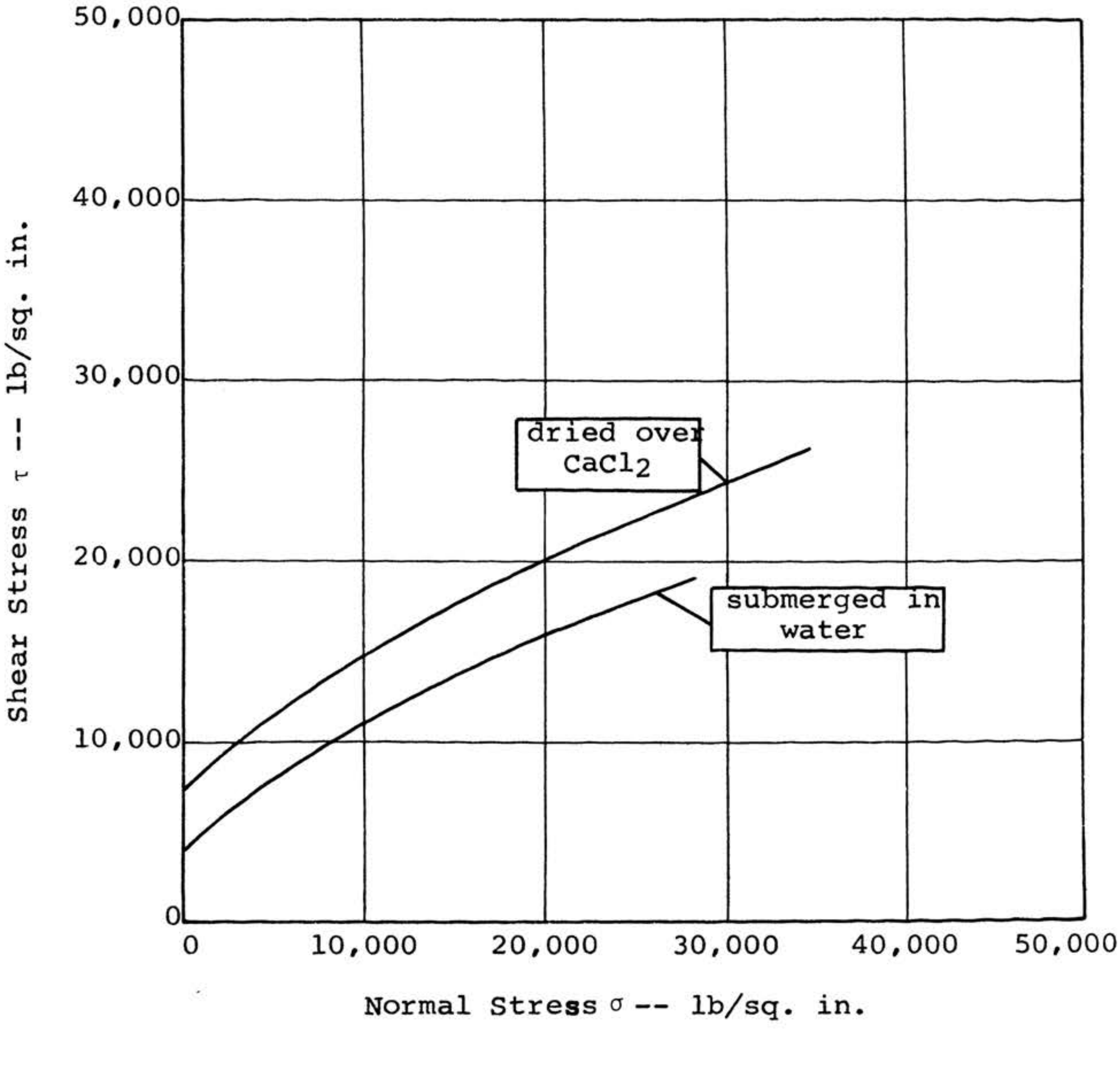


Figure 7 Mohr's Fracture Envelopes for Quartzitic Shale at Two Moisture Contents (2).

layer theory (14) that in a clay material, where the platy structure is broken at the ends, a predominately positive charge exists within the structure, tending to attract negative hydroxyl ions to the boundary. This layer of hydroxyl ions is known as the rigid layer. In turn, these negative ions are unsatisfied and attract cations such as hydrogen, sodium, calcium, etc. These cations are held to the rigid layer and are hydrophyllic and tend to attract and hold free moisture. The layer of cations and water is known as the diffuse layer. Here the surface free energy, or the potential for the rigid layer to hold the diffuse layer, is known as the Zeta Potential, Z, which may be written as:

$$Z = \frac{4 \pi u L Ve}{E \varepsilon}$$

where: Ve = fluid velocity.

E = electrical potential.

ε = dielectric constant of the fluid.

u = fluid viscosity.

L = distance between electrodes (soil particles).

The surface free energy is of definite importance to the strength of a material (2). One means of demonstrating the effect of surface free energy is by submerging the specimens in liquids possessing different surface tensions (Fig. 8). The surface free energy of a solid submerged in a liquid is partially satisfied because of the surface

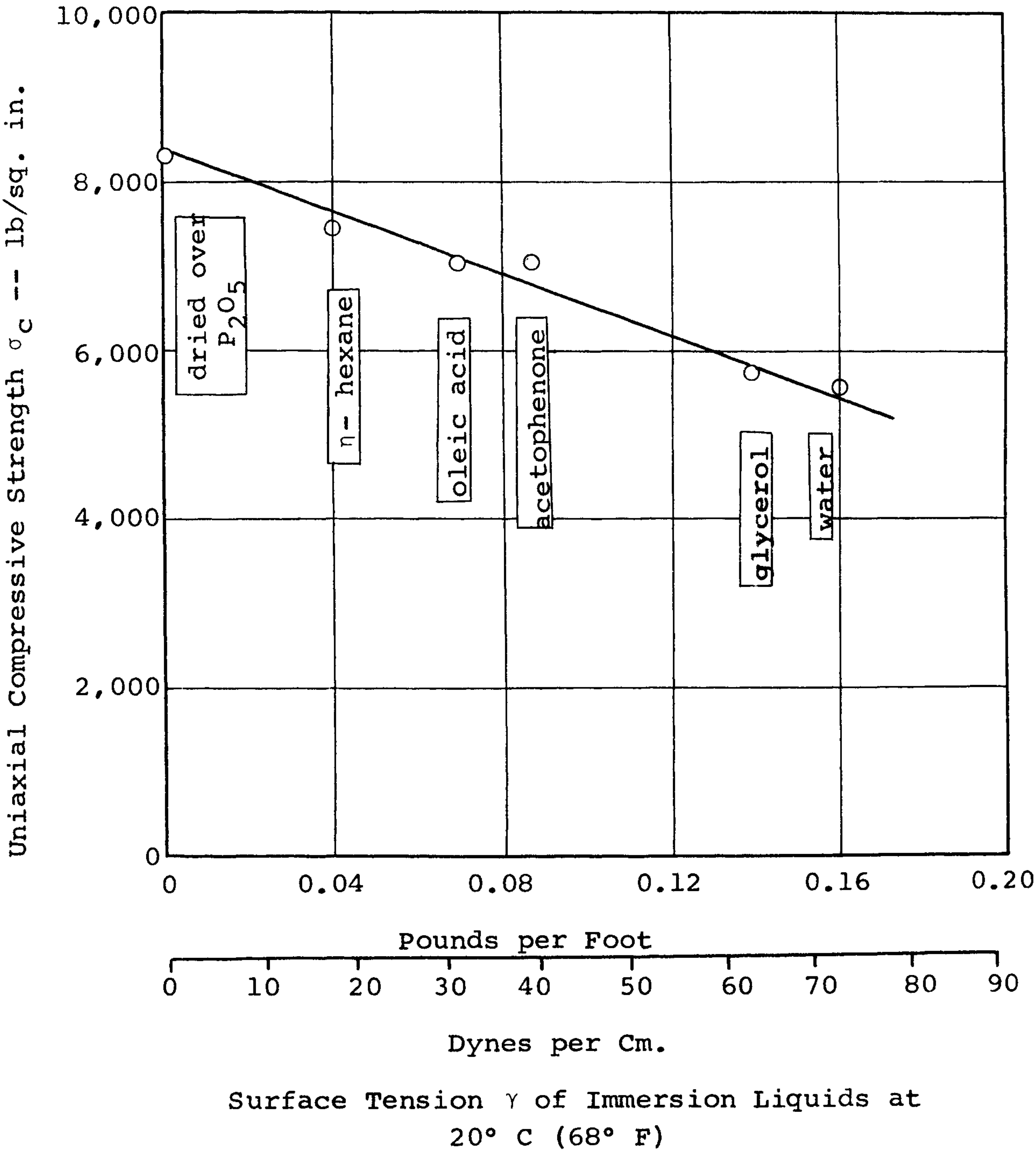


Figure 8 The Influence of the Surface Tension of Immersion Liquids on The Strength of Quartzitic Sandstone Specimens (2).

tension of the liquid. The higher the surface tension of the liquid, the closer the liquid molecules will be drawn together, and the more the free energy on the surface of the particles will be satisfied. Therefore, as the surface tension of the liquid increases, the more the surface free energy will be satisfied, and the molecular cohesive strength will decrease.

4. Young's Modulus

As seen from Orowan's equation, the molecular cohesive strength is proportional to the square root of Young's Modulus. In tests run at Colorado School of Mines (1), an increase in the moisture content of a specimen decreased the modulus of elasticity for limestone specimens (Figs. 9, 10, 11). It has also been found by others that Young's Modulus increases with an increasing moisture content in marble and granite (3).

Marble and granite have a much lower porosity than limestone and other sedimentary rock. The marble and granite act more elastically and show a tighter bonding. This seems to indicate that the water, being incompressible and prevented from draining by the small pore space, resists a reduction in the voids. There is a reduction in strength with an increased moisture content of the marble and granite which would indicate that interior stresses produced by the pore water pressure caused failure.

The higher strain in the sedimentary specimen at higher moisture content under the same load indicates a detrimental

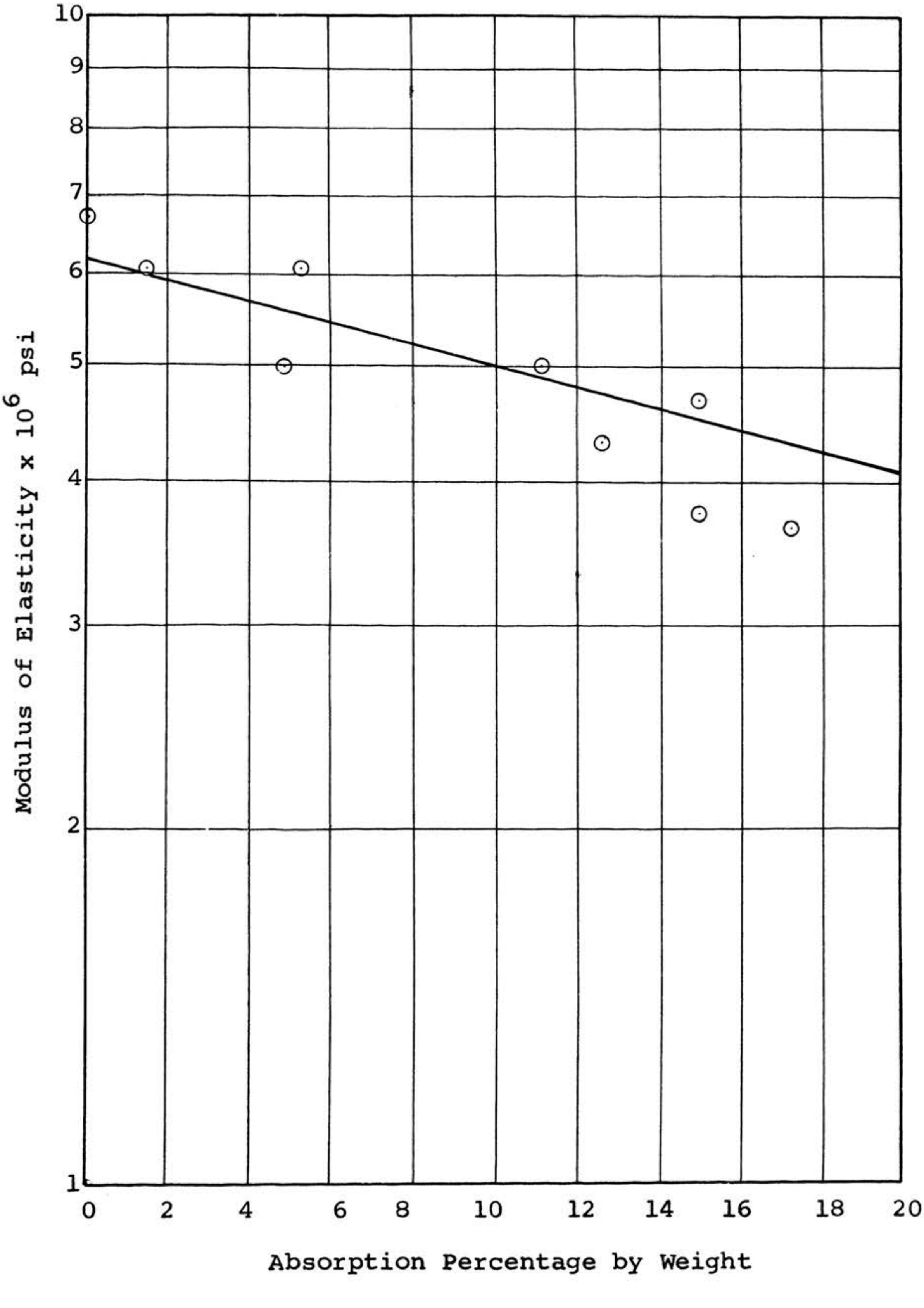


Figure 9 Absorption vs Modulus of Elasticity for Colorado Limestone (1).

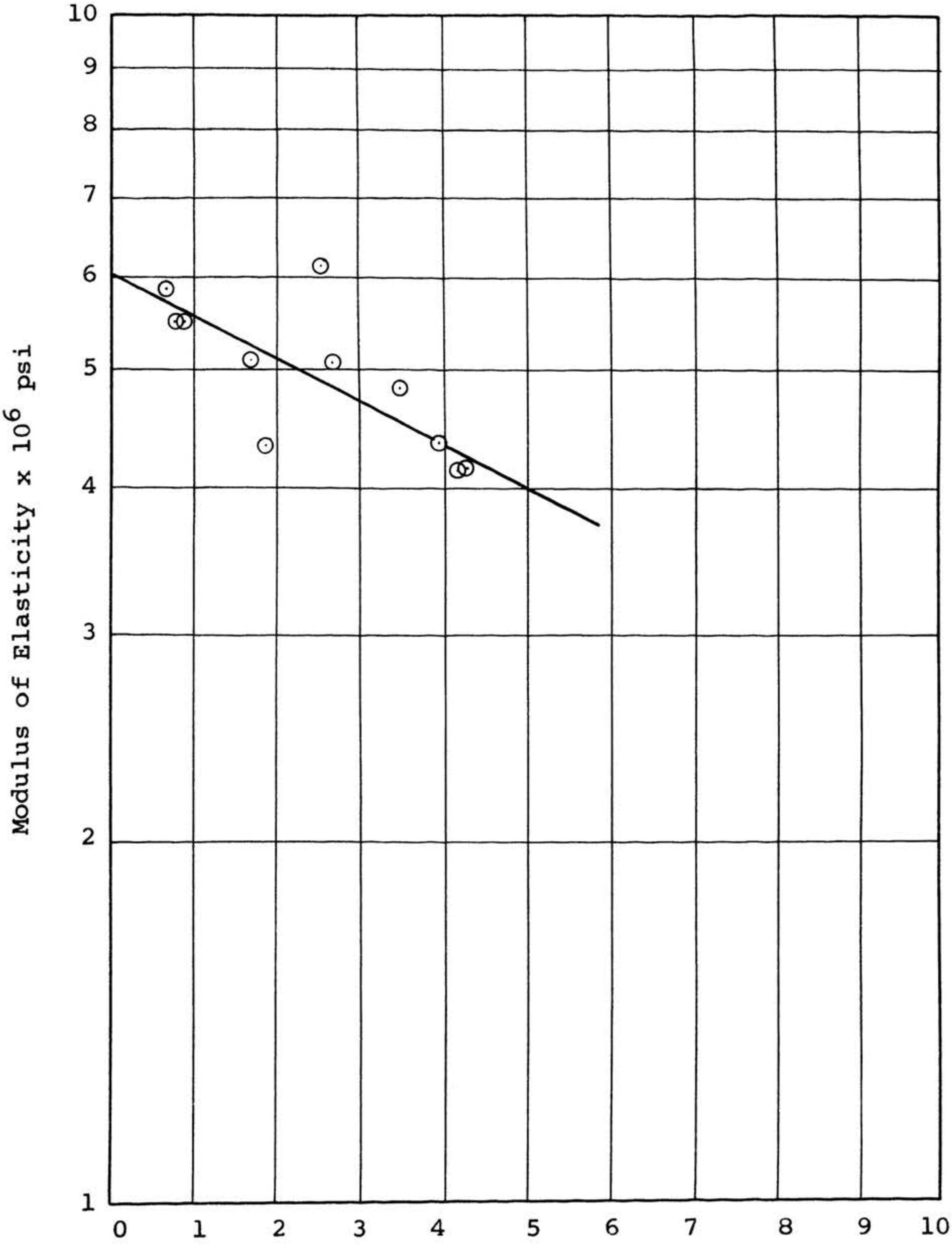


Figure 10 Absorption vs Modulus of Elasticity for Colorado Limestone (1).

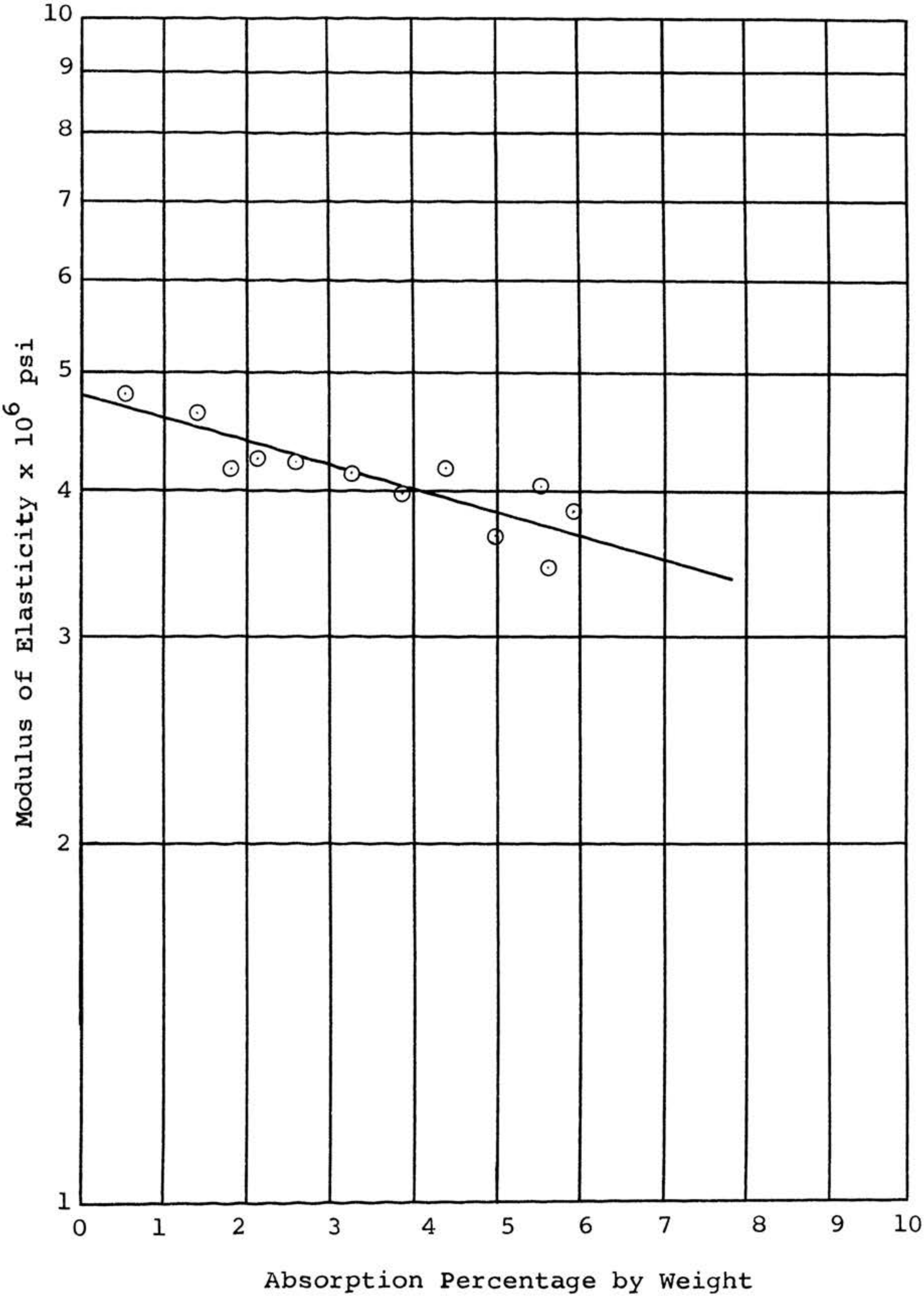


Figure 11 Absorption vs Modulus of Elasticity for Texas Limestone (1).

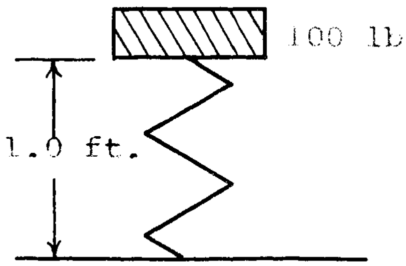
effect of moisture on the stability of the matrix. The structure of the material breaks down at a lower load with a higher moisture content.

B. Soil Mechanics Theory

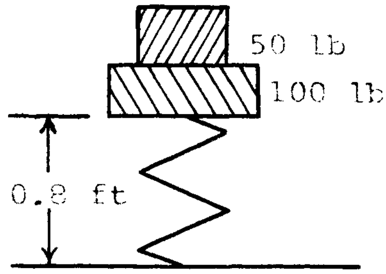
1. Pore Water Pressure

The strain occurring after a load is applied to a speciman is due to closure of the void space previously occupied by air. This must be the major area of movement under a load, for the compression of the grains and the interstitial moisture is small. If the void area in the rock or soil mass was completely filled by water, the initial strain would be small since the moisture cannot freely escape to relieve the pressure imposed.

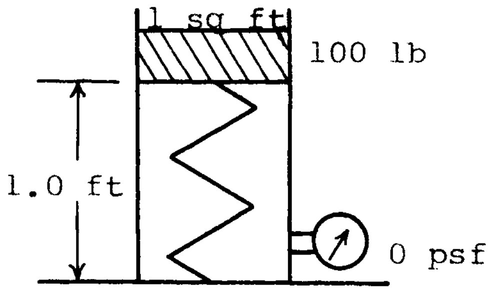
The stress carried by a saturated speciman can be demonstrated by the analogy of a spring, piston, and cylinder (6). To the top of the spring (Fig. 12a), a piston, whose cross-sectional area is 1 square foot and whose weight is 100 pounds, is attached. The length of the spring under this condition is 1.0 feet. A weight of 50 pounds is then applied to the spring (Fig. 12b) and the spring is compressed immediately to a length of 0.8 feet. This same condition may be applied in a closed cylinder (Fig. 12c) which is filled with water that is initially under no pressure. The load of 50 pounds is then applied (Fig. 12d), but there will be no appreciable compression of the spring since the added load is supported by the water. The spring still supports the 100 pound load but does not contribute to the support



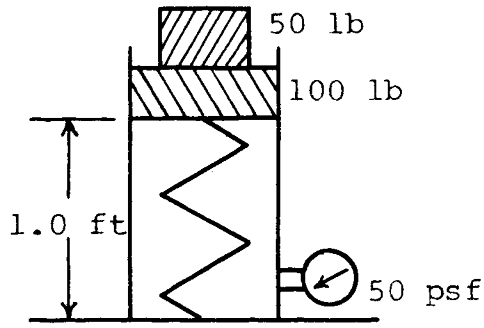
a. Spring with initial load



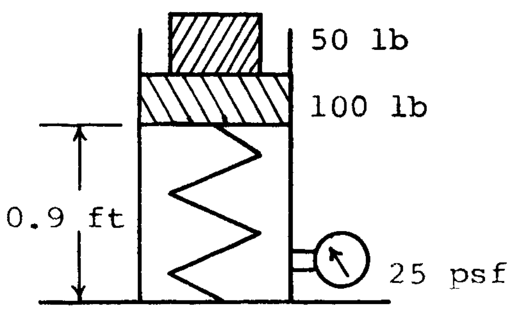
b. Spring compressed by added load



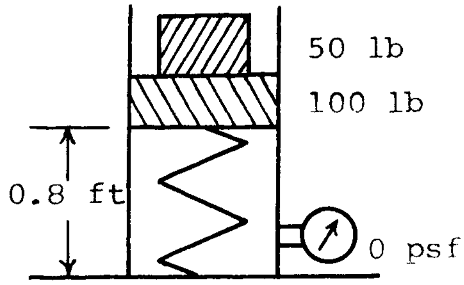
c. Spring with 100 lb initial load in closed cylinder of water. Water pressure is zero



d. Spring supports 100 lb load. Water pressure of 50 psf supports 50 lb load



e. Spring supports 125 lb load. Water pressure of 50 psf supports 50 lb load



f. Spring supports 150 lb. Water pressure is zero

Figure 12 Piston and Spring Analogy (6).

of added load. This water pressure is neutral stress and is given the symbol u. The total load of 150 pounds is denoted by σ, and the load supported by the spring represents the intergranular stress and is given the symbol $\bar{\sigma}$. The equation for the total load supported is:

$$\sigma = \bar{\sigma} + u$$

$$150 = 100 + 50$$

If the cylinder is allowed to drain (Fig. 12e) so that spring shortens to 0.9 feet and the water pressure is reduced to 25 psf the condition which now exists is:

$$150 = 125 + 25$$

When an additional 0.1 cubic feet of water is allowed to leak out, the spring will shorten to 0.8 feet (Fig. 12f) under this condition, the spring will carry the 150 pounds.

This analogy similates the reaction of a saturated soil or rock to a load. The spring represents the grain structure and the cylinder with water represents the saturated pores. As a load is applied to the speciman, the load is initially carried by the pore water. When the water seeps out and the soil compresses, the grain structure supports the load and the neutral stress or water pressure becomes zero.

2. Capillary Moisture

The resultant force associated with an air-liquid surface is the property known as surface tension of a liquid (5). The molecular attraction of molecules on the surface of a liquid differs from that of molecules in the interior of the mass of a liquid. This occurs because there is a difference in molecular attraction between one water molecule and another water molecule, and between a molecule of water and a molecule of air. Thus, the set of forces acting on a particle in the interior of a mass of water (Fig. 13), as at point A, differs from the set of forces acting on a particle in the air-water surface, as at point B. The resultant force is directed inward due to this difference in attraction forces. As a result, the mass of liquid attempts to occupy the least possible area of the container. The force required to oppose this tendency to contract is called surface tension.

A compressive stress is developed in the grains due to the capillary action. The capillary water is held up by the surface of the grains, and the grains are subjected to a compressive stress due to the weight of the column of water. According to Spangler (5), "It is as though a man were to hang by his hands inside a chimney. The chimney supports the man, and the reaction to his weight causes a compressive stress in the walls of the chimney." This is similar to the capillary moisture in the soil pores. The strength of the mass is increased due to the compressive stresses in the soil structure which are directed inward.

Capillary potential is a quantitative stress property of a soil or rock which expresses its potential for attracting capillary moisture. This value, denoted by Ψ, which is always less than zero, is expressed by:

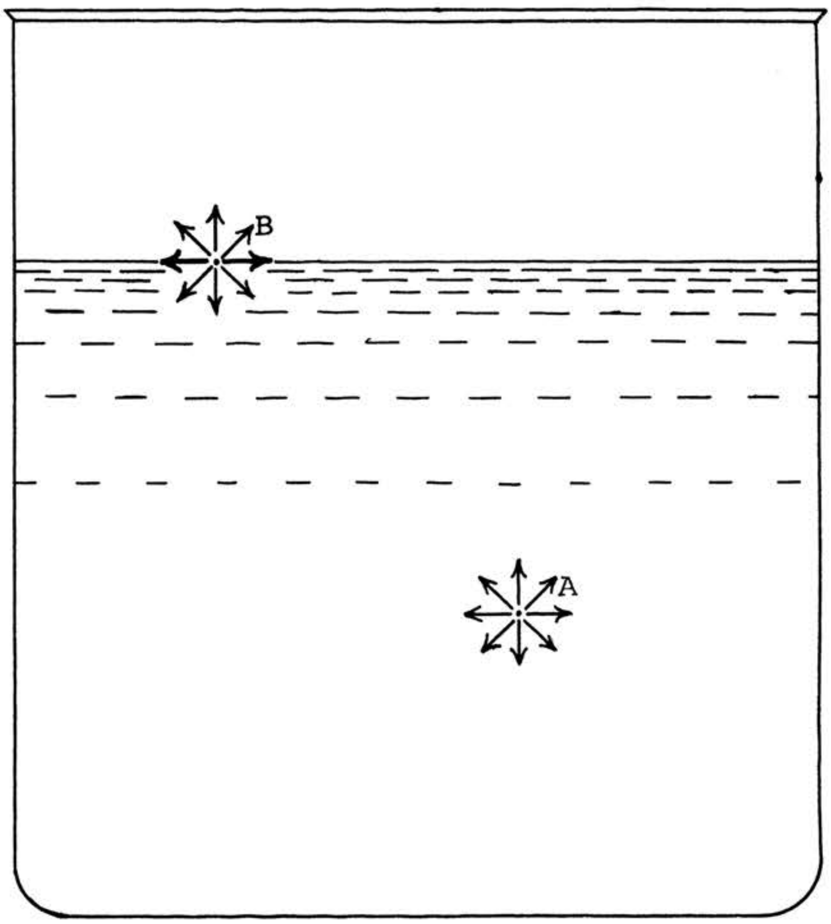


Figure 13 Surface Tension (5).

$$\Psi = -T \left(\frac{1}{R_1} + \frac{1}{R_2} \right)$$

where: T = the surface tension of the liquid.

R_1 and R_2 = the radii of curvature of the warped or saddle-shaped surface.

Because of their influence on the radii of curvature, the moisture content, the size of the grains, the angle of contact, and the state of packing affect the value of capillary potential.

a. Moisture Content

If the amount of moisture between two grains is decreased (Fig. 14), the water will recede further into the interstices between the grains and the curvature of the air-water interface will increase, causing a decrease in the radii. The neutral stress or pore water stress decreases and becomes more negative (tension is given a minus sign). The total stress has not changed, therefore the intergranular stress must increase by an equivalent amount.

b. Grain Size

If equal weights of a fine grained material and a coarse grained material have the same moisture content, the fine grained soil will have a larger surface area and a small radius and larger radius of curvature (Fig. 15). This indicates that, under the above conditions, the fine grained material will have a lower pore water pressure and a higher

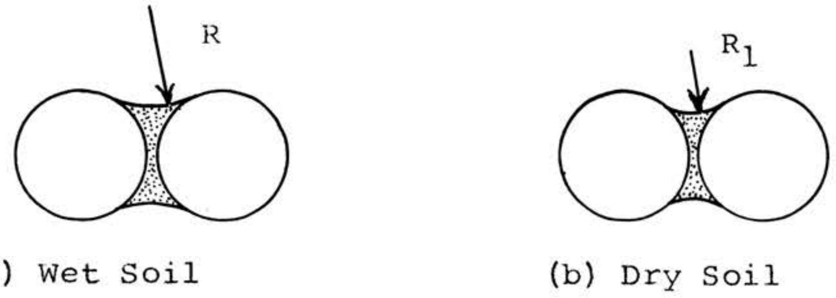


Figure 14 Effect of Moisture Content Upon Curvature of Air-Water Interface.

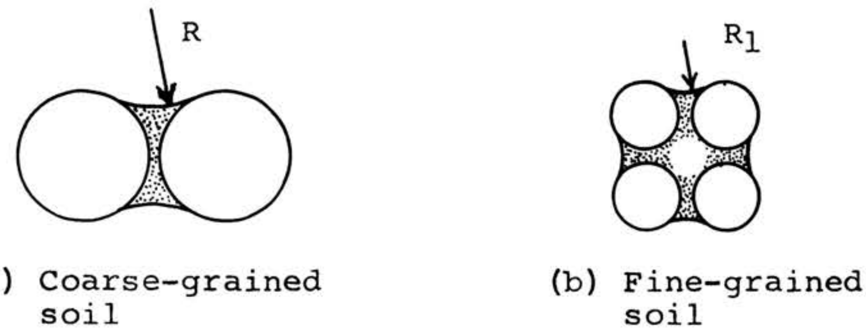


Figure 15 Effect of Particle Size Upon Curvature of Air-Water Interface.

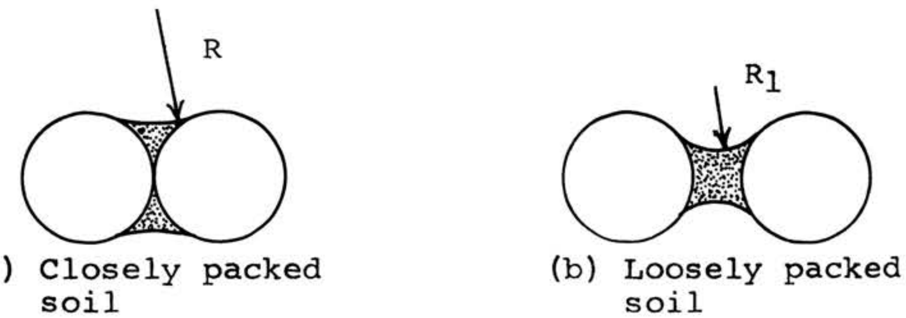


Figure 16 Influence of State of Packing of Soil on Curvature of Air-Water Interface.

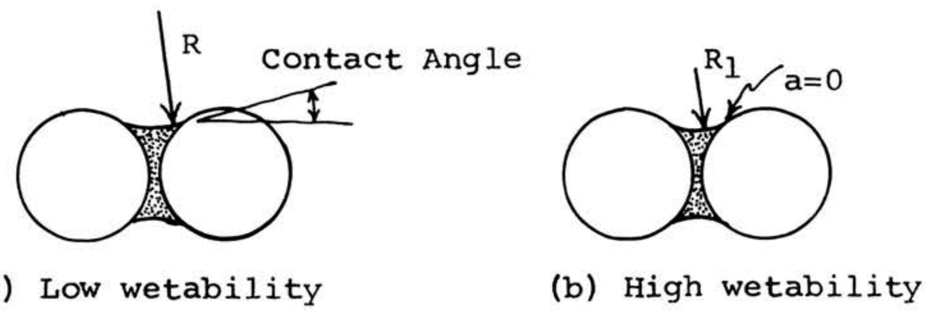


Figure 17 Influence of Wetability of Soil Grains on Curvature of Air-Water Interface.

intergranular pressure.

c. State of Packing or Degree of Consolidation

As two grains are pushed together (Fig. 16), the curvature of the meniscus will be decreased. With the same moisture content, the more compact sample will approach saturation as the air-voids are replaced by water. This indicates that the higher the porosity, in material having the same grain size and moisture content, the lower will be the neutral pressure and the higher will be the intergranular pressure.

d. Angle of Contact

The mineralogical composition of the rock or soil will determine what degree of wettability or angle of contact between the menisci and the grains can be accomplished. Keeping the other parameters constant, the greater will be the angle of contact, the lower the radius of curvature, and the lower the intergranular pressure (Fig. 17).

3. Shear Strength

From Mohr's envelope of failure (5, 6) the equation for shear strength, τ, may be written:

$$\tau = C + \bar{\sigma} \tan \phi$$

where: C = cohesion or the shear strength where the normal stress is zero, in psi.

$\bar{\sigma}$ = intergranular or effective stress in psi.

ϕ = internal angle of friction or the slope of the failure envelope.

The angle of friction is dependent on the following:

1) the coefficient of friction between the minerals.
2) the surface roughness.
3) the angle of contact between the grains.

The increase in intergranular stress associated with capillary tension produces a hydrostatic compressive stress in the mass. The increased shearing resistance along any section will be:

$$\Delta\tau = -u_w \tan \phi = \Delta\bar{\sigma} \tan \phi$$

where: u_w = the tension in the pore water.

In the unconfined compression test (Fig. 18), the cohesion, which appears for specimens affected by capillary tension and disappears completely after immersion, is referred to as apparent cohesion. The effective stresses at failure are, therefore:

$$\bar{\sigma}_1 = \Delta\sigma_1 - u_w$$

$$\bar{\sigma}_3 = -u_w$$

A sample in the saturated state is loaded axially at a rate which does not allow the speciman to drain. This means that there will be no further changes in the void ratio or water content until failure occurs. In this case, the added load, $\Delta\sigma_1$, is supported entirely by the pore water, or $u = \Delta\sigma_1$. The pore water pressure is exerted equally in all directions, a hydrostatic condition, and is therefore loaded equally in the σ_1 and σ_3 directions.

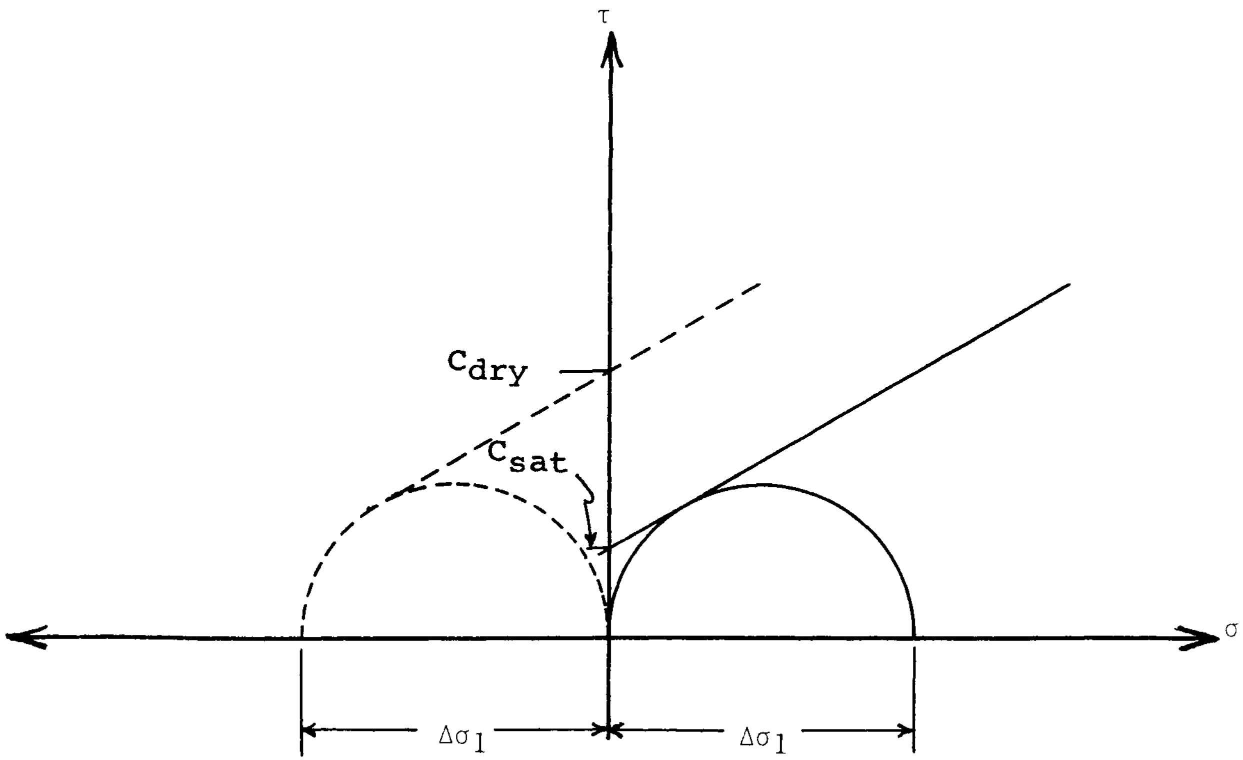


Figure 18 Mohr's Envelope and Stresses for Saturated Clay (5).

The effective stresses at failure are as follows:

$$\bar{\sigma}_1 = \bar{\sigma}_1 - u = \sigma_3 + \Delta\sigma_1 - \Delta\sigma_1 = \sigma_3$$

$$\bar{\sigma}_3 = \sigma_3 - u = \sigma_3 - \Delta\sigma_1$$

Under a uniaxial load:

$$\bar{\sigma}_1 = 0$$

$$\bar{\sigma}_3 = - \Delta\sigma_1$$

The specimen fails in tension equal to the axial stress (Fig. 18).

The Mohr's envelope for a partially saturated clay (Fig. 19) is ordinarily curved with a decreasing slope at increasing normal stresses. This indicates that capillary tension and pore water pressures have a definite effect on the strength of saturate clays. An apparent cohesion is caused by the capillary tension, but as this decreases (packing of the clay platelets) a positive pore water pressure develops. If the loads were increased, the Mohr's envelope would approach a horizontal asymptote.

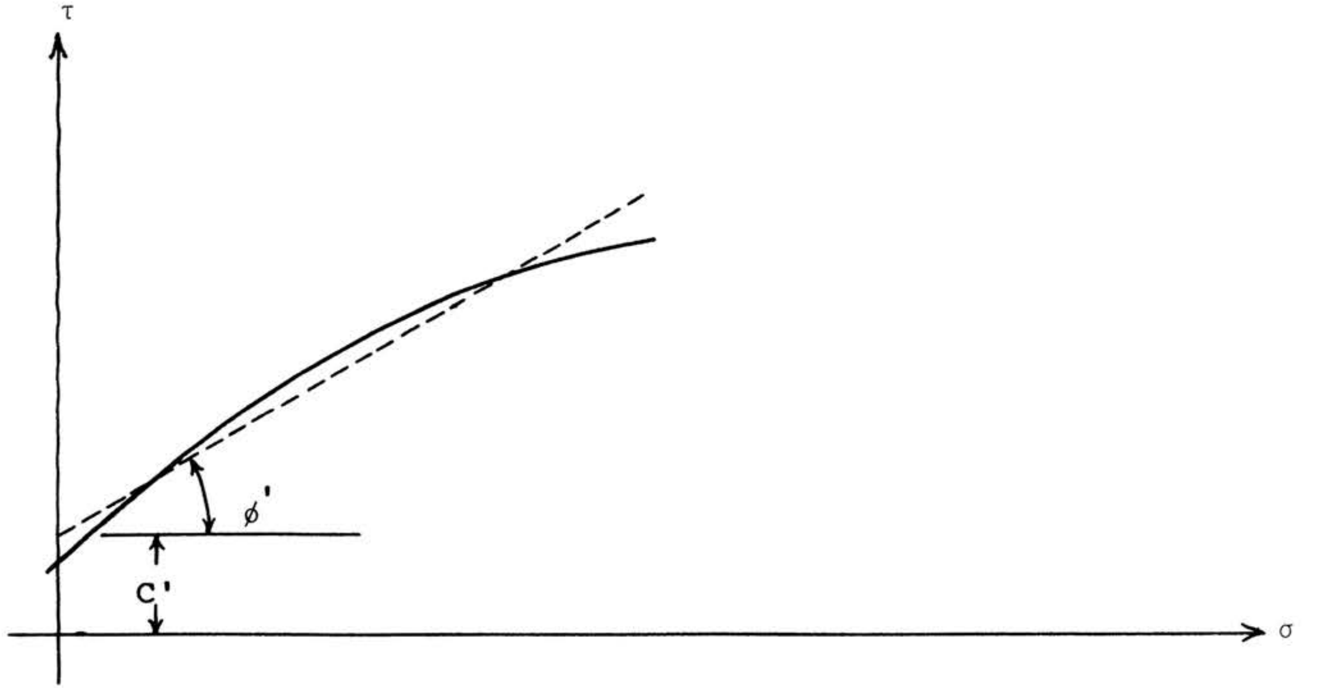


Figure 19 Mohr's Envelope of Total Stresses for Partially Saturated Clay (6).

III. TESTING PROCEDURE

A. Sample Preparation

The procedure for the preparation of test specimens was adopted from the results of tests conducted by the U.S. Bureau of Mines (3). Both "EX" (7/8-inch) and "AX" (1-1/8-inch) diameter of drill cores were used in the testing, but the results obtained from one size of core were not mixed with data from the other. This was done in order to define any deviation in strength which may have been associated with the size of core.

Because of the number of specimens expected to be handled, it was necessary to prepare the samples in bulk. A special holder (Figs. 20, 21, 22) was assembled, which was capable of retaining 40 specimens of the "AX" core and 84 specimens of the "EX" core. This holder was made to insure that the specimens were of approximately equal height and that the ends would be perpendicular to the sides when cutting and grinding.

In considering the proper height of the specimens to use in testing, results from the U.S. Bureau of Mines testing of 720 specimens with a varied length to diameter ratio was considered (3). The results from their tests (Fig. 23) indicated that a length to diameter ratio of approximately 2:1 would be preferred as a standard.

The ends of the specimens were then ground flat on a Norton Grinder (Fig. 21). The grinding wheel was

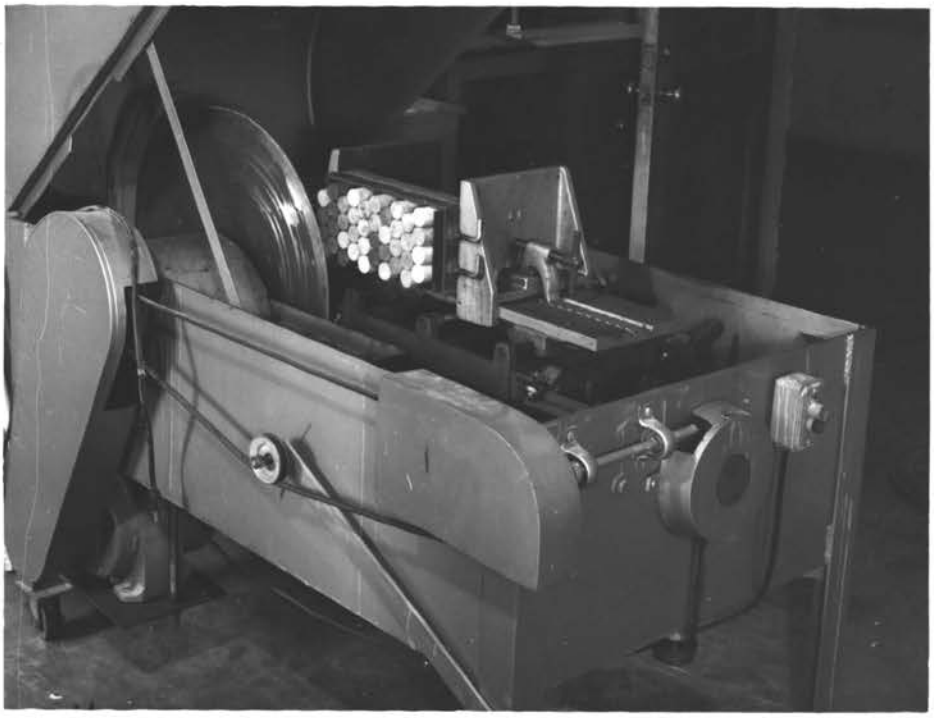


Figure 20 Cores in a Special Sample Holder Being Cut to the Same Height with a Diamond Saw.

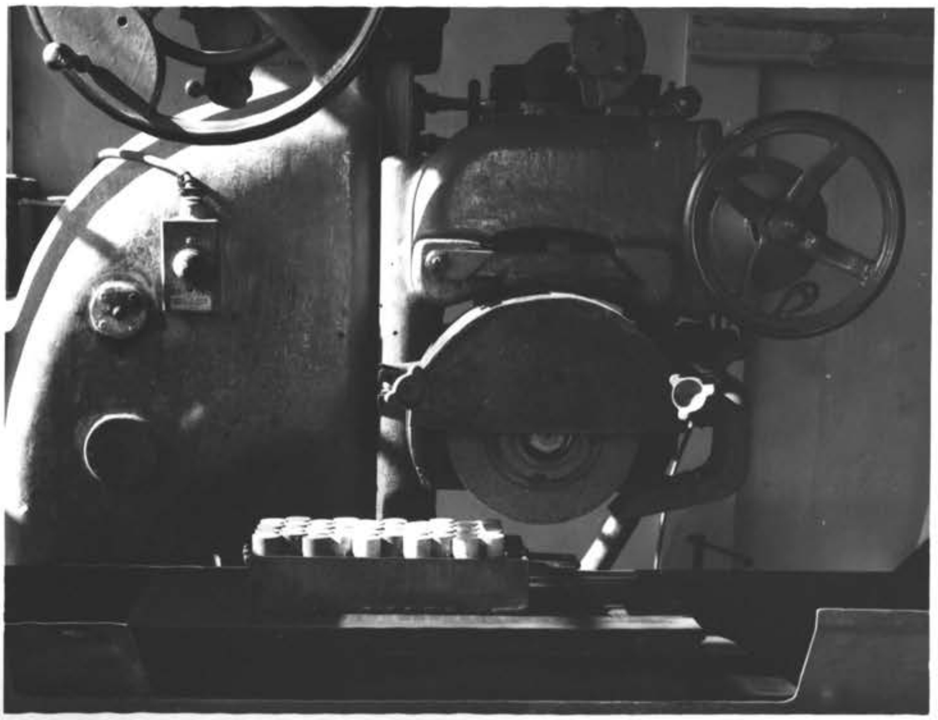


Figure 21 Sample Ends Being Ground Smooth in Norton Grinder.

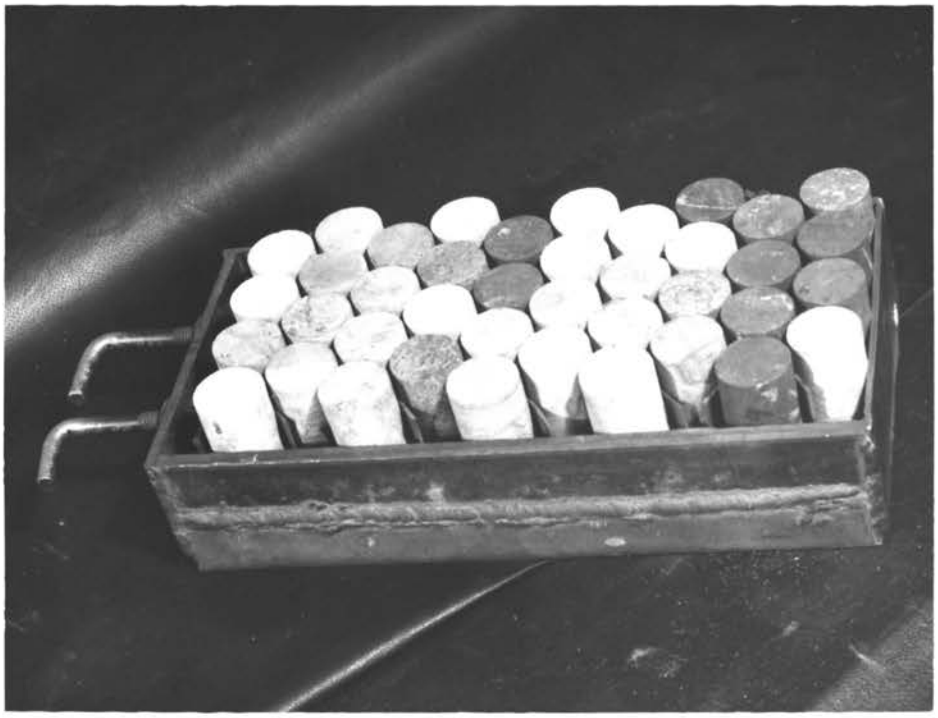


Figure 22 Finished Samples in Special Holder.

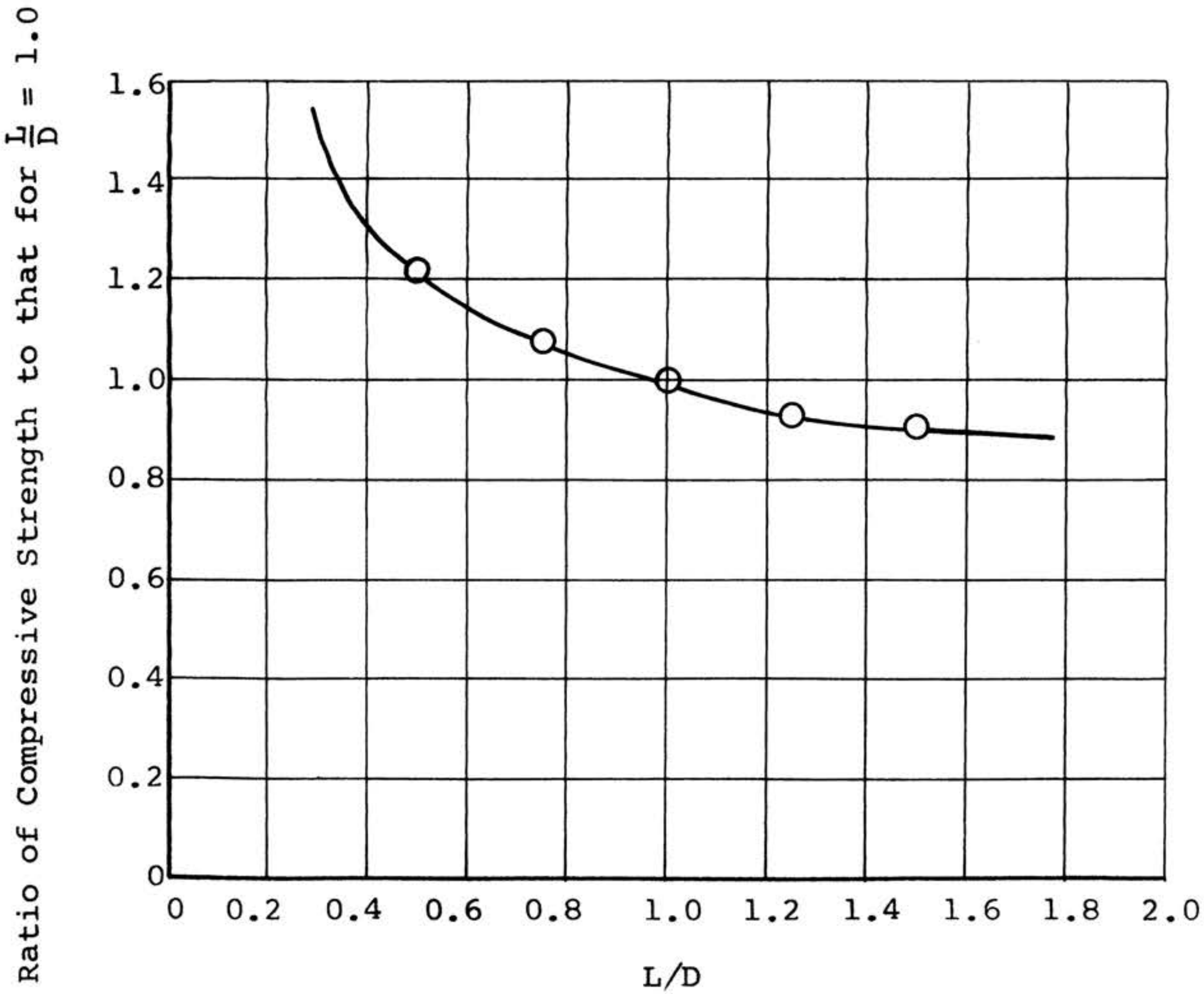


Figure 23 Effect of Ratio L/D on Compressive Strength (3).

made of alundum (aluminum oxide) with a grain size of 46 mesh. The cylindrical surfaces of the specimens were left in the as-drilled condition.

Specimens were marked with a permanent ink and stored in a room where the temperature and relative humidity was controlled. The specimens remained at a temperature of 68°F ± 2°F and a relative humidity of 45% ± 5% for 30 days, which approached a stable state. This moisture content was accepted as a zero datum to which all subsequent moisture contents were related.

Studies were conducted by the South African Council for Scientific and Industrial Research, Pretoria, of the effects of moisture on the unconfined and triaxial compressive strength of two quartzitic rocks. The procedure used in this investigation for producing a varied amount of moisture was adopted from the SACSIR paper.

It has been found (10) that a constant relative humidity can be maintained when a saturated aqueous solution is enclosed in a container and held at a constant temperature. If the relative humidity in a desiccator containing samples can be held constant at various degrees of partial saturation, then the samples would possess corresponding moisture conditions. In this manner, moisture can be induced into the pores at values ranging from dry to saturated in a relatively uniform manner when the separate containers are kept at the same temperature (20°C).

The moisture conditions which were used by SACSIR were as follows:

Saturated Solutions of	Relative Humidity at 20° C
-	$CaCl_2$ dried
$LiCl \cdot 6H_2O$	15.0%
$CaCl_2 \cdot 6H_2O$	32.3%
KNO_2	45.0%
$NaNO_2$	66.0%
NH_4Cl_2	79.5%
$Pb(NO_3)_2$	98.0%
-	Water

It was known, prior to testing, that the strength of the rock material from the Pea Ridge Mine was variable, and it was observed that the material was nonhomogeneous. In an attempt to group the specimens to reduce effects of the variable constituents, the density of each of the specimens was taken at the datum conditions. Four moisture conditions were selected to give a range of moisture contents and to have a sufficient amount of samples per group.

The first group to be tested was dried in the oven for 24 hours then placed into a desiccator containing calcium chloride, and then placed into the temperature controlled room until testing. A second group of specimens were placed in a saturated condition by arranging the samples in a container under a vacuum of 28.4 inches of mercury, then moisture was added slowly over the samples until they were submerged (Figs. 24, 25). When the emission of the air from the specimens ceased, they were considered saturated. The other two groups were placed in desiccators under conditions of 66% and 98% relative humidity (Fig. 26). This condition was maintained, at 68° F $\pm$ 2° F, for 30 days. The final moisture content of the specimens, at the time of testing, is expressed as follows:

$$\text{Moisture Content (\%)} = \frac{W_t - W_o}{W_o} \times 100\%$$

where: W_t = the weight of the specimen at the datum condition.

W_o = the weight of the specimen at the time of testing.

The amount of time the specimens were to remain under this condition was determined from tests conducted on samples of shale and quartz (2). From these graphs (Figs. 27, 28), a thirty day period was selected even though the period would be too long for material having the porosity of sandstone (15% by volume) and not long enough for the material having the porosity of shale (0.28% by volume). This period would be long enough to allow for the major moisture change to occur with an expected high degree of uniformity.

B. Materials Tested

The eight rock types tested were as follows:

<u>Pea Ridge Mine</u>, Missouri

1. Bonneterre dolomite - a fine to medium grained, light gray dolomite from the upper Cambrian sediments. Its

Figure 24 Set-Up for Saturated Specimens Under a Vacuum of 28.4-Inches of Mercury.

Figure 25 Water was Added Slowly to Allow for Complete Saturation of the Pore Area.

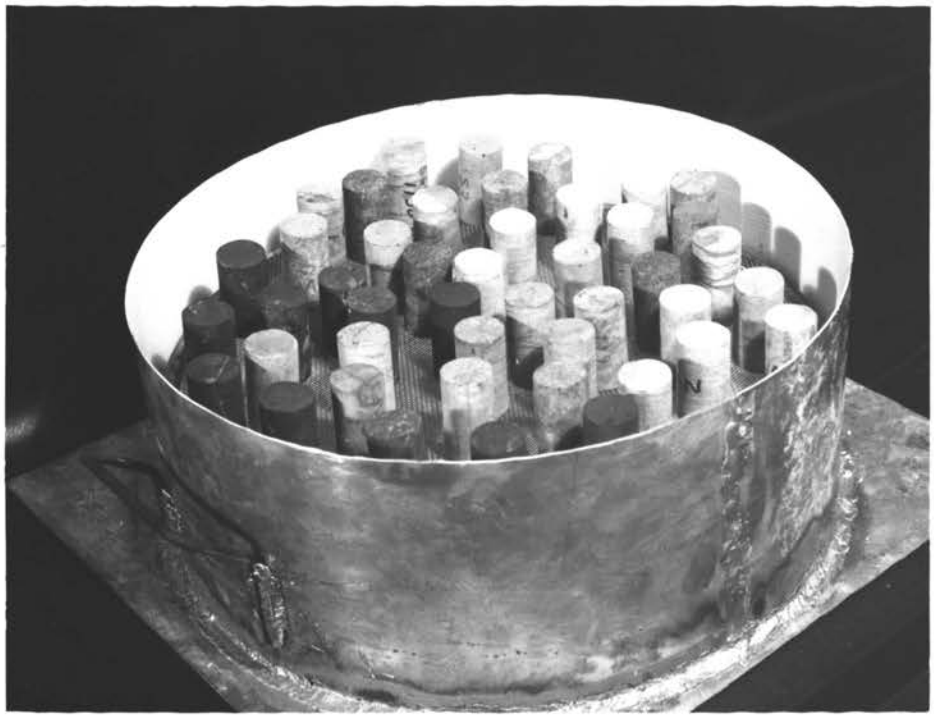


Figure 26 Desiccator, Without the Top, Containing Specimens Prepared for Testing.

Datum Weight- 45 Days in a 50% R. H. Environment

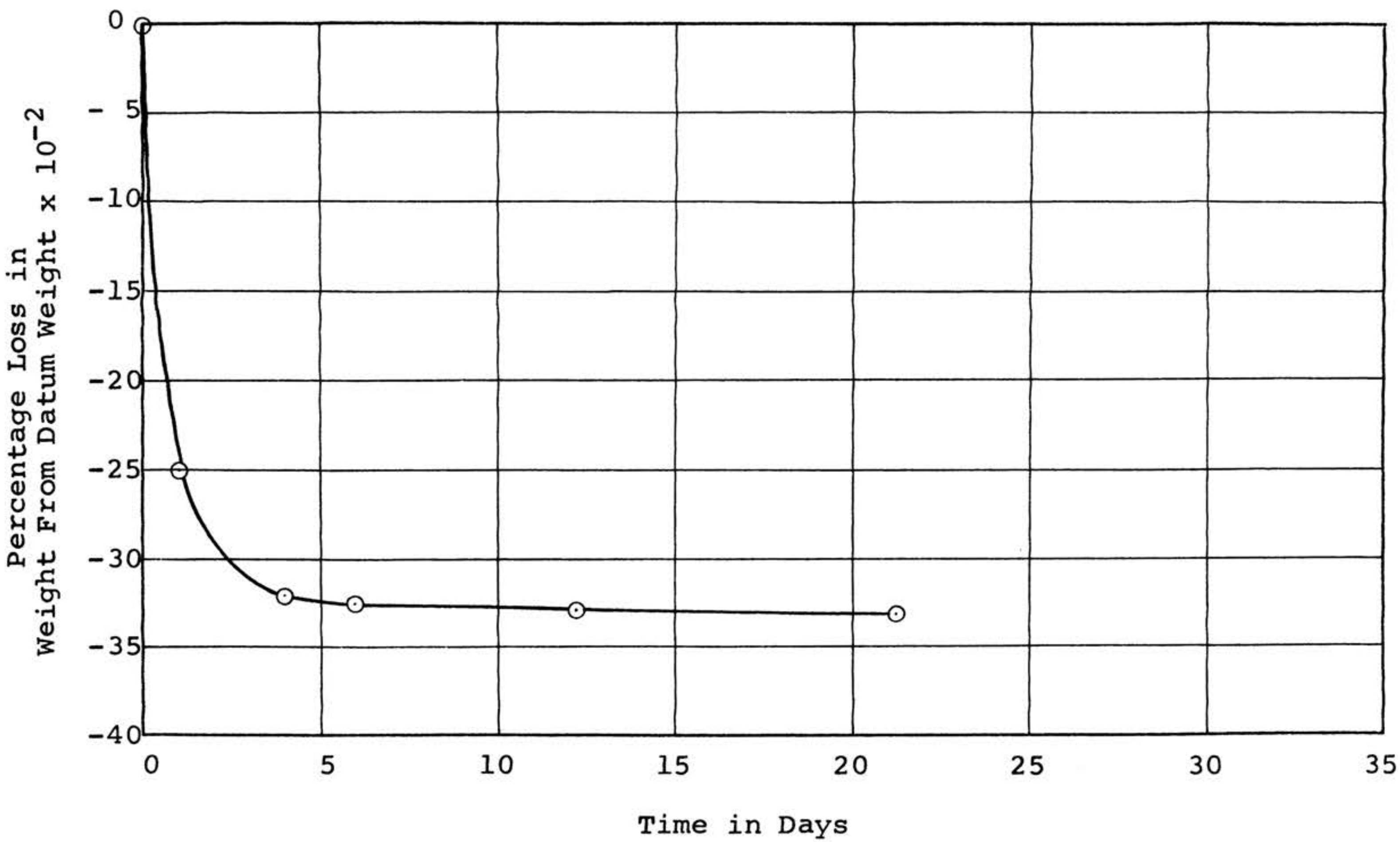


Figure 27 Typical Graph Showing Mean Percentage Loss in Weight of Quartzitic Sandstone with Time for Specimens Stored in Dry Environment (2).

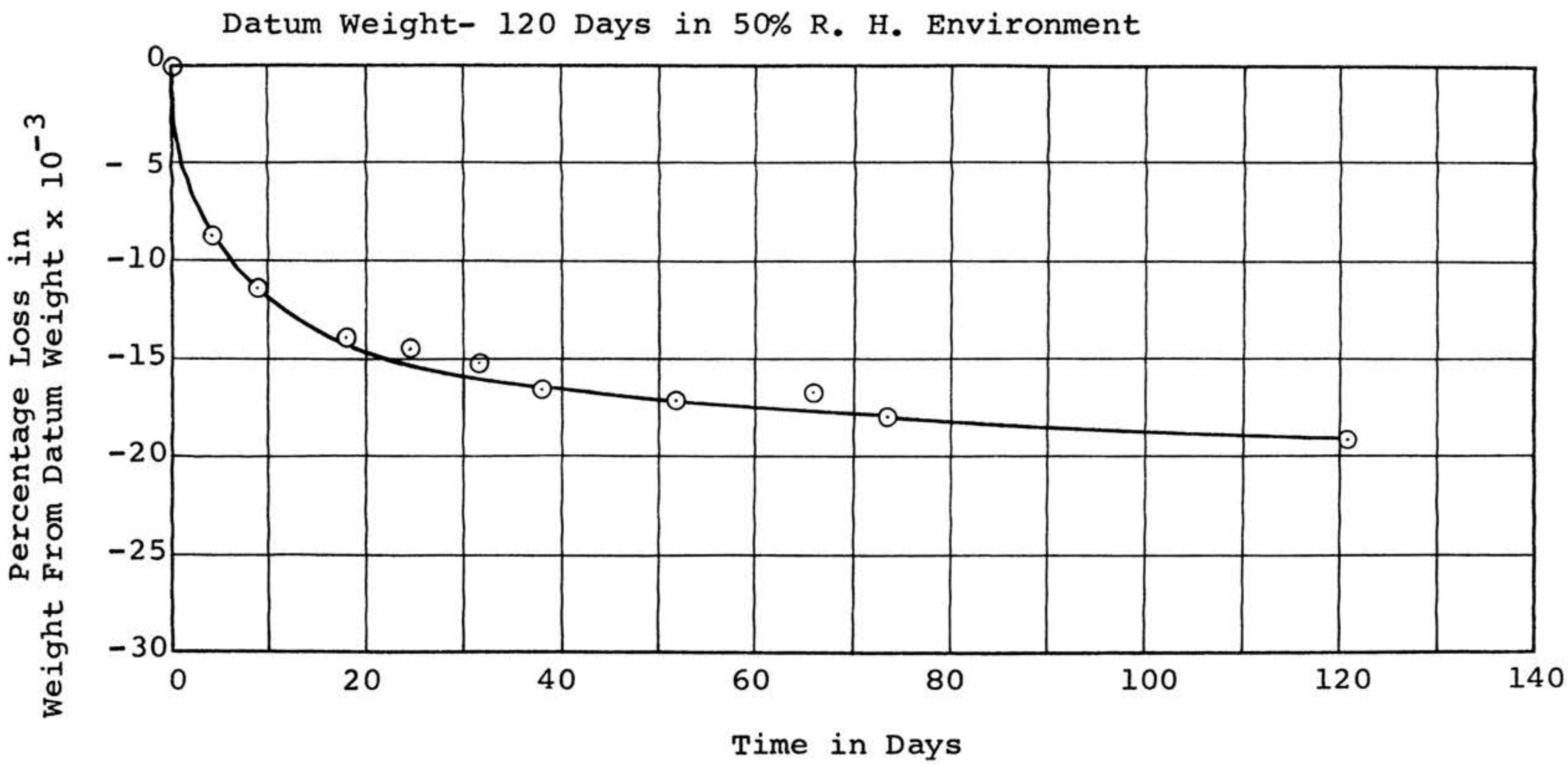


Figure 28 Typical Graph Showing Mean Percentage Loss in Weight of Quartzitic Shale with Time for Specimens Stored in Dry Environment (2).

porosity ranges between 1.6% and 3.7%.

2. Lamotte sandstone - a medium grained white quartzose sandstone from the upper Cambrian sediments. Its porosity ranges between 15.0% and 21.6%.

3. Rhyolite porphyry - a pink porphyry of Precambrian age. Its porosity ranges between 2.1% and 3.5%.

4. Magnetite ore - a high-silica (30% to 35%) iron ore with 35% to 45% iron formed by a high temperature replacement. Its porosity ranges from 2.0% to 7.1%.

5. Hematite - fine grained replacement body of iron associated with the magnetite deposit. Its porosity ranges between 2.2% and 7.1%.

6. Quartzite - a white fine grained well compacted quartzite of Precambrian age. Its porosity ranges between 1.7% and 4.1%.

White Pine Mine, Michigan

7. Shale - gray fine grained with well marked bedding planes. Its porosity ranges between 1.7% and 5.8%.

8. Sandstone - hard dense, fine to medium grained well-cemented sandstone. Its porosity ranges between 6.1% and 13.1%.

C. Compression Tests

The compression tests were performed on a 120,000 pound Tinius-Olsen hydraulic type machine with four loading ranges of 3,000 pounds, 12,000 pounds, 30,000 pounds, and 120,000 pounds.

A loading rate of 100 pounds per square inch per second was used following U.S. Bureau of Mines specifications.

This was accomplished with a "Pacer" accompanying the machine which was accurate to less than 4.7 pounds per square inch per second.

To prevent the upper platen from rotating and causing a moment to be produced in the specimens due to initial chipping of the corners, special jacks were made (Fig. 29) for leveling. After an initial load was placed on the specimens, the leveling jacks were placed in position.

The failure of each speciman was observed and classified according to the Canadian Department of Mines and Technical Surveys (4), as follows:

Degree	Shape	Fragment Size
1. very violent	a - top cone	A - dust
2. violent	b - bottom cone	B - 1/16" - 1/4"
3. semi-violent	c - double cone	C - 1/4" - 1/2"
4. quiet	d - longitudinal	D - larger than 1/2"
5. no data	e - diagonal	E - mixture
	f - irregular	F - no data
	g - no data	

For example, a speciman which failed very violently, leaving a double cone and a mixture of small and large fragments, would be coded as 1-c-E.

D. Brazilian Tensile Test

A compressive load applied perpendicularly to the axis of a cylinder and in a diametral plane (Fig. 30) gives rise to a uniform tensile stress over that plane. This indirect

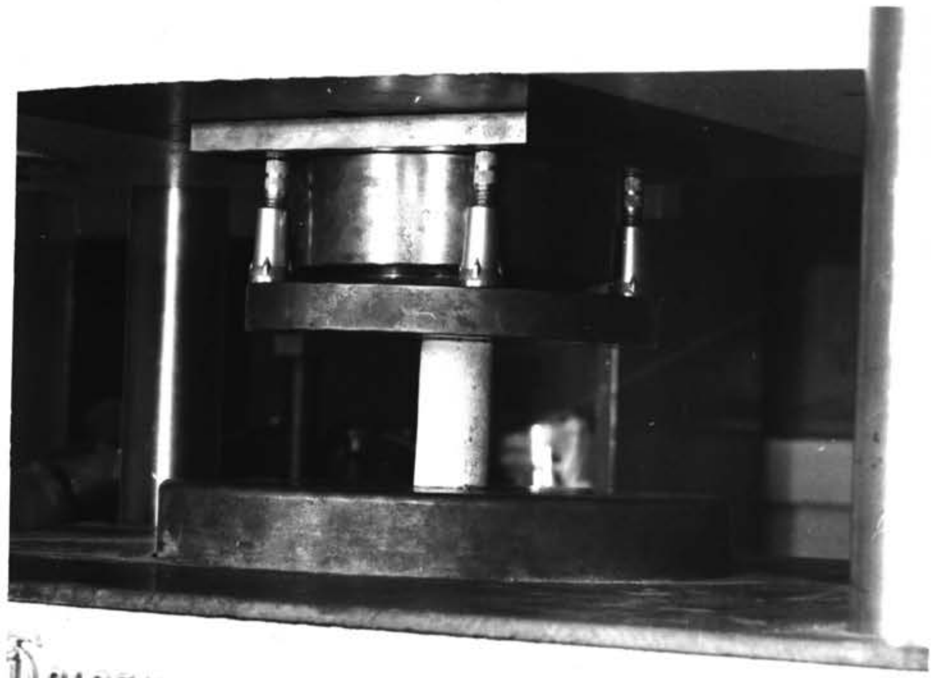


Figure 29 Special Levels Made to Prevent Rotation of Platen in Compression Tests.

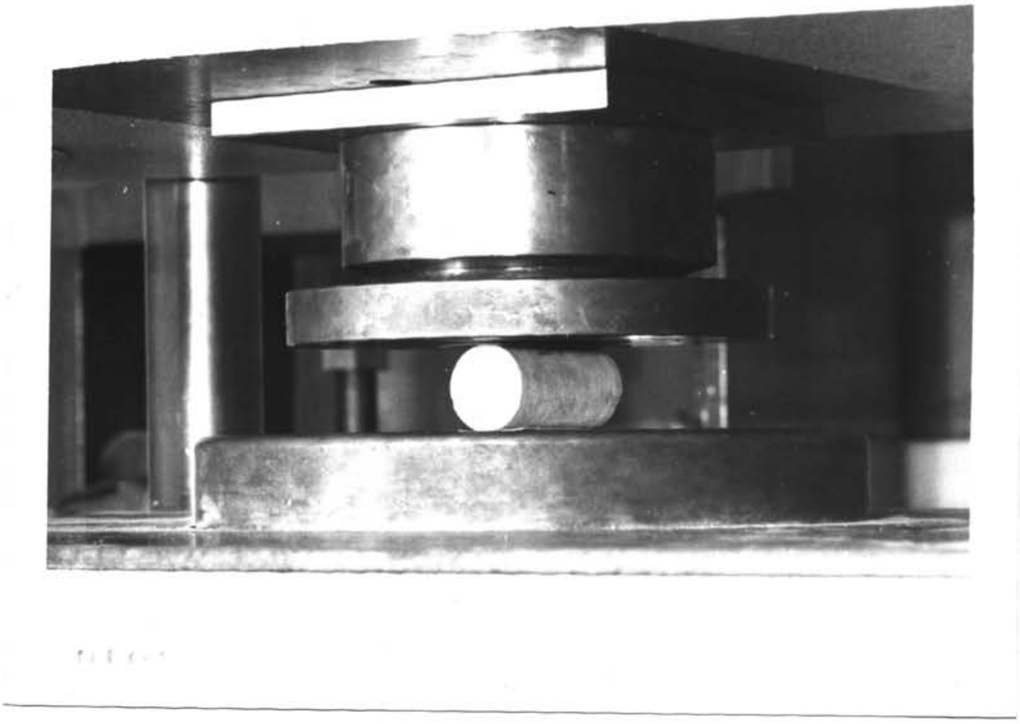


Figure 30 Loading a Speciman for a Brazilian Tensile Test.

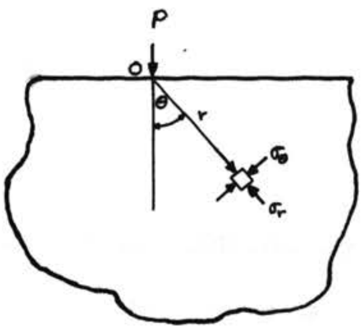


Figure 31 Stress in a plate due to a Concentrated Load P_1 Applied to an Edge.

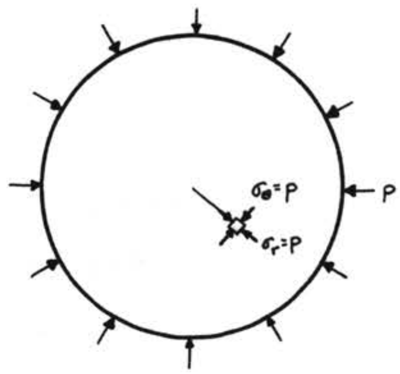


Figure 32 Stress in a Disc due to a Uniform Radial Pressure p.

tensile test for concrete, now being applied to rock, was introduced in Brazil by Fern and Carneiro in 1947. The mathematical analysis of this stress distribution (7, 8, 9) has been derived from two fundamental conditions.

The first condition to be investigated is that of a concentrated vertical load P acting on a horizontal straight plate of infinite length and of a thickness t (Fig. 31). Assuming a condition of plane stress and that the material obeys Hooke's Law, the stress components on any element at an angle θ from the vertical and at a distance r from the point of application of the load are:

radial stress, towards the point of application of the load	$\sigma_r = \frac{2P}{\pi t} \frac{\cos \theta}{r}$
circumferential stress, perpendicular to the radial stress	$\sigma_\theta = 0$
shear stress	$\tau_{r\theta} = 0$

The second stress distribution to be investigated is that of a disc loaded by a uniform radial pressure p (Fig. 32). The stress in any direction and at any point is equal to the applied pressure and there is no shear stress.

$$\sigma_r = p$$

$$\sigma_\theta = p$$

$$\tau_{r\theta} = 0$$

At any point on the circumference of the disk (Fig. 33)

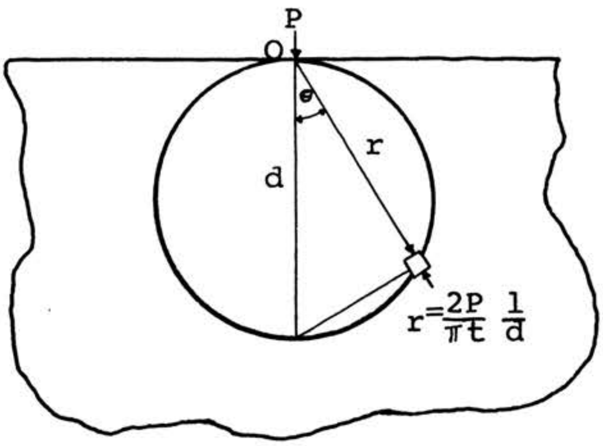


Figure 33 Stress at the Circumference of a Circular Area of the Plate Shown in Fig. 31.

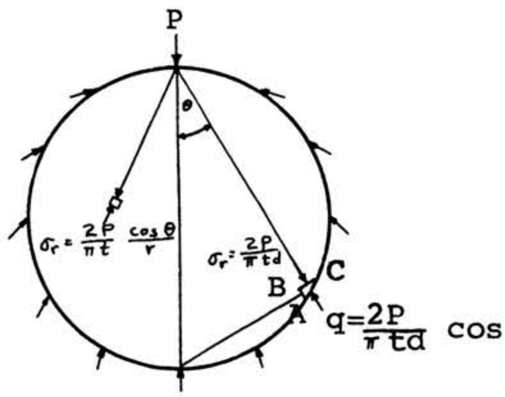


Figure 34 A Disc Subjected to the Same Loading as a Circular Area of the Plate Shown in Fig. 33.

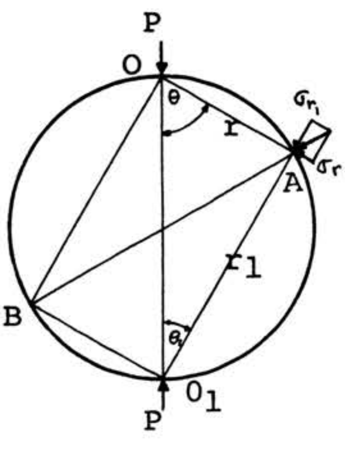


Figure 35 Two Sets of Forces Superimposed.

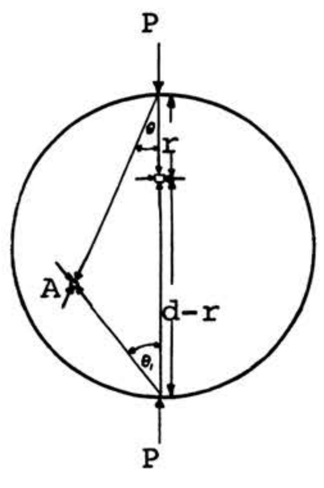


Figure 36 Disc Subjected to Two Concentrated Forces.

being considered part of the plate (Fig. 31), there is a stress $\frac{2P}{\pi t} \frac{\cos \theta}{r}$ acting toward O, and, from geometry, it can be seen that: $\frac{r}{d} = \cos \theta$

where: d = diameter of the disc.

Let the circular area be removed from the plate (Fig. 34) and such stress, q, be applied to the circumference as will maintain the same effect as that of the plate being whole. Under the conditions of equilibrium of element ABC,

$$\frac{2P}{\pi td} BC = q\ AC$$

$$q = \frac{2P}{\pi td} \frac{BC}{AC} = \frac{2P}{\pi td} \cos \theta$$

A similar system may be superimposed if the force P acted upon the bottom of the disc, and therefore, upon the system of stress already described. Now the disc is subjected to two opposite forces P acting along a diameter, and two sets of stresses, acting on the circumference (Fig. 35), of magnitude $\frac{2P}{\pi td} \frac{r}{d}$ along AO and $\frac{2P}{\pi td} \frac{r_1}{d}$ along AO_1.

The resultant of these two external stresses may be represented by AB = $\frac{2P}{\pi td}$, which is a constant and passes through the center of the disc. A uniform radial compression stress of magnitude $\frac{2P}{\pi td}$ is produced by the two systems together.

In order that the boundary of the disc is free from any external forces, a uniform radial tensile stress of equal magnitude will be superimposed. The disc is subjected to two opposite forces acting along a diameter (Fig. 36).

Any element A is therefore subjected to the two compressive stress components $\frac{2P}{\pi t} \frac{\cos \theta}{r}$ and $\frac{2P}{\pi t} \frac{\cos \theta_1}{r_1}$, as indicated, and a tensile stress of $\frac{2P}{\pi td}$ in all directions.

The exact stress, on the vertical diameter where $\theta = \theta_1 = 0$, can be readily calculated. The vertical stress component (compressive)

$$\sigma_V = \frac{2P}{\pi t} \frac{1}{r} + \frac{2P}{\pi t} \left(\frac{1}{d-r} \right) - \frac{2P}{\pi td}$$

$$\sigma_V = \frac{2P}{\pi td} \left(\frac{d}{r} + \frac{d}{d-r} - 1 \right)$$

and the horizontal stress component (tensile)

$$\sigma_H = -\frac{2P}{\pi td}$$

IV. ANALYSIS OF RESULTS

The results of the compression and tensile tests have been compiled in Tables 3 through 20 in Appendix I and II. These results have been plotted in Figs. 46 through 52 and 54 through 64.

A. Data Reduction

All the available samples were tested in this program, with no samples being eliminated, initially, due to structural irregularities as could visibly be seen. Any abnormal characteristic was recorded and considered when the data were analyzed.

Conventional methods of data reduction by employing calculation of standard deviation (3) were attempted without success. All the materials demonstrated a high degree of deviation and data which obviously should have been eliminated, remained intact because of this wide range of values. This required that a more direct process of elimination be used, without any mathematical basis. This method was as follows; omitting samples which showed structural irregularities first and then, by observation, omitting data which did not appear to "fit" with the general trend.

The data collected on the Lamotte sandstone was not shown graphically because of the wide spread of values. The samples were of different origin and environment and were not of one composition, structure, or grain size. It was known prior to testing that probably no conclusions could be drawn for this material. The results are entered for general interest and to show this material's general characteristics.

B. General Testing Procedure

When the testing program was initiated, a central moisture content (45% relative humidity) was picked for two reasons. Under natural conditions in most areas, a rock would most generally be under this condition of humidity. A 45% relative humidity may be considered a "norm" and demonstrates the strength of a rock in its most natural condition. By showing the effects of strength due to moisture condition on either side of this "norm", the change in strength with a change in the climatic condition is shown. Also, by using this condition as a "zero" moisture content, to which moisture was either added or removed before testing, the time required to alter the moisture content would be reduced.

It became apparent that the curve that might best fit the plotted results would be a log-log relationship. In order for this relationship to be shown - the log of a negative number being undefined - the zero moisture content for the samples from the White Pine Copper Company was picked at the oven-dried state.

During the testing of the samples which were intended to be in a state of saturation, it became apparent that the samples were not being completely saturated. This was discovered in comparing the moisture content at saturation to what the moisture content should have been as determined from the porosity test. An example of this would be sample SS-11 whose porosity was 10.33% and moisture content at saturation was 2.36% and specific gravity was 2.51. The degree of saturation was calculated to be only 57.1% and not 100% as expected.

C. Results

The test results from both the Pea Ridge Mine and the White Pine Mine were plotted showing the relationship of compressive and tensile strength per unit area versus the moisture content. As mentioned earlier, a log-log plot of the White Pine samples was drawn to compare the compressive and tensile strength of the rock material to the moisture content.

The conditions under which the specimens failed were not uniform. Figs. 37 through 45 show a few of the samples after failure. The system (tension, compression, or shear) under which each sample failed is not always clear. The planes of failure, as seen from these figures, are complex. Many of the sedimentary samples appeared to fail in a diagonal plane which would alter to a longitudinal plane near the center of the specimen. Many specimens failed in two or more planes. In many of the violent and very violent failures, little was left of the specimens for reconstruction.

There was a general trend for a decrease in compressive strength with an increase in moisture content. Table 2 shows the degree to which the moisture affected the strengths of

Figure 37 Compressive Failure of a White Pine Sample of Sandstone Under a Condition of 4cE-65°.

Figure 38 Compressive Failure of a White Pine Sandstone Sample Under a Condition of 4cE- 70°.

Figure 39 Compressive Failure of a White Pine Sandstone Sample Under a Condition of 3eE- 65°

Figure 40 Compressive Failure of a White Pine Shale Under a Condition of 4eE-70°.

Figure 41 Compressive Failure of a White Pine Shale Under a Condition of 3eE- 70°.

Figure 42 Compressive Failure of a White Pine Shale Sample Under a Condition of 2e,dE-70°.

Figure 43 Compressive Failure of a White Pine Sandstone Sample Under a Condition of 4eE-70°.

Figure 44 Compressive Failure of a Pea Ridge Dolomite Sample Under a Condition of 3dE.

Figure 45 Compressive Failure of a Pea Ridge AX Magnetite Sample Under a Condition of 2eE- 65°.

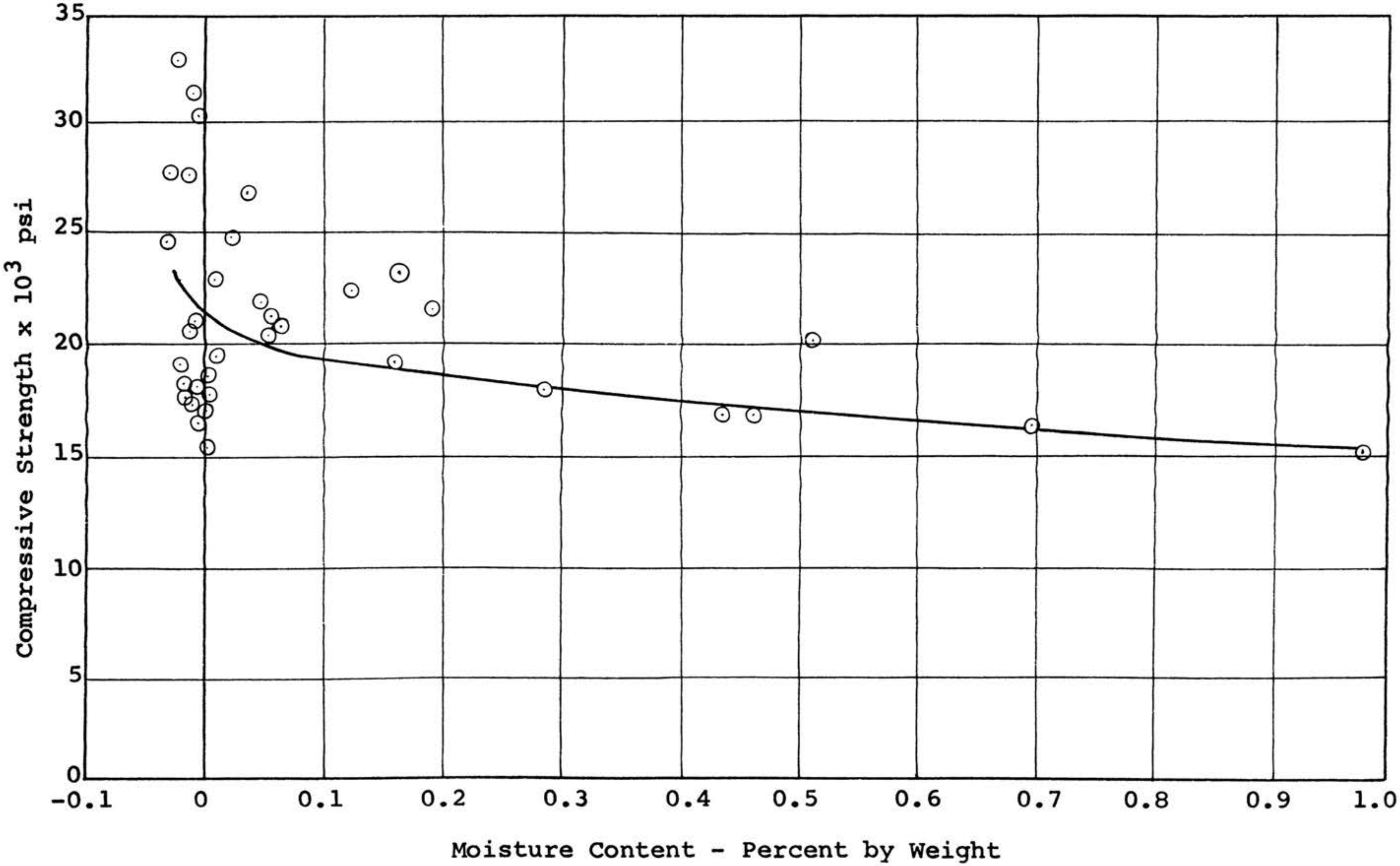


Figure 46 Moisture Content vs Compressive Strength for Magnetite.

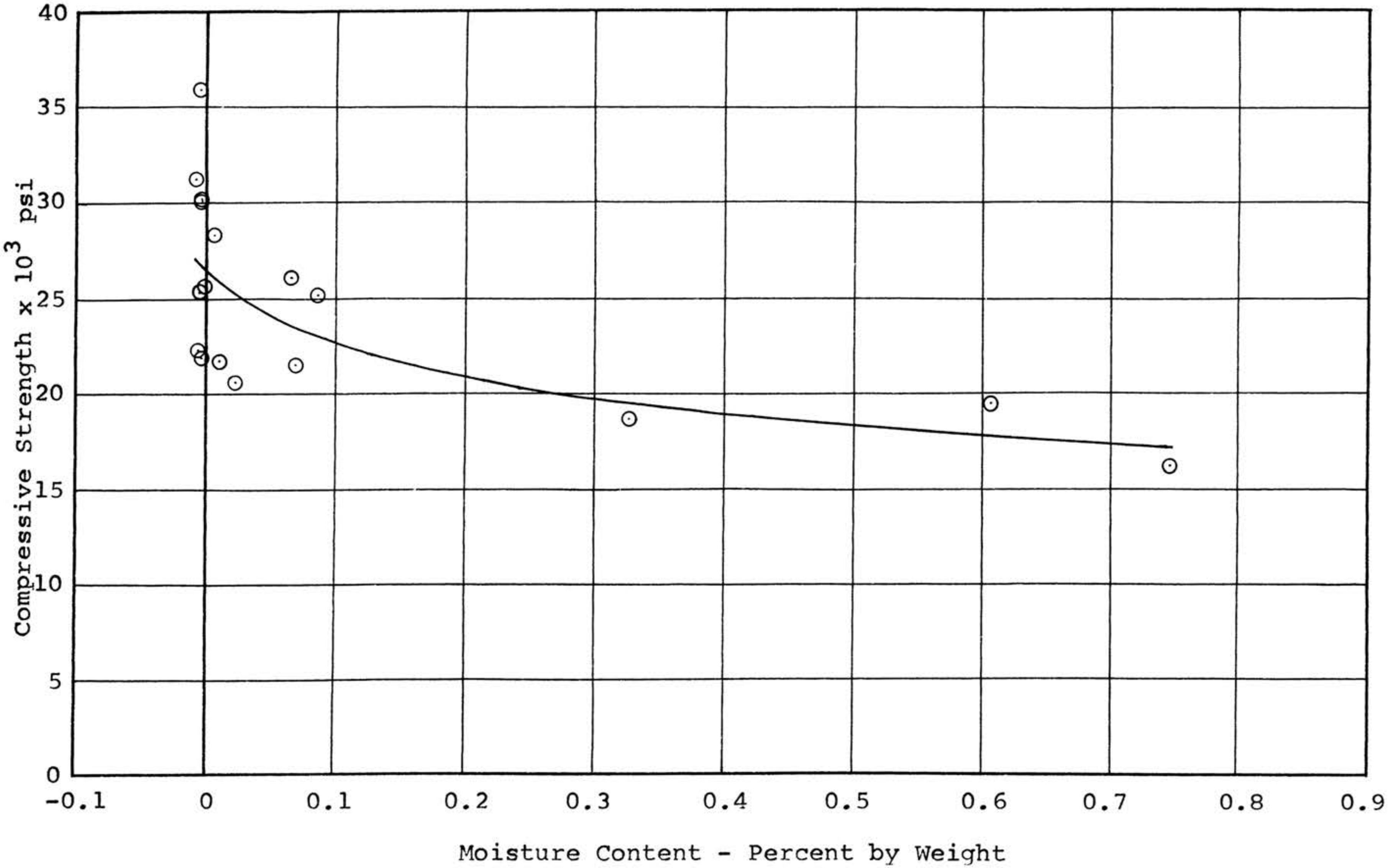


Figure 47 Moisture Content vs Compressive Strength for Hematite.

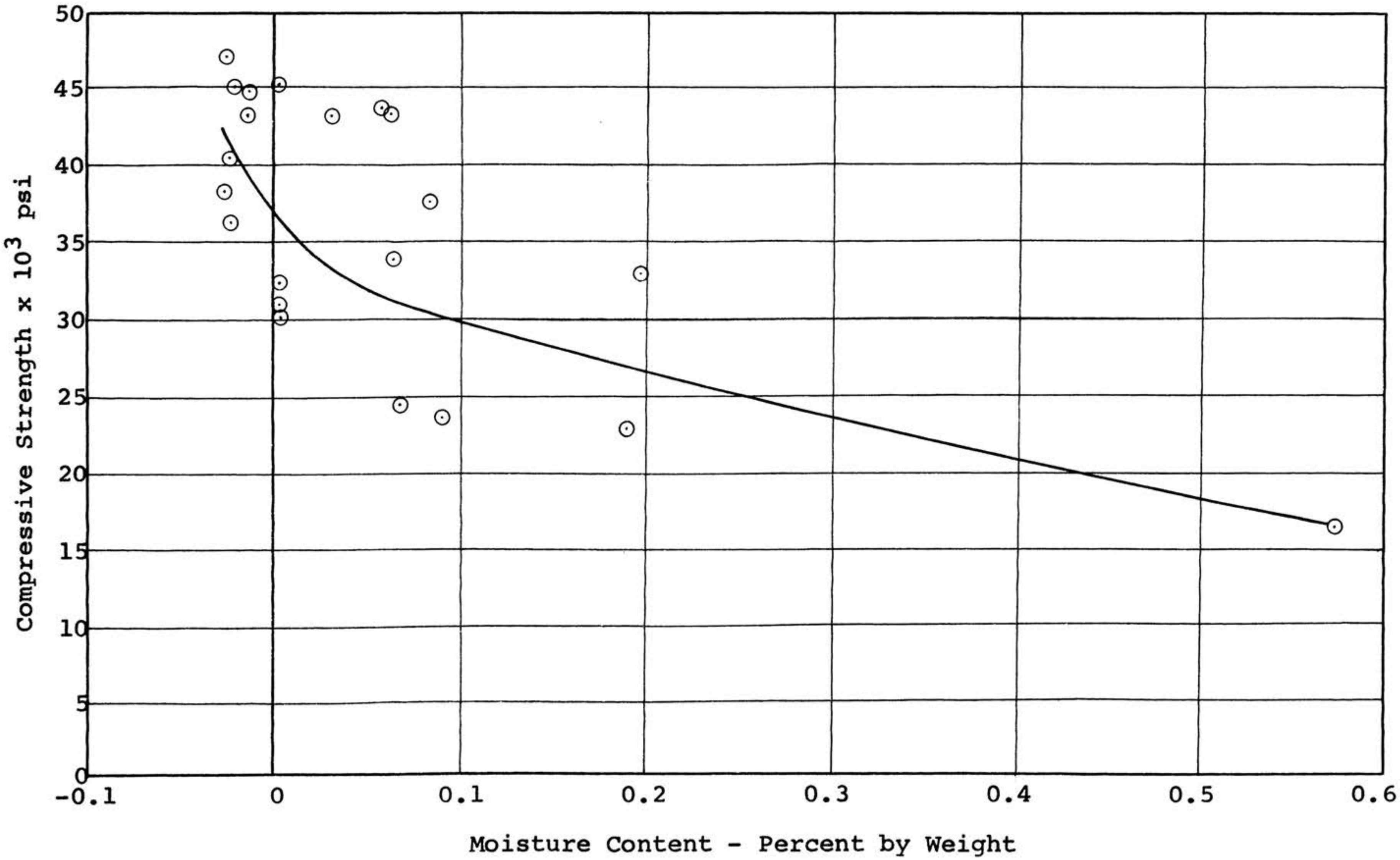


Figure 48 Moisture Content vs Compressive Strength for Porphyry.

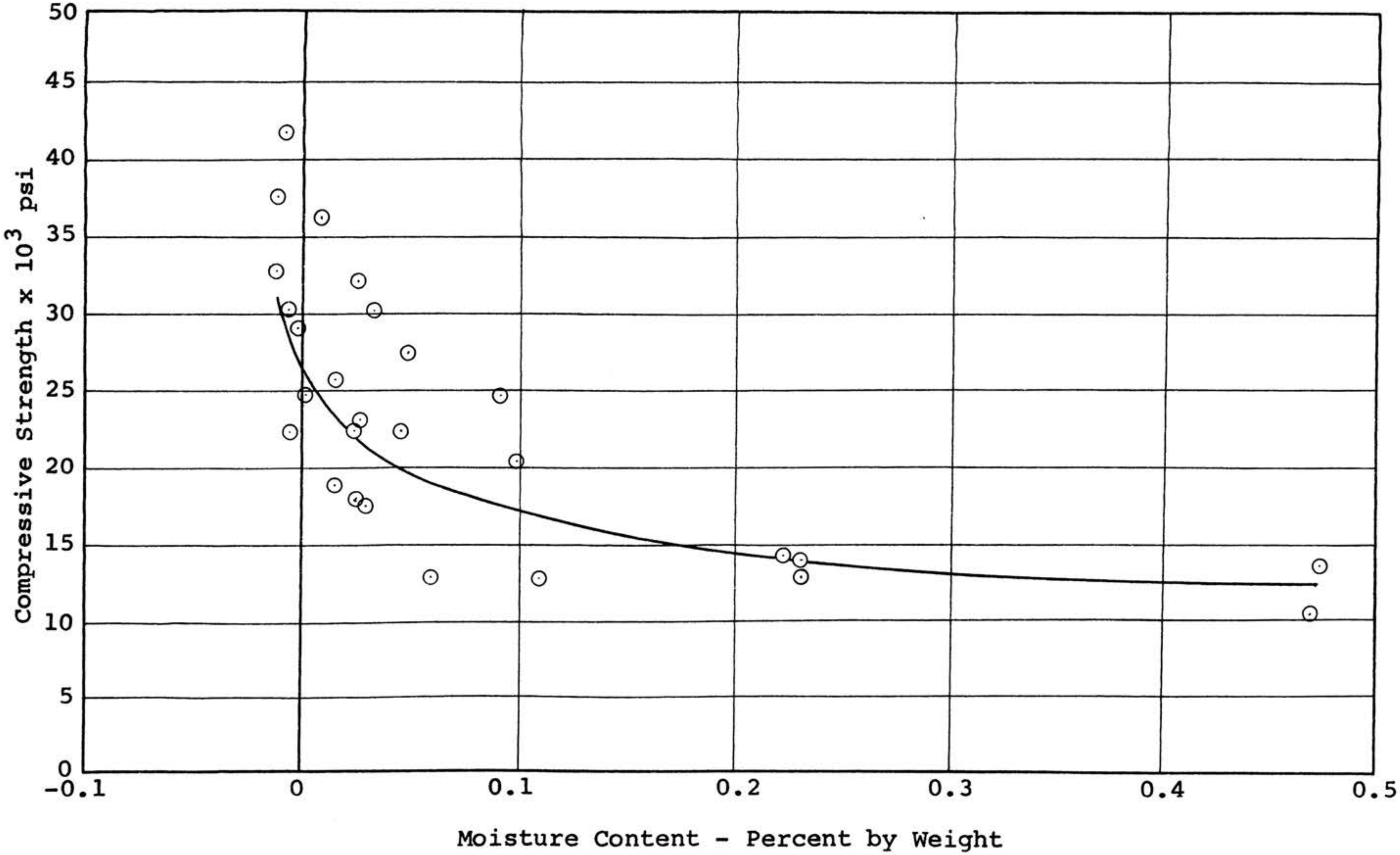


Figure 49 Moisture Content vs Compressive Strength for Quartzite.

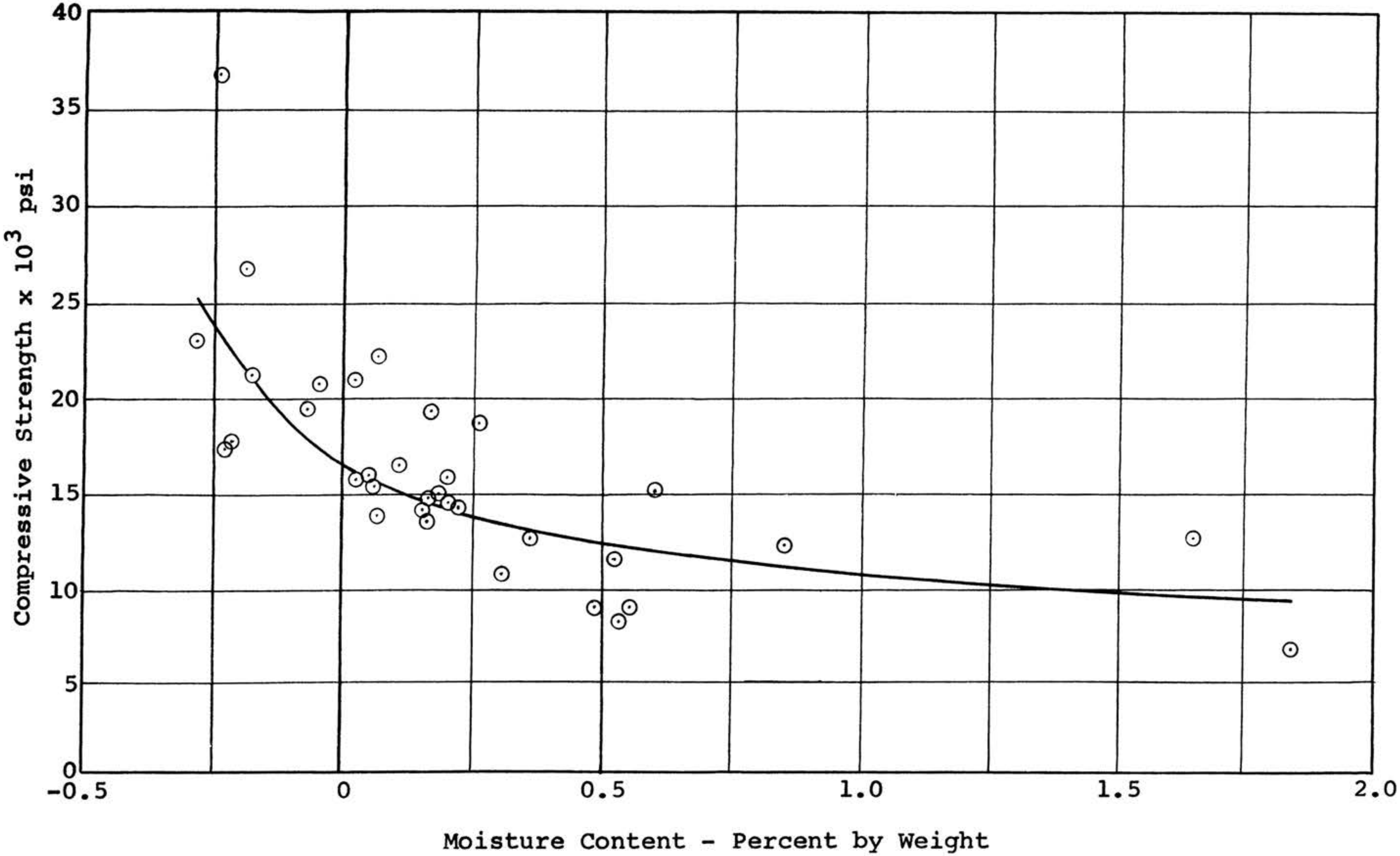


Figure 50 Moisture Content vs Compressive Strength for Dolomite

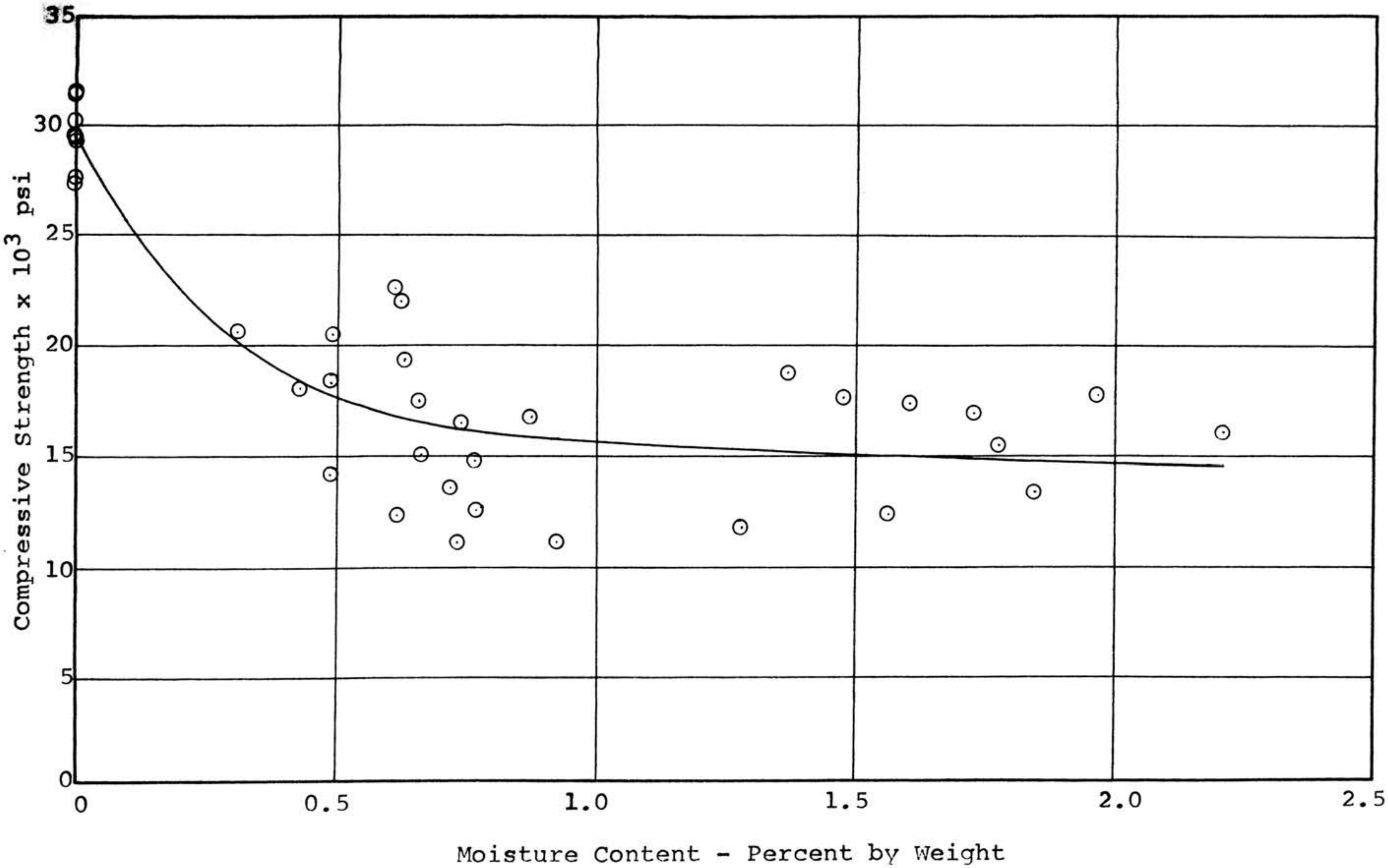


Figure 51 Moisture Content vs Compressive Strength for Shale.

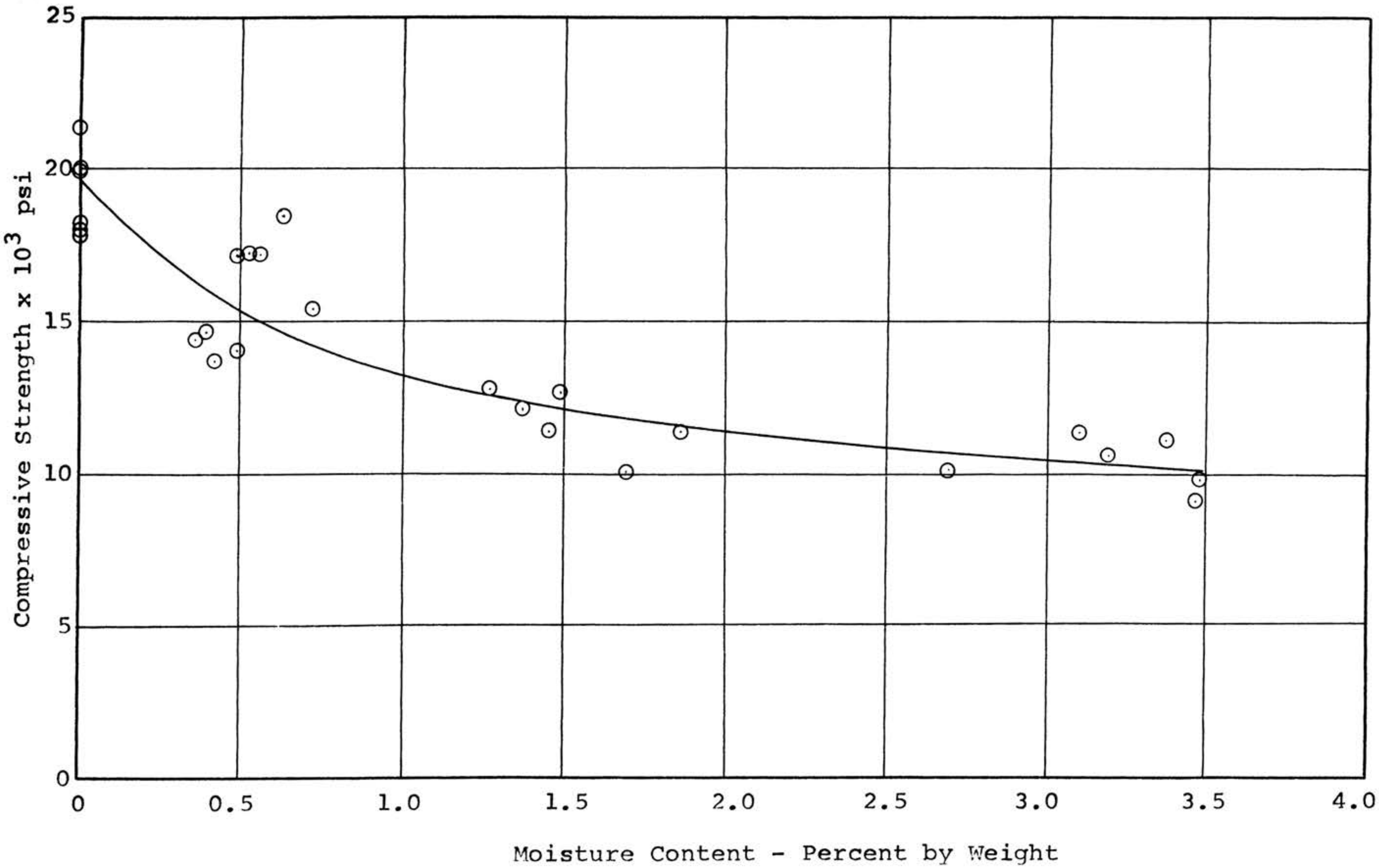


Figure 52 Moisture Content vs Compressive Strength for White Pine Sandstone.

the various samples. The maximum and minimum strength values were taken from the curves drawn to represent the general trend of the data. The tabulated values represent the ratio of the compressive and tensile strengths of a speciman in the saturated state to compressive and tensile strengths of a speciman in the dry state.

In compression, the highest percentage of change in the dry and saturated strengths is that of the quartzite and the dolomite. The dry specimens of quartzite failed generally along a longitudinal plane while the saturated specimens failed irregularly, indicating failure along planes of weakness. Dolomite, on the other hand, failed generally by splitting along a longitudinal axis in both the dry and saturated state.

The smallest percentage of change in the compressive strength of the rock material tested was in the shale and magnetite. The shale specimens failed mainly along a diagonal plane at an angle of 70°. The general plane was the same for both the dry and saturated samples, but the dry sample failed very violently while the saturated samples failed semi-violently. This would seem to indicate that the added moisture acted as a lubricant on the plane of failure. The double cone was the predominant failure pattern in the magnetite specimens. The angle of these planes of failure generally varied $\pm$ 5° from a 70° plane.

There was also a general trend for a decrease in tensile strength with an increase in moisture content, with

Table 2. Approximate Ratio of Strengths at Maximum to Strength at Minimum Moisture Content

	Compression	Tension
Magnetite	64%	67%
Hematite	64%	68%
Porphyry	38%	130%
Quartzite	35%	141%
Dolomite	36%	42%
Shale	50%	59%
Sandstone	51%	51%

the exception of the quartzite and the porphyry specimens. The tensile strength of quartzite and porphyry showed a trend of increasing with an increasing moisture content.

The centers of the specimens tested in the indirect tensile test are under a biaxial loading condition. Along the vertical axis, there is a compressive stress of magnitude $\frac{6P}{\pi td}$ applied parallel to the axis of loading, and a tensile stress of magnitude $\frac{-2P}{\pi td}$ applied perpendicular to the axis of loading. Pore water pressures would exist under this compressive stress and would tend to increase the stress applied on the plane of failure.

This condition could exist only in a heterogeneous material, while the equations were derived for a homogeneous mass. The exact distribution of local stresses cannot be determined, but a redistribution of the pore pressures could also conceivably oppose the tensile stresses produced, giving the appearance of a higher tensile strength with an increased moisture content. Highly stressed areas of the specimen will tend to throw the stress onto those areas where the stress is lower.

There was little moisture change from the zero moisture content in both the porphyry and the quartzite specimens as compared to the rest of the materials tested. The specific gravity was lower and the degree of violence at failure of both types of material was higher than with the other materials tested. The material is of a higher degree of compaction and consolidation.

A plane of failure is forced along the longitudinal axis of loading in the Brazilian tensile test (Figs. 53a and 53b). The failure is caused by a tensile force, but the sample cannot fail along a plane of weakness or a more preferred orientation. The quartzite samples showed planes of weakness in the saturated compression test, but these planes were not utilized in the tension test.

It might also be noted, in the tests run by the Bureau of Mines on Young's modulus (3), that the highly consolidated materials (igneous and metamorphic) showed an increase in modulus with an increase in moisture content. This would seem to indicate that the moisture was preventing the strain of the material. The water, being relatively incompressible, is redistributing the load to areas which are able to withstand the increase. The water acts to transmit the increasing load.

The highest percentage of decrease in the tensile strength from the dry to the saturated state was in the dolomite specimens. It might be noted that the degree of change in the dolomite and the other samples tested, excluding the porphyry and quartzite, showed approximately the same change in strength in tension and compression (Table 2).

D. Log-Log Graphs

The second group of tests on ths shale and sandstone specimens from the White Pine Copper Mine was set up with the zero moisture content base being the oven-dried samples.

a. White Pine Shale.

b. White Pine Sandstone.

Figure 53 Resulting Failure Plane in the Brazilian Tensile Tests.

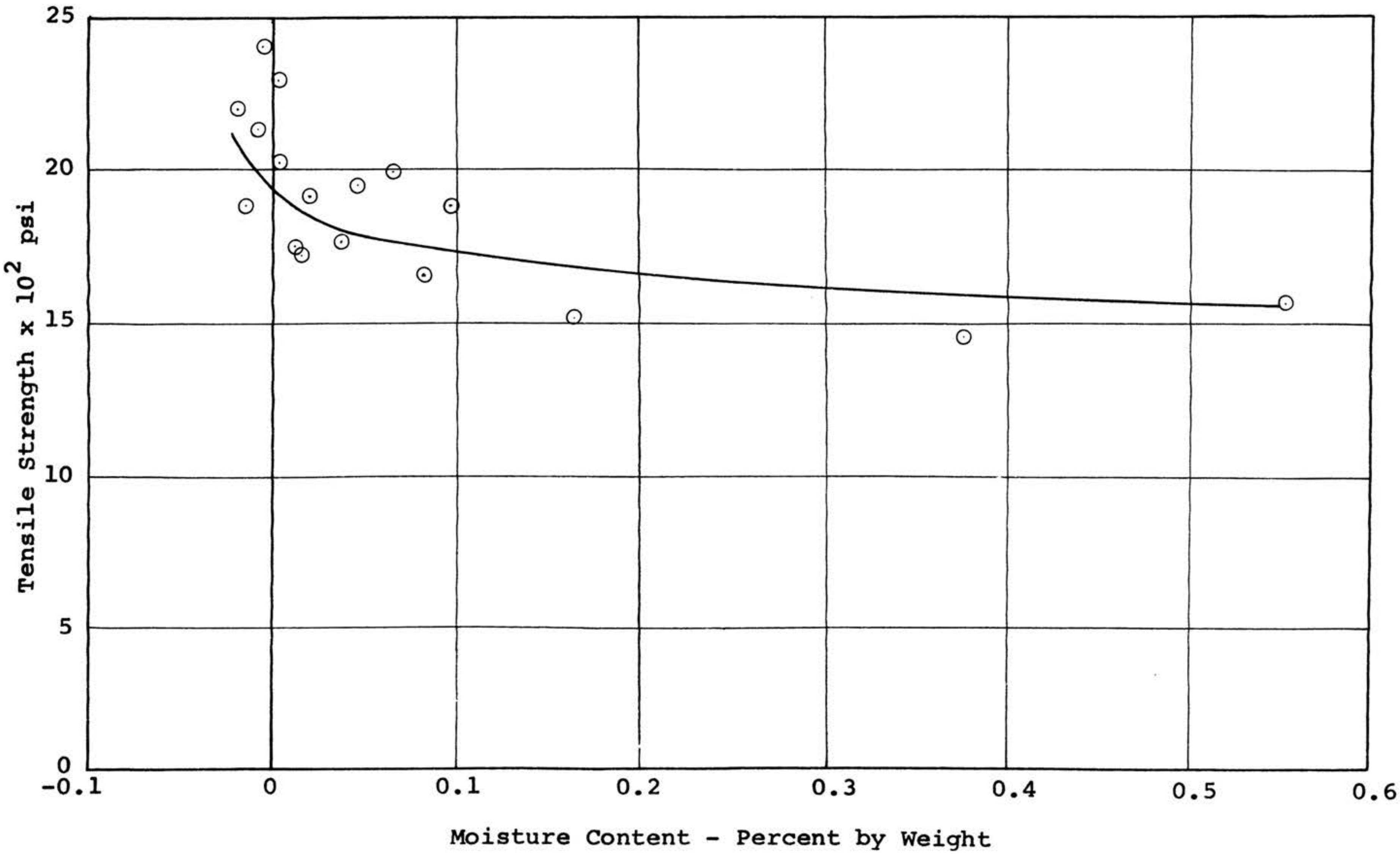


Figure 54 Moisture Content vs Tensile Strength for Magnetite.

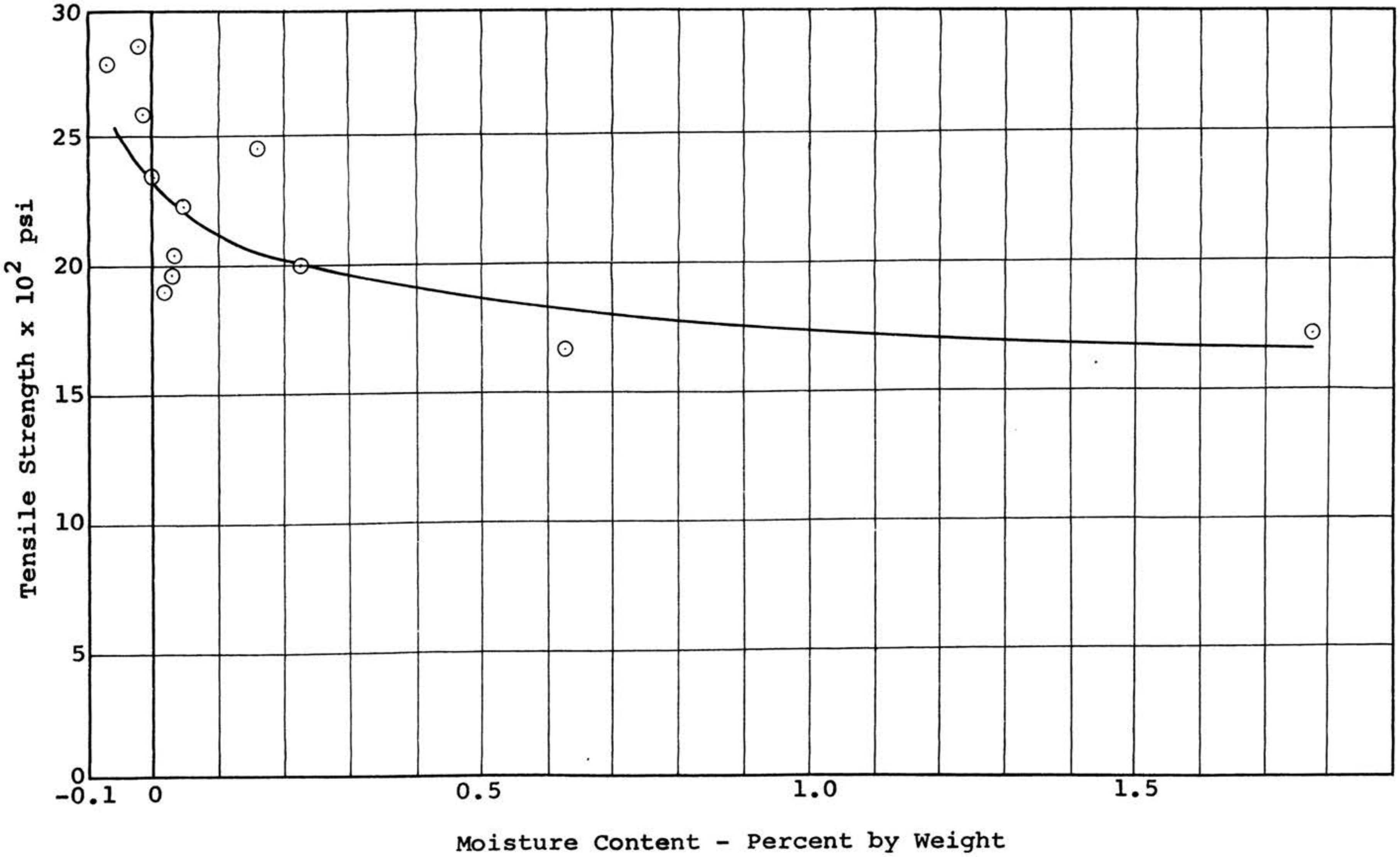


Figure 55 Moisture Content vs Tensile Strength for Hematite.

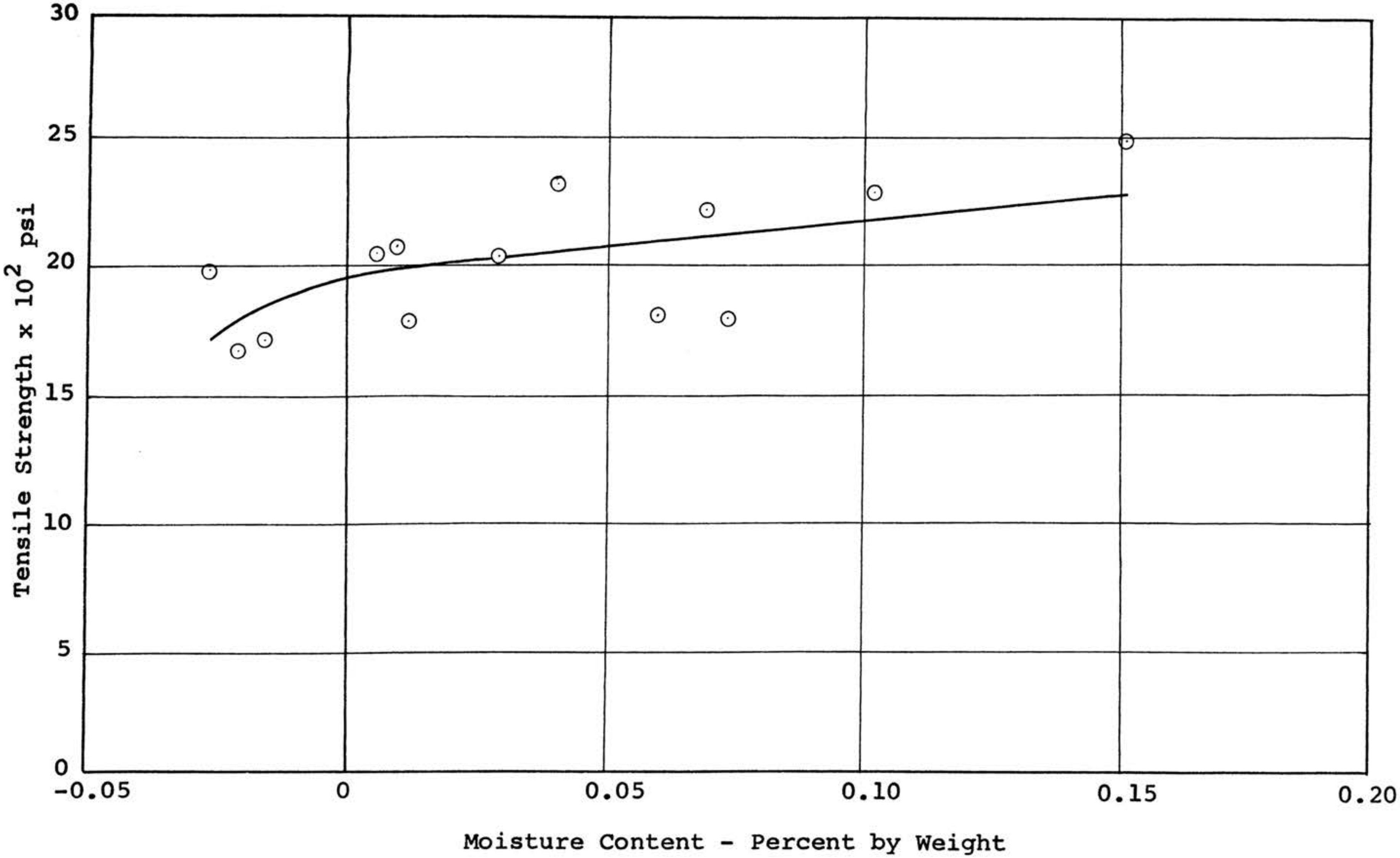


Figure 56 Moisture Content vs Tensile Strength for Porphyry.

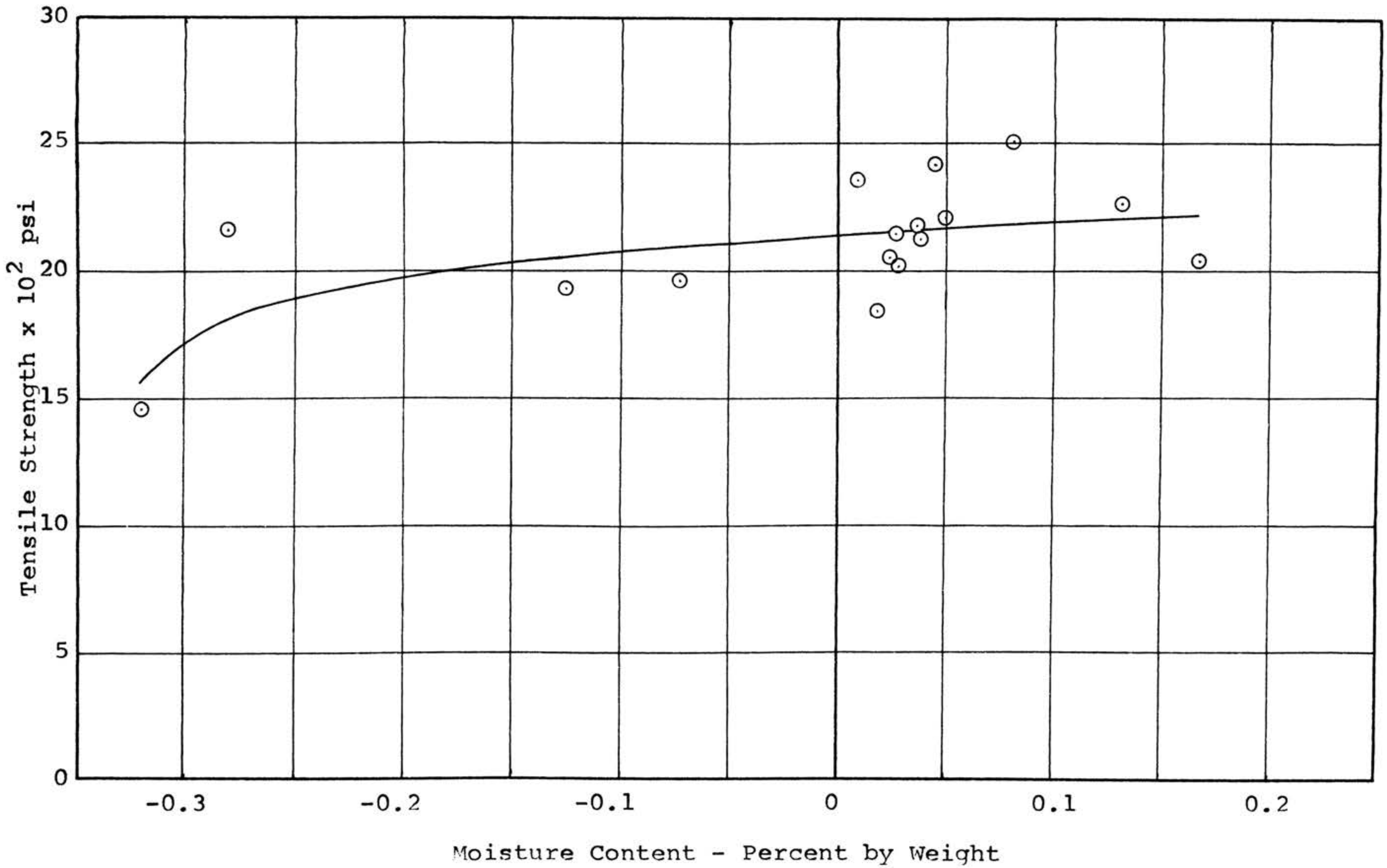


Figure 57 Moisture Content vs Tensile Strength for Quartzite.

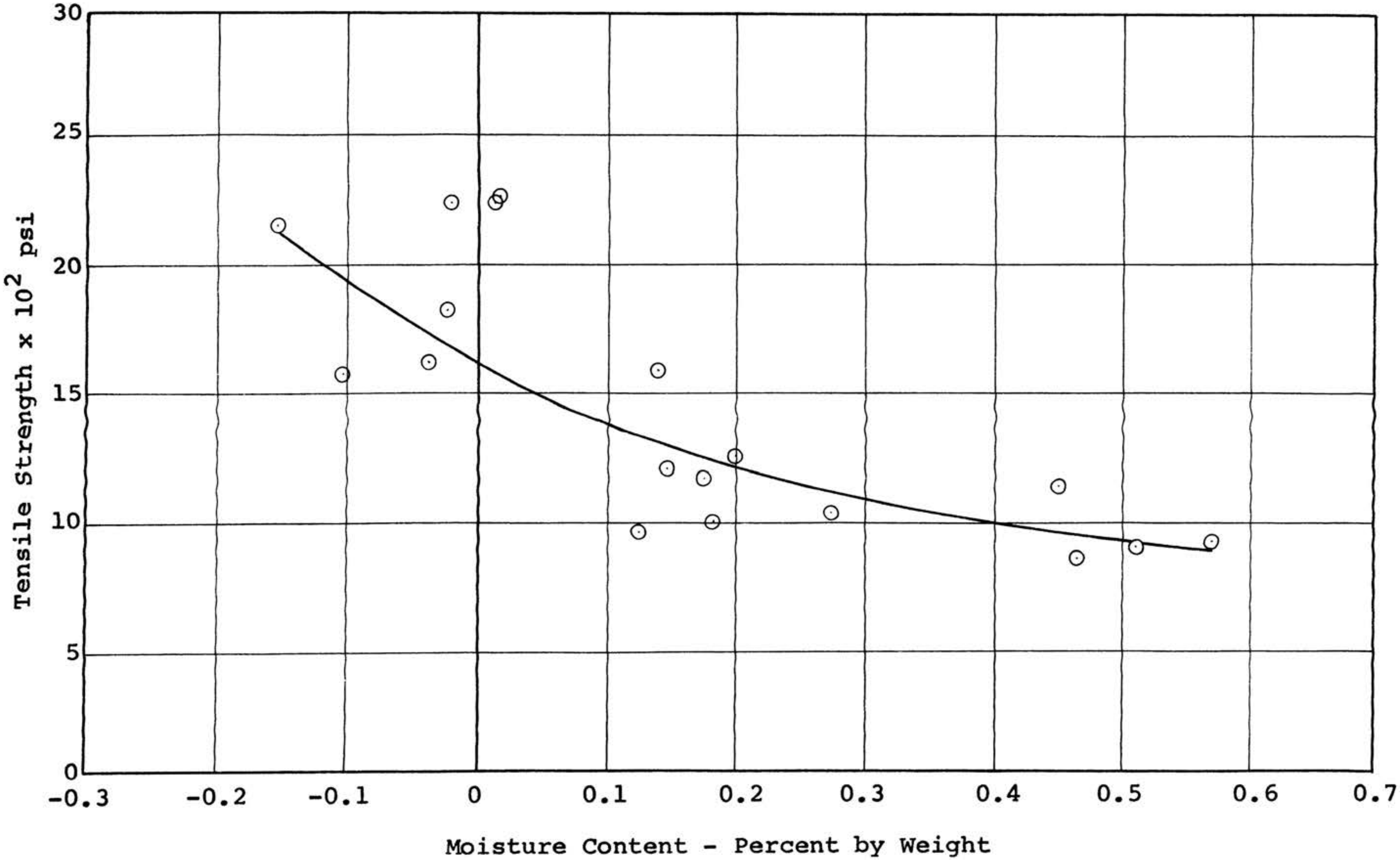


Figure 58 Moisture Content vs Tensile Strength for Dolomite.

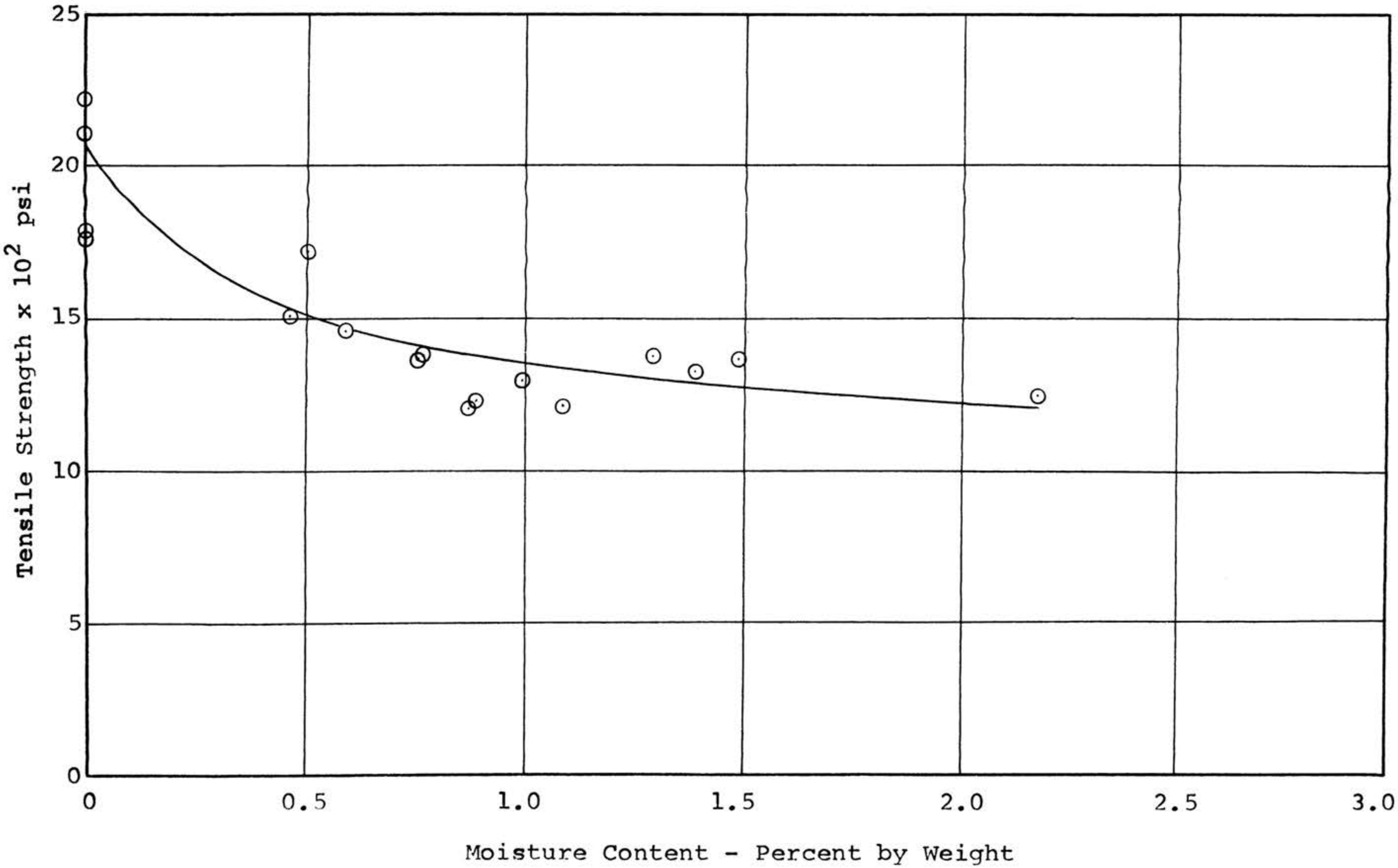


Figure 59 Moisture Content vs Tensile Strength for Shale.

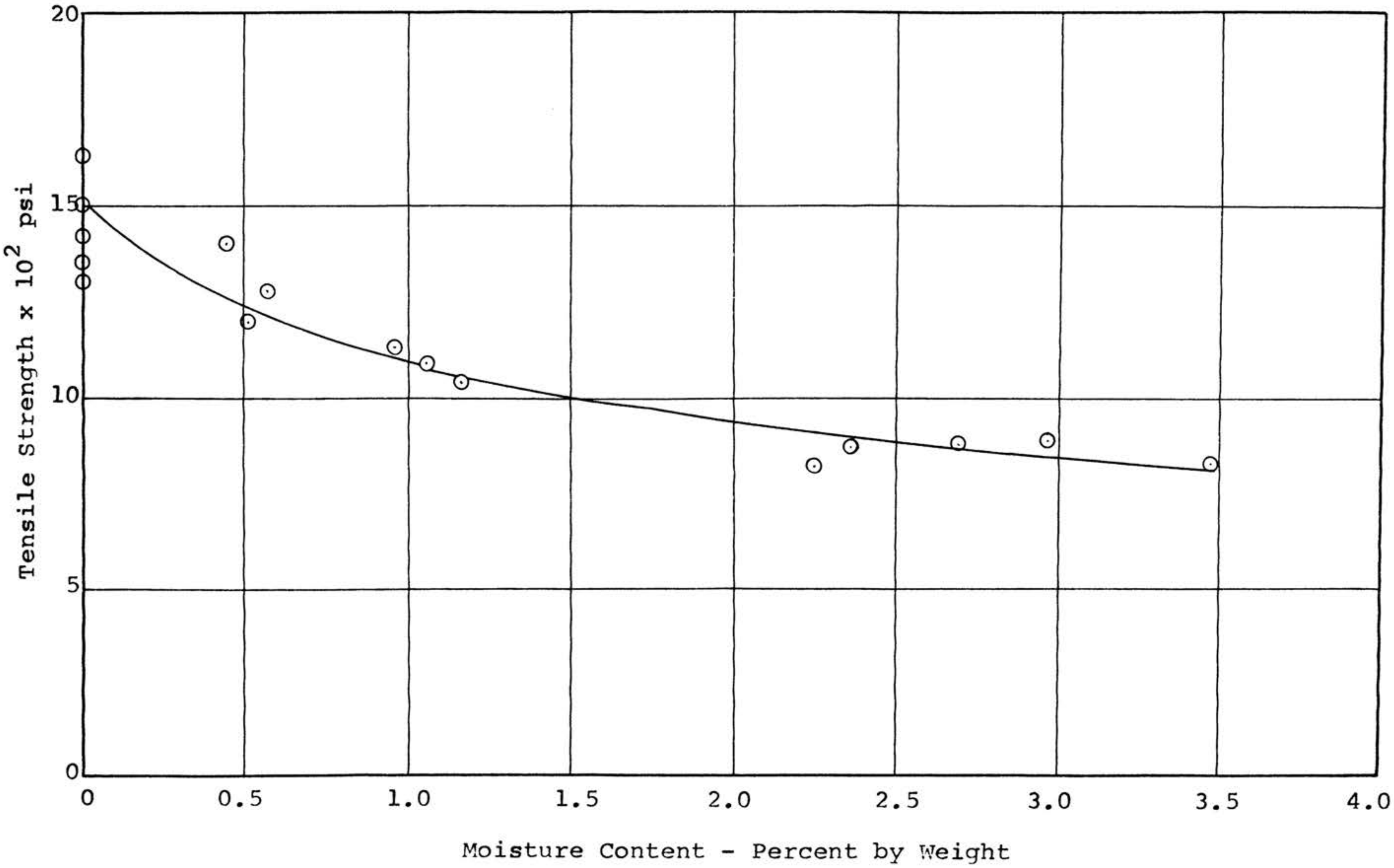


Figure 60 Moisture Content vs Tensile Strength for White Pine Sandstone.

The results were plotted on a log-log graph (Figs. 61 through 64) comparing the compressive and tensile strength per unit area to the moisture content. The best straight line was drawn between the results by using the method of least squares.

The log-log comparison of the results appears to generally form a straight line relationship. There was quite a bit of scatter in the results of the shale tests (Figs. 61, 63), with a general plane of weakness of 70° to the horizontal. The compression and tension tests of the sandstone specimens plotted on a log-log graph (Figs. 62, 64) shows a good straight-line fit.

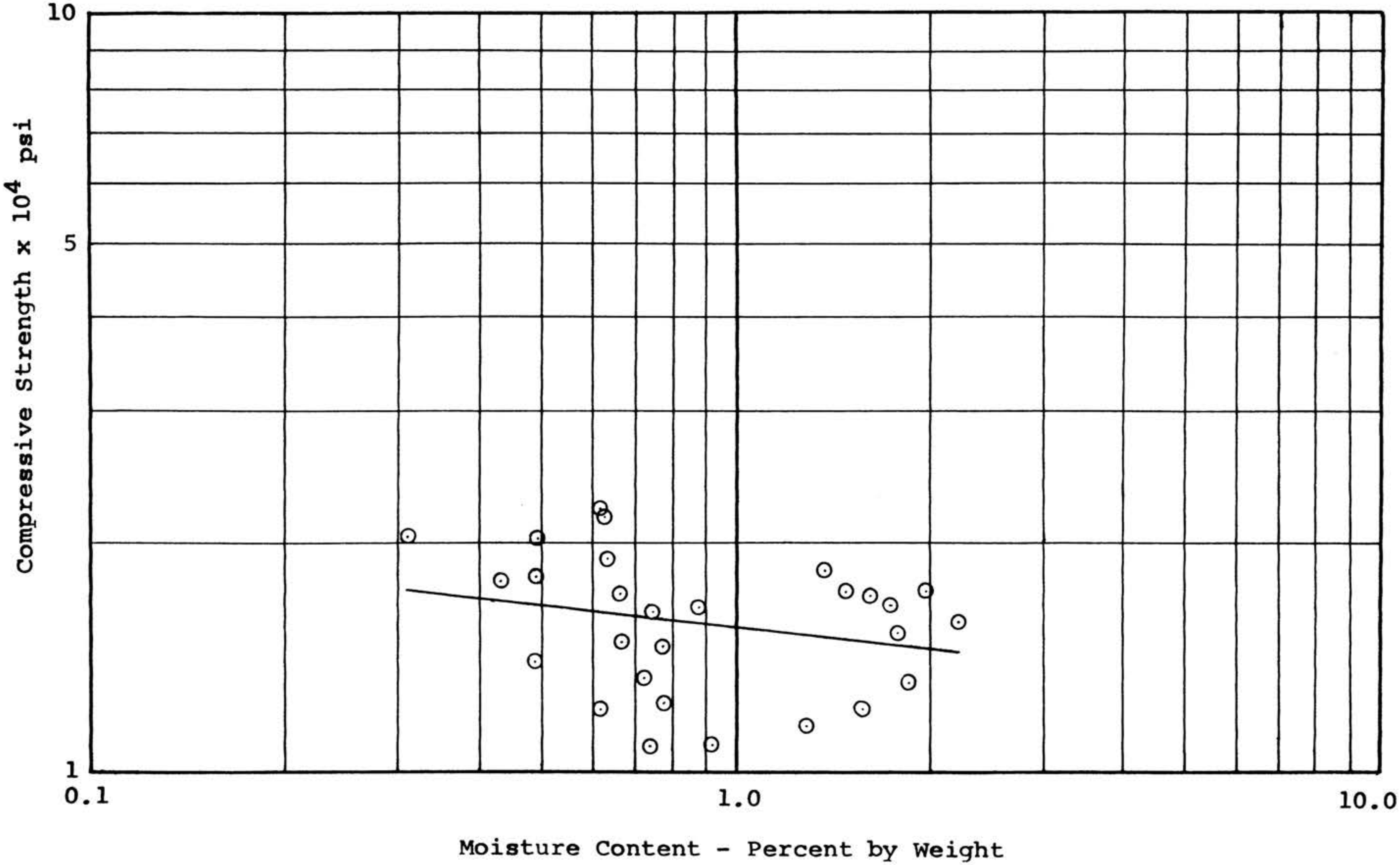


Figure 61 Moisture Content vs Compressive Strength for Shale.

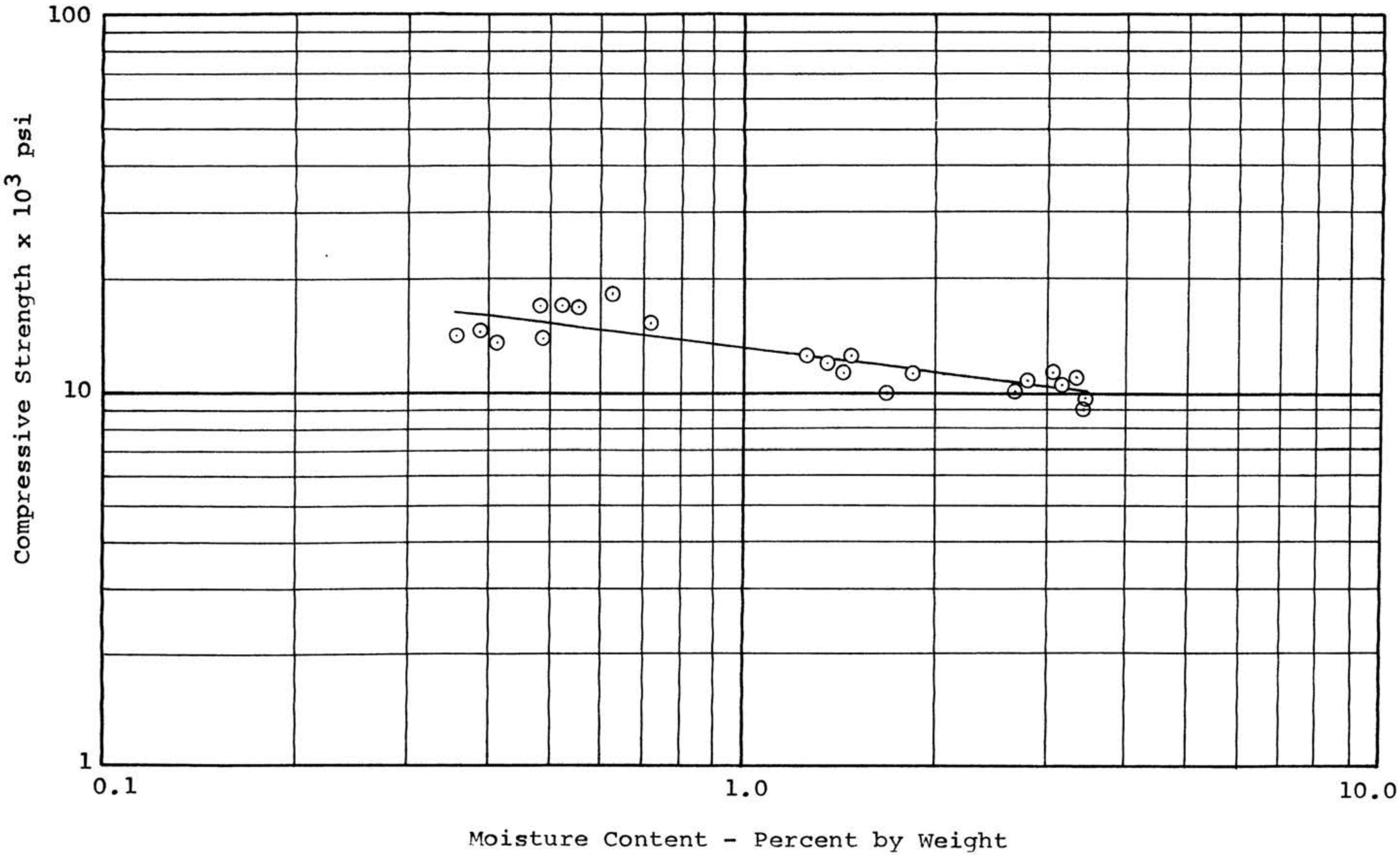


Figure 62 Moisture Content vs Compressive Strength for White Pine Sandstone.

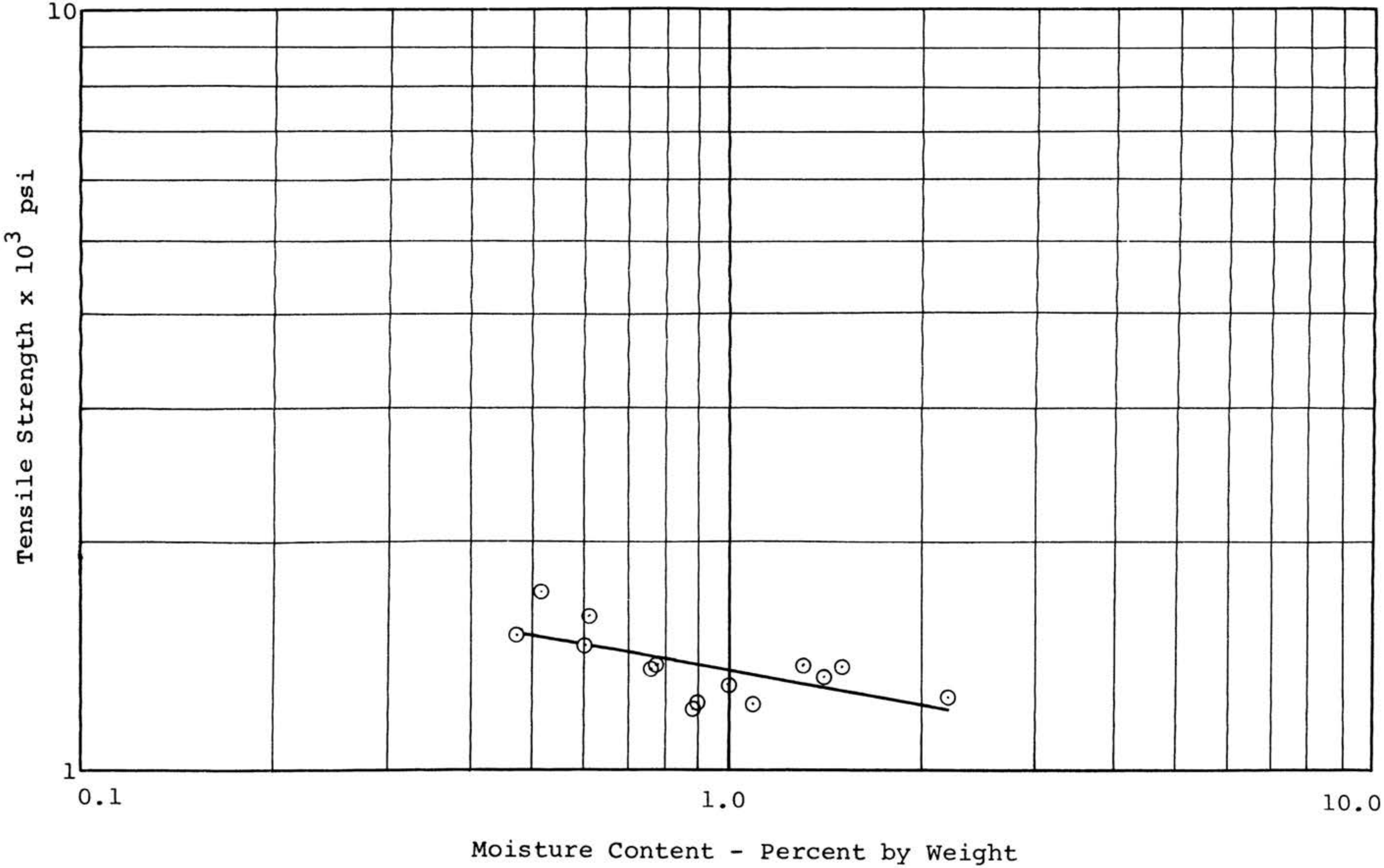


Figure 63 Moisture Content vs Tensile Strength for Shale.

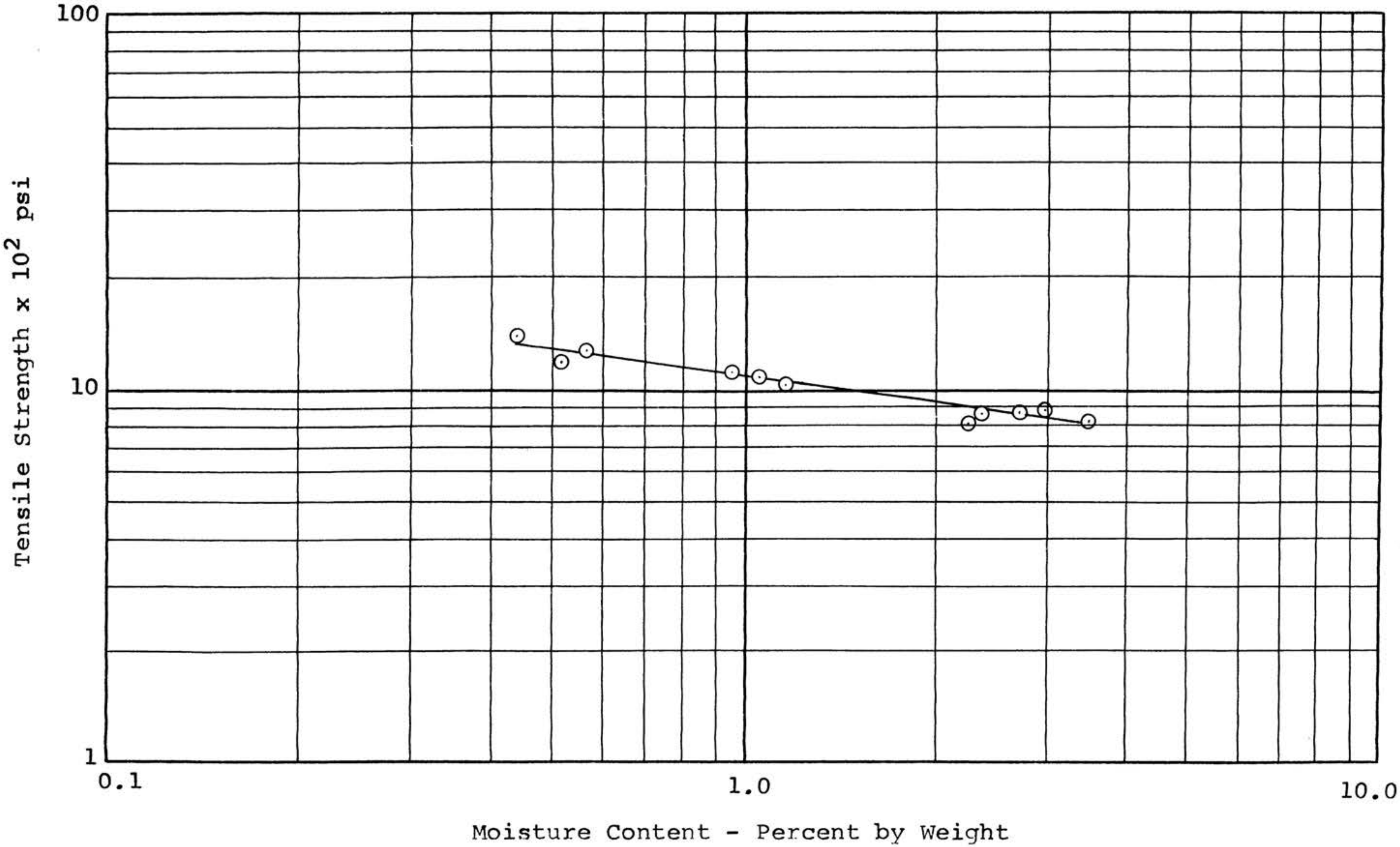


Figure 64 Moisture Content vs Tensile Strength for White Pine Sandstone.

V. CONCLUSIONS

A. Failure Pattern

Rock material can fail under a load either by fracturing the bonds between grains, by fracturing the bonds on a plane of weakness, by fracturing through the grains, or by a combination of these phases. The material will fail along a critical path where the localized stress overcomes the strength. The plane of least resistance will not be the region of failure unless a load of sufficient magnitude is applied to sever the bonds.

Sedimentary and metamorphic rock will tend to fracture through bonds which hold the grains together and along zones where there is little or no bonding. Igneous rock will fail along planes of weakness or through the solidified mass.

The degree of variation in rock strength in material of the same origin and general location must have been caused by the chemical and mechanical action imposed upon them over their geologic history. The strength characteristics of the Pea Ridge samples show a high degree of variability which may have been caused by one or more of the following:

1) A local difference in geologic history.
2) A locally different chemical composition.
3) Being non-isotropic, a different internal loading pattern.

B. Effect of Moisture

In the compressive tests, the samples were more apt to fail along planes of weakness. As more moisture was present in the void areas, specifically in zones of weakness, the resistance to failure was decreased.

The strength characteristics of the samples seem to be independent of porosity. The material with the highest porosity does not seem to show a higher or lower degree of change in strength from the dried to the saturated state. Rock which has a relatively small void area can be affected by an increase in moisture just as pronounced as materials with a higher void area. Therefore, soil mechanics theories of pore water pressures may also be applicable to the field of rock mechanics. The major discontinuity in the theory as applied to rock material stems from the fact that rock does not have one-tenth the strength in tension that it does in compression, as can be seen from the test results. If the load applied to a specimen in compression was directly applied to the moisture in the pore areas, which in turn applied a tensile stress perpendicular to the load, the sample should fail at a very low compressive force. This does not happen. Therefore, not all the load can be transmitted to the interstitial moisture. The amount of pore water pressure will always be questionable.

The capillary tensions produced, the lubricative effects, and the chamical effects of moisture on the bonding which resist and assist in failing a rock material must also be taken into consideration. The chemical composition and physical structure of the material are the factors which most influence their relative effects on strength.

C. Significance of Research

There is no one specific reason for a decrease or increase in strength of the rock materials tested. The effects of moisture on rock strength have been presented to bring out the fact that moisture, which is always present in mine rock, must be considered in future rock mechanics design. It may also become economical to dewater a zone which shows pronounced weakening with the presence of moisture or, on the other hand, saturate an area where caving is desired. The effect of moisture and strength on open pit mining operations and slope stability designs must also be considered. Determining the allowable design strength of rock at the present time is a difficult task, but the effect of moisture should not be neglected.

D. Recommendations

This investigation suggests that the following problems should be studied in greater detail:

1) Measurements on the pore water pressures developed to determine their influence on the saturated strength of rock.
2) Tri-axial tests to determine if the effects of moisture can be offset by a confining load.
3) Determination of the effect of ground water, not atmospheric moisture, on rock strength.

4) An economic study of the feasibility of dewatering an area as opposed to using a low wet strength for design of a mining operation.

5) A more precise study of the mechanism of failure of a dry material as opposed to a saturated speciman.

BIBLIOGRAPHY

1. RAZVI, M.A. (1962). The Effect of Moisture on the Compressive Strength and Modulus of Elasticity of Limestone. Thesis, Colorado School of Mines, 86 p.

2. COLBACK, P.S.B. and WIID, B.L. (1964). The Influence of Moisture on the Compressive Strength of Rock. South African Council for Scientific and Industrial Research, Pretoria, 8 p.

3. OBERT, L., WINDES, S.L., and DUVALL, W.I. (1946). Standardized Tests for Determining the Physical Properties of Mine Rock. U.S. Bureau of Mines Report of Investigation, No. 3891, p. 12-23.

4. HARDY, H.R. (1959). Standardized Procedures for Determination of the Physical Properties of Mine Rock Under Short-Period Uniaxial Compression. Canadian Department of Mines and Technical Surveys, Tech. Bull. 8, p. 80.

5. SPANGLER, M.G. (1960). Soil Engineering. Second Edition, International Textbook, Pennsylvania, p. 104-114.

6. SOWERS, G.B. and SOWERS, G.F. (1961). Introductory Soil Mechanics and Foundations. Second Edition, Macmillan, New York, P. 46-47 and P. 69-77.

7. WRIGHT, P.J.E. (1955). Comments on an Indirect Test on Concrete Cylinders. Magazine on Concrete Research, Vol. 7, No. 20, p. 87-96.

8. FROCT, M.M. (1948). Photoelasticity. John Wiley and Sons, New York, Vol. 2, p 121-129.

9. TIMOSHENKO, S. (1934). Theory of Elasticity. First Edition, McGraw-Hill Book Co., Inc., New York, p. 104-108.

10. SPENCER, H.M. (1926). Laboratory Methods for Maintaining Constant Humidity. International Critical Tables of Numerical Data- Physics, Chemistry, and Technology, Vol. I. McGraw-Hill Book Co., Inc., p. 67.

11. -------------, (1964). Meramec Iron Ore Project Starts Production at Pea Ridge. Engineering and Mining Journal, Vol. 165, No. 4, p. 94.

12. BATEMAN, A.M. (1958). Economic Mineral Deposits. Second Edition, John Wiley and Sons, Inc., p. 571.

13. OROWAN, E. (1949). Fracture and Strength of Solids. Reports on Progress in Physics, Vol. 12, p. 185-232.

14. SCOTT, R.F. (1963). Principles of Soil Mechanics. Addison-Wesley Publishing Co., Inc., p. 52-54.

VITA

The author was born on December 25, 1941, in Santa Barbara, California. He received his primary and secondary education in Oakland, California. With the aid of the Women's Auxiliary to the American Institute of Mining, Metallurgical, and Petroleum Engineers Scholarship-Loan, he entered the University of Missouri School of Mines and Metallurgy in 1959, and received the degree of Bachelor of Science in Mining Engineering in 1964.

In 1964, he enrolled in the graduate school of the University of Missouri, School of Mines and Metallurgy, Rolla.

His practical experience included summer employment with Peabody Coal Company, Ohio, and Phelps Dodge Copper Corporation, Arizona. He was also on a co-operative training program with Pittsburg and Midway Coal Mining Company for the period February, 1963 to September, 1963.

In August, 1965, he accepted a permanent position with Dames and Moore Consulting Engineers in their Los Angeles office.

APPENDIX I

COMPRESSION TEST DATA

TABLE 3 Results of the Compressive Tests of the Pea Ridge AX Magnetite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.0074	22,260	4.74		1eD-75°	
0	-0.0082	31,320	5.00		2eD-70°	
0	-0.0147	36,260	4.87		1cE	Omitted
0	-0.0184	18,250	4.68		2dE	
0	-0.0167	22,570	4.96		1eE-65°	
0	-0.0276	8,550	4.52		2eD-65°	Omitted
0	-0.0203	32,850	4.63		1bE-70°	
0	-0.0070	16,380	4.09		1cE-70°	
66	+0.0491	9,160	4.60		4eE-60°	Omitted
66	+0.0031	18,630	4.96		3dE	
66	+0.0039	15,660	4.58		3b,dE	
66	+0.0000	17,030	4.82		1eD-60°	
66	+0.0011	34,120	4.53		1cE-70°	
66	+0.0056	17,770	4.74		3dE	
66	-0.0015	38,280	5.03		1fE	Omitted
98	+0.0543	40,460	5.01	2.6323	1dE	Omitted
98	+0.0365	26,820	4.78	4.5375	1eE-75°	
98	+0.1265	22,440	4.64	7.1331	2aE-70°	
98	+0.0159	35,990	4.98	2.9084	2fE	Omitted
98	+0.0620	20,980	4.53	2.0456	3cE-70°	
98	+0.2758	9,580	4.86		3eE-65°	Omitted
98	+0.0545	21,250	4.65		3dE	

TABLE 3 Continued

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
100	+0.1591	19,210	4.53		2bE-60°	
100	+0.4598	16,770	4.65		3cE-70°	
100	+0.6940	16,340	4.72		4cE-70°	
100	+0.1616	23,230	4.80		1cE-70°	
100	+0.1899	21,540	4.85		2eE-65°	
100	+0.1550	27,000	4.78		1cE-70°	Omitted
100	+0.0310	42,040	4.92		1cE-70°	Omitted
100	+0.1886	40,370	4.63		1cE-70°	Omitted
100	+0.4328	10,490	4.29		4dE	Omitted

TABLE 4 Results of the Compressive Tests of the Pea Ridge EX Magnetite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.0311	29,700	4.14		1cE-70°	Omitted
0	-0.0199	19,130	4.35		3cE-70°	
0	-0.0446	20,670	4.17		3dE	Omitted
0	-0.0050	30,390	4.86			
0	-0.0085	21,040	4.74		2dD	
0	-0.0098	40,970	4.66		1cE-75°	Omitted
0	-0.0131	27,710	4.74		3cE-60°	
0	-0.0282	27,700	4.34		3cE-70°	
0	-0.0124	17,250	3.94		3cE-55°	
0	-0.0071	18,110	4.46		4eE-60°	
66	-0.0041	7,690	4.77		4bE-70°	Omitted
66	+0.0076	23,010	4.31		3c,dE-60°	
66	+0.0095	19,510	4.18		3fE	
66	-0.0124	20,550	3.77		3cE-60°	
66	+0.0136	11,180	4.50		4dE	Omitted
66	+0.0030	15,400	4.71		4cE-70°	
66	+0.0052	39,310	4.80		2bE-70°	Omitted
66	-0.0009	15,530	4.85		4cE-70°	
66	+0.0005	11,810	4.56		4dE	Omitted
66	-0.0016	12,820	4.61		4fE	Omitted
98	+0.0206	5,050	4.75		4eE-70°	Omitted
98	+0.0367	12,050	4.20		4dE	Omitted

TABLE 4 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
98	+0.0974	19,180	4.50		3cE-70°	
98	+0.0531	20,440	4.34		3fE	
98	+0.0472	21,980	4.06		3eE-70°	
98	+0.1258	14,580	3.17		3cE-75°	Omitted
98	+0.0258	15,830	4.65		4cE-70°	Omitted
98	+0.0243	24,820	4.71		4aE-70°	
98	+0.0470	8,050	4.77		4fE	Omitted
98	+0.0129	23,120	4.83		2cE-70°	
100	+0.0842	13,630	4.06		4fE	Omitted
100	+0.1022	13,040	4.32		4fE	Omitted
100	+0.0657	29,600	4.79		2cE-70°	Omitted
100	+0.4843	14,670	4.79		4cE-70°	
100	+0.2830	17,960	3.82		4bE-75°	
100	+0.5091	20,170	4.61		3eE-75°	
100	+0.9736	15,070	4.48		4eE-75°	
100	+0.2542	29,650	4.75		2cE-70°	Omitted
100	+0.4346	16,910	4.76		3fE	
100	+0.3204	12,480	4.10		3cE-65°	Omitted

TABLE 5 Results of the Compressive Test of the Pea Ridge Hematite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.1178	8,550	3.52		4eE	Omitted
0	-0.0035	53,100	4.78		1cE-70°	Omitted
0	-0.0116	9,840	4.66		4eE-60°	Omitted
0	-0.0367	7,960	4.74		4eE-70°	Omitted
0	-0.0033	22,270	4.56		3d,eE-80°	
0	-0.0056	31,140	4.86		1cE-70°	
0	-0.0019	25,220	4.85		1cE-65°	
0	-0.0029	35,790	4.98		1dE	
66	-0.0005	14,160	4.62		4cE-70°	Omitted
66	+0.0027	8,210	4.72		4cE-70°	Omitted
66	+0.0000	21,820	4.74		3eE-70°	
66	-0.0005	30,050	4.86		2cE-65°	
66	+0.0363	41,820	4.82		2cE-65°	Omitted
66	+0.0084	28,260	4.59		1aE-65°	
66	-0.0004	30,140	4.93		2cE-70°	
98	+0.0891	25,140	4.00	4.7842	3eE-70°	
98	+0.1821	15,820	4.80	5.3899	4cE-70°	Omitted
98	+0.1266	61,000	4.86	2.2129	1gC	Omitted
98	+0.0133	21,660	4.96	4.0441	3aE-70°	
98	+0.0259	20,540	4.79	7.1314	2eE-75°	
98	+0.0473	9,840	4.57		4a,dE-70°	Omitted
98	+0.0026	25,320	4.62		3eE-70°	

TABLE 5 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
100	+3.1557	6,710	3.44		4cE-70°	Omitted
100	+0.0080	55,750	4.78		1cE-70°	Omitted
100	+0.0725	21,480	4.61		3eE-65°	
100	+0.0680	26,110	4.81		2cE-70°	
100	+0.3300	18,830	4.81		4eE-70°	
100	+0.6094	19,720	4.75		3dE	
100	+0.1345	36,720	4.78		1dE	Omitted
100	+0.7479	16,590	4.53		4fE	
100	+0.0673	37,090	4.93		1eE-70°	Omitted

TABLE 6 Results of the Compressive Test of the Pea Ridge Porphyry.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.0174	45,130	2.60		1dC	
0	-0.0229	38,320	3.07		1dE	
0	-0.0107	43,290	2.57		1dE	
0	-0.0205	40,470	2.83		1b,dE-70°	
0	-0.0869	10,480	2.66		3b,dE-70°	Omitted
0	-0.0196	36,370	2.55		1dE	
0	-0.0098	44,740	2.56		1d,aE -70°	
0	-0.0172	33,790	2.56		1d,cE-75°	
0	-0.0075	27,350	2.57		1eE-80°	Omitted
0	-0.0223	47,090	2.57		1d,aE-70°	
66	+0.0228	22,850	2.64		2c,dE-75°	Omitted
66	-0.0061	59,390	2.59		1gC	Omitted
66	+0.0933	23,600	2.71		1eD-60°	
66	+0.0000	26,650	2.56		1dE	
66	+0.0075	15,120	2.69		3eE-70°	Omitted
66	+0.0073	30,180	2.84		1dE	
66	+0.0063	32,470	2.57		1cE-75°	
66	+0.0069	45,220	2.55		1cC-75°	
66	+0.0074	58,600	2.57		1gC	Omitted
66	+0.0066	31,050	2.56		1aE-65°	
98	+0.1029	54,870	2.57	3.0505	1dC	Omitted
98	+0.1486	45,130	2.58	2.6100	1gC	Omitted

TABLE 6 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
98	+0.0638	19,430	2.77		1dE	Omitted
98	+0.1217	19,780	2.80	3.5404	1eE-60°	Omitted
98	+0.1237	17,110	2.68	2.1640	3eE-55°	Omitted
98	+0.0665	55,560	2.57	2.0733	1cE-70°	Omitted
98	+0.0862	37,620	2.72		2dE	
98	+0.0607	43,720	2.58		1dE	
98	+0.0663	43,350	2.56		1fE	
98	+0.0062	51,860	2.52		1dE	Omitted
98	+0.1423	46,210	2.55		1gC	Omitted
100	+0.2728	42,570	2.69		1fE	Omitted
100	+0.1999	33,920	2.64		2cE-70°	
100	+0.0671	33,990	2.88		1cE-80°	
100	+0.5760	16,510	2.99		2dE	
100	+0.0259	50,190	2.57		1gC	
100	+0.0703	24,450	2.53		1fE	
100	+0.0339	43,270	2.55		1gE	
100	+0.0739	47,400	2.57		1gC	Omitted
100	+0.1929	22,920	2.54		1fE	

TABLE 7 Result of the Compressive Tests of the Pea Ridge Quartzite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.4249	11,820	2.95		4dE	Omitted
0	-0.0093	32,830	2.80		3dC	
0	-0.0208	18,500	2.84		3d,bE	Omitted
0	-0.0626	12,970	3.15		3eD	Omitted
0	-0.0064	41,880	2.77		1dC	
0	-0.0041	22,470	2.65		3dE	
0	-0.0097	37,570	2.58		1dE	
0	-0.0041	30,350	3.63		2e,cD	
0	-0.0298	33,120	3.04		3d,bD	Omitted
66	+0.0034	24,870	2.95		2eE-75°	
66	+0.0099	36,280	2.70		1dE	
66	+0.2313	12,700	3.02		4aE-70°	
66	+0.2235	14,210	3.10		4bE-65°	
66	+0.0000	29,060	2.86		2c,dE-75°	
66	+0.0344	30,230	2.88		2c,dE-70°	
66	+0.0447	7,790	2.77		4dE	Omitted
66	+0.0052	44,590	2.50		1gC	Omitted
66	+0.0164	18,940	2.61		4bE-60°	
98	+0.0927	24,640	3.17		2dE	
98	+0.0283	22,950	2.86		2dE	
98	+0.0170	25,820	2.79		2eE-70°	
98	+0.1004	20,360	3.01			
98	+0.0274	32,110	2.64		1gC	

TABLE 7 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
98	+0.0257	17,900	2.88		2fE	
98	+0.0257	22,540	2.86		2eE-75°	
98	+0.2312	13,810	2.83		3fE	
98	+0.0847	37,410	2.57		3gC	Omitted
100	+0.0465	22,350	2.95			
100	+0.0312	17,580	3.17		2eD-60°	
100	+0.0492	27,500	3.03		1dE	
100	+0.2628	28,140	3.91		2a,dE-70°	Omitted
100	+0.1638	43,950	2.84		1gC	Omitted
100	+0.4689	10,310	3.26		4fE	
100	+0.1108	12,700	2.86		3fE	
100	+0.0731	35,240	2.55		2dE	
100	+0.0599	12,830	2.63		3dE	
100	+0.4731	13,450	2.85		4fD	

TABLE 8 Results of the Compressive Tests of the Pea Ridge Sandstone.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.0641	6,510	2.14		3dE	Omitted
0	-0.1049	13,470	2.30		2eE-75°	Omitted
0	-0.1816	12,100	2.24		2cE-75°	Omitted
0	-0.0374	11,420	2.25		3eE-65°	Omitted
0	-0.0396	25,020	2.40		1cE-65°	Omitted
0	-0.0519	7,490	2.30		2eE-60°	Omitted
0	-0.1038	11,800	2.30		3a,eE-65°	Omitted
0	-0.0675	10,560	2.30		3eE-70°	Omitted
0	-0.0809	7,140	2.12		3eE-70°	Omitted
0	-0.1386	14,630	2.28		2b,dE-70°	Omitted
0	-0.1216	14,500	2.29		2fE	Omitted
66	+0.0311	5,300	2.23		4b,eE-65°	Omitted
66	+0.0386	9,430	2.31		4eE-65°	Omitted
66	+0.0457	10,960	2.57		3dE	Omitted
66	+0.0852	7,920	2.19		4cE-65°	Omitted
66	+0.0228	5,900	2.21		4eE	Omitted
66	+0.0413	5,980	2.24		4fE	Omitted
66	+0.0765	4,430	2.23		4eE-60°	Omitted
66	+0.0848	6,180	2.09		4eE-65°	Omitted
66	+0.1305	8,250	2.38		4dE	Omitted
66	+0.1325	13,250	2.29		4cE-70°	Omitted
66	+0.0232	8,250	2.44		4aE-70°	Omitted

TABLE 8 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
98	+1.4100	7,020	2.33		4cE-70°	Omitted
98	+0.0882	8,470	2.29		3cE-65°	Omitted
98	+0.4949	5,130	2.17	15.7039	4fE	Omitted
98	+0.7493	7,190	2.12		3cE-70°	Omitted
98	+1.4246	9,320	2.21	21,5831	3b,dE-70°	Omitted
98	+1.4803	6,180	2.34		4fE	Omitted
98	+0.4393	11,050	2.14		3cE-65°	Omitted
98	+2.4959	10,320	2.22		2eE-70°	Omitted
98	+2.7197	2,900	2.25		4fE	Omitted
98	+0.1723	7,350	2.48		3dE	Omitted
100	+5.1663	6,960	2.28		4eE-60°	Omitted
100	+4.0561	15,840	2.36		2eE-70°	Omitted
100	+6.8606	2,160	2.16		4eE-65°	Omitted
100	+5.6510	12,400	2.26		4b,eE-60°	Omitted
100	+1.0796	12,250	2.57		2dE	Omitted
100	+6.4620	4,230	2.22		4dE	Omitted
100	+2.9554	6,040	2.47		4eE-70°	Omitted
100	+1.9478	14,210	2.38		2cE-70°	Omitted
100	+4.1510	12,450	2.36		3eE-65°	Omitted
100	+7.6379	3,340	2.16		4fE	Omitted
100	+3.6983	11,360	2.33			Omitted

TABLE 9 Results of the Compressive Tests of the Pea Ridge Dolomite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
0	-0.1840	46,110	3.55		1cE-65°	Omitted
0	-0.0878	26,470	2.70		1dE	
0	-0.2365	36,400	2.67		1eE-70°	
0	-0.0432	20,510	2.67		1dE	
0	-0.0158	33,930	2.83		1dE	Omitted
0	-0.2791	22,750	2.60		1eE-65°	
0	-0.2224	17,170	2.64		2dE	
0	-0.2100	17,540	2.60		2dE	
0	-0.0668	19,230	2.66		2a,dE-65°	
0	-0.1712	20,880	2.64		2a,dE-65°	
0	-0.1333	10,880	2.55		3b,dE-65°	Omitted
66	+0.1758	25,850	3.36		2dE	Omitted
66	+0.5592	8,980	2.57		4fE	
66	+0.1436	22,160	2.63		2dE	Omitted
66	+0.3102	10,670	2.62		3dE	
66	+0.0267	15,520	2.72		3a,dE-70°	
66	+0.1118	16,300	2.68		3dE	
66	+0.0624	15,230	2.67			
66	+0.0684	21,920	2.66		2cE-65°	
66	+0.0683	13,640	2.62		3dE	
66	+0.0245	20,700	2.70			
66	+0.1664	13,290	2.64			
66	+0.0557	15,750	2.81		2d,eE-60°	

TABLE 9 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams per Cubic Cent.)	Porosity (%)	Fracture Classification	Remarks
98	+0.1572	21,940	2.64	2.5729	2dE	Omitted
98	+0.1575	13,940	2.64	3.7396	3dE	
98	+0.0430	46,020	2.80		1fD	Omitted
98	+1.0591	5,530	2.60		4eE-55°	Omitted
98	+0.2645	18,580	2.60	2.7737	2eE-70°	
98	+0.2559	5,530	2.57		4fE	Omitted
98	+0.1626	20,930	2.70	2.4770	2dE	Omitted
98	+0.5286	11,450	2.62		3eE-70°	
98	+0.3650	12,480	2.61		2fE	
98	+0.2056	14,380	2.61		3dE	
98	+0.2044	15,680	2.45		3dE	
98	+0.0851	19,890	2.78	2.1463	2dE	
100	+0.1872	14,810	2.66		3fE	
100	+1.8412	6,870	2.55		4eE	
100	+1.6500	12,600	2.65		3dE	
100	+0.1683	19,100	2.64		3dE	
100	+0.6051	15,020	2.67		3e,dE-70°	
100	+0.4901	8,920	2.69		4eE-70°	
100	+0.1667	14,540	2.95		2dE	
100	+0.8526	12,270	2.63		3dE	
100	+0.3740	28,700	2.77		2c,dE-75°	Omitted
100	+0.5383	8,240	2.60			
100	+0.2270	14,110	2.65		3dE	

TABLE 10 Results of the Compressive Tests of the White Pine Shale.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Fracture Classification	Remarks
0	0	29,790	2.57		2cD-70°	
0	0	31,610	2.68		1cD-70°	
0	0	27,290	2.68		2eD-70°	
0	0	30,230	2.68		1eD-75°	
0	0	20,500	2.67		2dE	Omitted
0	0	29,600	2.57		1eD-80°	
0	0	30,220	2.62		1cD-70°	
0	0	31,570	2.66		1eD-70°	
0	0	29,410	2.67		1cD-70°	
0	0	27,640	2.59		1eD-70°	
66	0.4303	17,970	2.69		2dE	
66	0.6125	22,480	2.63		1eE-65°	
66	0.6155	12,190	2.68		3fE	
66	0.6227	21,920	2.66		2dE	
66	0.4925	20,420	2.56		1eE-65°	
66	0.7657	14,690	2.71		3dE	
66	0.4886	18,200	2.61		2eE-70°	
66	0.6303	17,260	2.64		2dE	
66	0.5150	8,160	2.68		4eD-60°	Omitted
66	0.7399	16,390	2.69		2fE	
66	0.3089	20,530	2.61		3cE-70°	

TABLE 10 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Fracture Classification	Remarks
98	1.4745	17,530	2.64		2cE-70°	
98	1.6069	17,280	2.62		2cE-70°	
98	0.6610	14,930	2.67		1eE-75°	
98	1.2146	2,390	2.69		4eE-25°	Omitted
98	0.7705	12,430	2.67		2eE-80°	
98	1.8453	13,290	2.63		2dE	
98	1.3679	18,640	2.59		3eE-60°	
98	0.9263	10,960	2.61		4dE	
98	0.6572	17,370	2.63		2bE-70°	
98	0.4853	14,050	2.69		2eE-80°	
98	1.7294	16,810	2.62		2cE-70°	
100	1.6525	1,970	2.67		4dE-70°	Omitted
100	0.7186	13,410	2.70	3.7219	3eE-70°	
100	0.7309	10,960	2.67		3eE-70°	
100	0.8464	6,340	2.67		3eE-70°	Omitted
100	1.7762	15,400	2.63		3aE-60°	
100	1.9675	17,570	2.63		3cE-70°	
100	0.7698	24,100	2.69		1fE	Omitted
100	1.5644	12,260	2.65		3dE	
100	2.2089	15,970	2.59		2eE-70°	
100	1.2814	11,640	2.69		3eE-75°	
100	0.8741	16,660	2.68		2eE-75°	

TABLE 11 Results of the Compressive Tests of the White Pine Sandstone.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Fracture Classification	Remarks
0	0	18,260	2.49		4eE-60°	
0	0	20,090	2.48		3cE-60°	
0	0	19,930	2.52		3cE-60°	
0	0	21,350	2.54		3cE-60°	
0	0	17,010	2.49		4cE-70°	Omitted
0	0	17,960	2.48		3eE-65°	
0	0	23,260	2.57		3cE-65°	Omitted
0	0	17,780	2.47		3cE-70°	
0	0	24,740	2.61		2cE-65°	Omitted
66	0.4912	13,940	2.55		3eE-70°	
66	0.5275	17,180	2.61		3eE-65°	
66	0.4116	11,720	2.51		4eE-65°	Omitted
66	0.3920	14,580	2.51		4cE-70°	
66	0.3602	14,310	2.55		4aE-70°	
66	0.4885	17,090	2.59		3bE-70°	
66	0.5585	17,130	2.60		3cE-70°	
66	0.4175	13,650	2.47		4cE-70°	
66	0.6322	18,410	2.63		2eE-60°	

TABLE 11 Continued.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Fracture Classification	Remarks
98	1.4894	12,690	2.59		2bE-70°	
98	1.2688	12,700	2.54		3cE-70°	
98	1.4499	11,420	2.49		4aE-70°	
98	0.7229	15,380	2.62		2e,dE-60°	
98	1.6920	10,080	2.49		4cE-70°	
98	1.3684	12,110	2.53		4cE-70°	
100	3.1880	10,650	2.50	9.9763	4eE-70°	
100	3.4576	9,130	2.48		4cE-65°	
100	3.0941	11,440	2.48		4eE-70°	
100	2.6931	10,190	2.50	10.3754	4eE-60°	
100	1.1976	20,300	2.60	5.4661	1dE	Omitted
100	1.8595	11,420	2.58		3dE	
100	3.4726	9,840	2.50	11.0913	4eE-65°	
100	3.3677	11,180	2.48	13.5676	4cD-70°	
100	3.5181	6,900	2.49		4eE-65°	Omitted

APPENDIX II

TENSILE TEST DATA

TABLE 12 Results of the Tensile Tests of the Pea Ridge AX Core Magnetite

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch	Density Grams Per Cubic Cent.	Porosity (%)	Remarks
0	-0.0024	1660	5.02		Omitted
0	-0.0296	790	4.81		Omitted
0	-0.0154	1130	4.79		Omitted
0	-0.0150	1230	5.07		Omitted
0	-0.0410	880	4.56		Omitted
0	-0.0251	1100	4.61		Omitted
0	-0.0217	880	4.98		Omitted
0	-0.0191	1020	4.91		Omitted
0	-0.0246	1720	4.41		Omitted
66	+0.0093	1540	4.66		Omitted
66	+0.0058	730	4.63		Omitted
66	+0.0023	1030	4.79		Omitted
66	+0.0007	1160	4.87		Omitted
66	-0.0027	1530	4.78		Omitted
66	+0.0070	1430	4.59		Omitted
66	+0.0041	1590	4.50		Omitted
66	-0.0013	1060	4.43		Omitted
98	+0.0397	1630	4.74		Omitted
98	+0.1152	3630	4.70		Omitted
98	+0.0356	1320	4.58		Omitted
98	+0.0363	1510	4.66		Omitted
98	+0.0515	1170	4.83		Omitted
98	+0.0151	1180	4.98		Omitted
98	+0.0570	1040	4.41		Omitted
98	+0.0321	1110	4.96		Omitted
98	+0.0276	1230	4.69		Omitted
100	+0.5954	1220	4.85		Omitted
100	+0.1649	1550	4.56		Omitted
100	+0.1012	1900	4.07		Omitted
100	+0.1222	1130	4.94		Omitted
100	+0.1243	1250	4.85		Omitted
100	+0.2180	1040	4.53		Omitted
100	+0.1540	810	4.80		Omitted
100	+0.1063	1310	4.77		Omitted
100	+0.1000	1310	4.94		Omitted

TABLE 13 Results of the Tensile Tests of the Pea Ridge EX Core Magnetite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.0148	1870	3.29		
0	-0.0074	2120	4.76		
0	-0.0056	2390	5.00		
0	-0.0076	1480	4.62		Omitted
0	-0.0160	1400	4.55		Omitted
0	-0.0200	2180	4.92		
66	+0.0118	1730	3.20		
66	+0.0036	2010	4.82		
66	+0.0050	1410	4.47		
66	+0.0154	1560	4.50		
66	+0.0050	2280	4.73		Omitted
66	+0.0026	1290	4.77		Omitted
98	+0.1128	1340	4.19		Omitted
98	+0.0195	1900	4.80		
98	+0.0818	1640	3.77		
98	+0.0364	1750	4.45		
98	+0.0148	1710	4.71		
98	+0.0458	1930	4.73		
100	+0.0863	1370	4.33		Omitted
100	+0.0646	1990	4.86		
100	+0.0968	1870	4.93		
100	+0.5504	1560	3.35		
100	+0.1625	1510	4.65		
100	+0.3747	1440	4.56		

TABLE 14 Results of the Tensile Tests of the Pea Ridge Hematite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.1388	2070	4.23		Omitted
0	-0.0630	2770	4.49		
0	-0.1036	970	4.09		Omitted
0	-0.0144	2840	4.46		
0	-0.0106	2580	5.01		
66	+0.0318	1960	3.52		
66	+0.0227	1900	4.36		
66	+0.0335	1440	4.43		Omitted
66	+0.0051	2340	4.83		
98	+0.4087	1140	4.53		Omitted
98	+0.2284	2000	3.63		
98	+0.2065	1390	4.46		Omitted
98	+0.0511	2230	4.77		
100	+1.7747	1730	3.27		
100	+0.1622	2450	4.25		
100	+0.6285	1670	4.42		
100	+0.0382	2040	4.27		

TABLE 15 Results of the Tensile Tests of the Pea Ridge Porphyry.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.0268	1980	2.72		
0	-0.0160	1710	2.62		
0	-0.0211	1670	3.17		
0	-0.0098	2320	2.54		Omitted
0	-0.0425	1640	2.58		Omitted
66	+0.0119	1790	2.64		
66	+0.0096	2080	2.80		
66	+0.0738	1810	2.52		
66	+0.0057	2050	2.55		
66	+0.0295	2050	2.53		
98	+0.3616	1630	2.80		Omitted
98	+0.0405	2330	2.57		
98	+0.0087	2540	2.59		Omitted
98	+0.0599	1820	2.68		
98	+0.1508	2510	2.48		
100	+0.0617	2800	2.57		Omitted
100	+0.2083	1360	2.94		Omitted
100	+0.0693	2230	2.56		
100	+0.1017	2300	2.51		

TABLE 16 Results of the Tensile Tests of the Pea Ridge Quartzite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.0132	1400	2.88		Omitted
0	-0.4456	1350	2.86		Omitted
0	-0.3204	1440	2.94		
0	-0.0068	2280	2.57		Omitted
0	-0.0726	1950	2.72		
0	-0.0330	1400	3.42	2.2351	Omitted
0	-0.2805	2140	3.14		
0	-0.1250	1920	2.62		
66	+0.2040	1790	2.90		Omitted
66	+0.0237	2050	2.90		
66	+0.0185	1840	2.75		
66	+0.0087	2350	2.73		
66	+0.2200	1620	3.16		Omitted
66	+0.1247	1620	2.98		Omitted
66	+0.0404	2660	3.02		Omitted
66	+0.0425	1470	2.60		Omitted
66	+0.1207	1270	2.63		Omitted
98	+0.0379	2120	2.91	4.0663	
98	+0.1675	2040	3.02		
98	+0.0265	2140	2.79	1.8138	
98	+0.0282	2010	2.70	1.7459	
98	+0.0432	3140	3.14	3.2712	Omitted
98	+0.0334	2170	2.60		
98	+0.0091	2520	2.68		Omitted
100	+0.0443	2410	3.21		
100	+0.0654	2530	2.78		Omitted
100	+0.0492	2200	2.59		
100	+0.0809	2500	3.03		
100	+0.0720	2760	2.62		Omitted
100	+0.2088	1720	2.92		Omitted
100	+0.1308	2260	3.12		

TABLE 17 Results of the Tensile Tests of the Pea Ridge Sandstone.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.0882	620	2.24		Omitted
0	-0.0198	965	2.34		Omitted
0	-0.0375	850	2.26		Omitted
0	-0.1417	1030	2.34		Omitted
0	-0.0398	460	2.14		Omitted
0	-0.1036	720	2.18		Omitted
0	-0.0771	640	2.12		Omitted
0	-0.1846	1370	2.47		Omitted
66	+0.0327	1050	2.18		Omitted
66	+0.0102	520	2.24		Omitted
66	+0.0149	1020	2.28		Omitted
66	+0.0876	620	2.30		Omitted
66	+0.0314	990	2.54		Omitted
66	+0.0039	410	2.22		Omitted
98	+0.8118	300	2.11		Omitted
98	+0.7291	780	2.58		Omitted
98	+0.1016	610	2.33		Omitted
98	+0.4342	550	2.15		Omitted
98	+3.0973	180	2.22		Omitted
98	+0.1056	510	2.27		Omitted
98	+0.1357	600	2.28		Omitted
100	+3.3019	560	2.35		Omitted
100	+5.3088	420	2.29		Omitted
100	+5.7282	480	2.25		Omitted
100	+3.8741	980	2.34		Omitted

TABLE 18 Results of the Tensile Tests of the Pea Ridge Dolomite.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Density (Grams Per Cubic Cent)	Porosity (%)	Remarks
0	-0.1566	2160	3.57		
0	-0.0252	1840	2.66		
0	-0.0392	1630	2.66		
0	-0.1878	1560	2.62		Omitted
0	-0.0224	2260	2.83		
0	-0.1053	1590	2.57		
66	+0.1387	1600	3.52		
66	+0.5074	920	2.58		
66	+0.1797	1010	2.64		
66	+0.2717	1050	2.64		
66	+0.4154	1600	2.63		Omitted
66	+0.0495	990	2.65		Omitted
66	+0.1966	1270	2.68		
98	+0.0149	2280	3.01	1.5780	
98	+0.0111	2250	2.72		
98	+0.1453	1220	2.65		
98	+0.0753	2670	3.12		Omitted
98	+0.5443	1200	2.60		Omitted
98	+0.4476	1150	2.62		
100	+0.0465	1940	2.90		
100	+0.5664	940	2.62		
100	+0.1728	1180	2.65		
100	+0.1227	980	2.67		
100	+0.1109	850	2.65		Omitted
100	+0.4613	870	2.71		
100	+0.0221	2500	2.82		Omitted

TABLE 19 Results of the Tensile Tests of the White Pine Shale.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Remarks
0	0	1090	2.60		Omitted
0	0	2220	2.65		
0	0	980	2.66		Omitted
0	0	2100	2.60		
0	0	1760	2.69		
0	0	2770	2.71		Omitted
0	0	1780	2.62		
66	0.5116	1720	2.69		
66	0.7797	900	2.69		Omitted
66	0.6000	1460	2.67		
66	0.4692	1510	2.67		
66	0.7104	2320	2.64		Omitted
98	0.9502	610	2.69		Omitted
98	0.6043	1600	2.69		
98	1.3006	1380	2.70		
98	1.8911	1220	2.66		
98	0.9266	570	2.66		Omitted
98	0.8784	1210	2.59		
98	1.0020	1300	2.57		
98	0.8920	1230	2.65		
100	0.7736	1380	2.67	1.7002	
100	0.7600	1370	2.71	1.8183	
100	2.1890	1250	2.63	5.7989	
100	1.1569	1480	2.65	3.9182	Omitted
100	1.4004	1330	2.71		
100	1.4978	1370	2.64	5.2166	

TABLE 20 Results of the Tensile Tests of the White Pine Sandstone.

Relative Humidity (%)	Moisture Content (%)	Strength (Pounds per Square Inch)	Specific Gravity	Porosity (%)	Remarks
0	0	1500	2.49		
0	0	1630	2.56		
0	0	1310	2.58		
0	0	1350	2.48		
0	0	1420	2.50		
0	0	2500	2.66		Omitted
0	0	2200	2.63		Omitted
66	0.4408	1400	2.56		
66	0.4587	940	2.49		Omitted
66	0.5353	980	2.60		Omitted
66	0.3264	970	2.49		Omitted
66	0.5140	2210	2.58		Omitted
66	0.5173	1200	2.60		
66	0.5675	1280	2.55		
66	0.3251	1010	2.61		Omitted
98	1.0575	1090	2.51		
98	1.0034	940	2.46		Omitted
98	1.1637	1040	2.48		
98	0.9589	1130	2.54		
100	2.2487	820	2.49		
100	2.6917	880	2.49	10.4039	
100	2.3588	870	2.51	10.3322	
100	3.4705	830	2.45	10.2132	
100	2.9646	890	2.50	9.7969	
100	3.1940	700	2.52	13.1161	Omitted
100	1.3675	1030	2.60	6.0695	Omitted
100	1.3249	690	2.66		Omitted