PHYSICAL PROPERTIES
OF THE
PERMIAN RED CLAYS

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By

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PREFACE

For many years structural engineers were of the opinion that equal settlements could be produced for all footings of a building, if those footings were proportioned for equal contact pressures. In such designs very little consideration was given to the soil upon which the structures were to be built. Many sad experiences resulted from those designs.

Buildings of the skeleton type structural frame, both of concrete and steel, constructed on the clays have in many cases been damaged by differential movement within a relatively short time after construction. Instances have been recorded where damage was done to parts of a building by the movements even before construction was complete.

Professor R. E. Means became interested in the problem during the 1930's. About 1940 he began research in an effort to determine the cause of the differential movements of the buildings. His program had to be suspended soon after the beginning of World War II because of the demand on his time by the many war training programs conducted on the campus of Oklahoma A. and M. College in which he was participating.

Late in 1946 the research program was revived and expanded. Since that time the program has been divided into three coordinated projects. In one project research is being carried on in an attempt to develop a usable method of analysis of building frames in which the supporting soil is treated as an integral part of the structure, i.e., the relative stiffness of the frame and the supporting soil are considered in the design. The second is a project in which the actual measurements of vertical movements between parts of buildings are being made. An attempt is being made to find a correlation between the movements of the building and the fluctuations in the water table. The third is the laboratory measurement of the physical properties of the soil. The laboratory work is being done under the supervision of the author.

This thesis is essentially a progress report on one phase of the overall research program. The Permian clay was chosen for this first work because of the local interest in the behavior of that material under stress.

The writer is indebted to Professor R. E. Means for his guidance and helpful suggestions throughout the investigation; to Professor Ren G. Saxton, Head of the School of Civil Engineering, for providing space for the investigation; to Dr. Clark A. Dunn, Vice-Director of the Engineering Experiment Station, for his assistance in obtaining equipment for the investigation and his encouragement in the preparation of this thesis; and to all those men who have been instrumental in developing testing equipment and methods of tests for soils.

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I. INTRODUCTION

PURPOSE.

It is surprising that soil, the oldest construction material known to man, was the last of the more common materials to attract the interest of engineers. For many years the physical properties of such materials as wood, steel, and concrete have been measured in laboratories. The engineers and architects have applied the results of those measurements to the design of structures. Even though soil is the supporting material and becomes an integral part of a large percent of all structures built, it is only in the last relatively few years that any attempt has been made to measure the physical properties of soil, and to use those measurements in rational designs of structures.

Late in the nineteenth century a few of the larger cities throughout the world performed simple load tests in an attempt to determine safe supporting power of the different soil types. The results of those tests were misleading, the interpretations were false, and as a result designs based on those values have led to untold millions of dollars of damage to structures. The greater portion of the damage has been done to structures built on ~~the~~ clays.

The purpose of this investigation is to increase the knowledge of the physical properties of the Permian Red Clays, which are common to much of the area of Oklahoma; and to better understand the behavior of those clays under stress and the effects of climate.

SCOPE.

A complete analysis of the properties of the Permian clays is beyond the scope of this investigation. Due to the limited time, laboratory and sampling equipment, it was necessary to limit the tests to samples which could be taken in the vicinity of Stillwater, Oklahoma, from reasonably shallow depths. No claim is made that the results herein reported are representative of the Permian deposit or any subdivision of that deposit. The research performed the function of determining a pattern for more extensive research that is to follow.

OUTLINE OF TESTS.

Where possible the method of testing used was in accordance with adopted standard methods so that the results could be compared with those of other investigators.

Tests run to determine the properties of plasticity, grain size, and grain size distribution were performed in accordance with the methods described in the Standard Specifications (Tests) of the American Society of State Highway Officials.

Confined Compression tests, Triaxial Shear tests, and Swelling tests were run in apparatus similar to that developed by Professor A. Casagrande.(6)*

Each sample of soil was tested in a routine manner. Values of the following properties were measured; (a) Liquid Limit, (b) Plastic Limit, (c) Plastic Index, (d) Grain Size and Grain Size Distribution, (e) Consolidation, (f) Triaxial Shear, and (g) Swelling under Loads.

The results of all tests run are not reported in this thesis, but only those which seem to represent the extremes.

II. NATURE OF MATERIAL UNDER CONSIDERATION

CLAYS IN GENERAL.

Clay deposits exist as the result of sedimentation in bodies of

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*Number in parenthesis refers to Bibliography at the end of thesis.

relatively still fresh or salt waters. Where deposition in fresh water occurs the very fine fraction (colloidal sizes) remain in suspension for a sufficient time to permit the silt and coarse clay sizes to settle out as more or less single grains. That is not true where deposition occurs in salt waters.

Since the clay under consideration is a marine clay, i.e., one deposited in salt water, this discussion will be confined to the formation of clays deposited in salt water.

The origin of all soils is the rock minerals located in the upper reaches of the water sheds of the drainage systems found on the land portion of the earth's surface. Through a process of disintegration by mechanical and chemical weathering, the rock minerals are broken down into sizes that may be transported.

Transportation of the disintegrated rocks may be accomplished by either wind, water, or glacial action.[10] In mountainous areas the stream gradients are such that stream velocities are great enough to start large boulders on their trip to the sea. As the boulders are moved by the stream, the grinding action of one upon the other reduces them continually into smaller and smaller sizes. As the streams approach the sea, their gradients become less steep, the velocities diminish, and rock fragments of varied mass are deposited along the flood plain of the stream almost directly proportional to the stream gradient.

As one would expect, the soil grains which reach the sea are very finely divided fragments of those minerals which offered least resistance to abrasion. The sediment carried by the streams entering the sea is in general a suspension of soil grains of silt sizes down.

The surface of every soil grain is the seat of a negative charge of electricity. The strength of this charge is a function of the size of the particle, its shape, and its chemical composition.[10] When the size of the particle is in the order of magnitude of its mass, it will not settle out of suspension in a neutral water.[4] However, in the presence of an electrolyte the negative charges are neutralized, allowing flocs to form of sufficient mass to settle out of suspension. Marine clays are formed by a process of continuous sedimentation of flocs.

The floculation of marine clays has occurred at such a rapid rate that the flocs include many of the heavier silt grains. The flocs, deposited on the bed of the sea, have been consolidated under their own load and the load of subsequent depositions.

The clay constituents of soils are derived principally from minerals which have flat scale-like crystals. Regardless of the minimum size to which a given mineral is divided, the particle will maintain the characteristic shape of the crystal. The physical properties of clay and its behavior under stress are largely determined by the shape and arrangement of the individual particles in the soil mass, and the pore water contained in the void spaces.

The flat scale-like shape of the grains imparts to the clay its property of cohesion and plasticity. The arrangement of those particles in flocs and the flocs in the integrated structure impart the property of compressibility and elasticity. The size of the grains and the arrangement determine the property of permeability and rate of compression if saturated.

The Permian clay under consideration is a marine clay that was deposited on the bed of an inland sea which covered a good portion of western Oklahoma and extended into western Texas and parts of western Kansas. The extent of the area covered by the sea is not well defined. It is thought that the sea was surrounded by a system of steep mountains and hills which have since been removed by erosion. The existence of extensive gypsum deposits and salt beds substantiates the theory that the sea dried up or nearly dried up three times during the history of that period.

This clay deposited in the Permian period differs considerably from most marine clays in that it includes a high percentage of soil particles of the find sand sizes. This inclusion of large percentages of sand sizes in the deposit is thought to be the result of relatively short and steep drainage systems into the area.

Although the major drainage systems of the area are well entrenched in the deposit and in many cases have cut entirely through the deposit, the tributaries have shifted considerably over the area. The shifting of the tributaties and resultant erosion along with a considerable amount of resedimentation has contributed much to the variations one encounters in the properties of the clay over the area.

The climate of the area has a marked influence upon the properties of the clay. The annual rainfall varies from about thirty-three inches down to less than eighteen inches over the area. The greater portion of the rain falls on the area in two of the spring months, with rather poor distribution of the balance over the year. The poor distribution of rainfall over the year permits seasonal drying out of the soil to appreciable depths. The depth of the zone of drying has been found to exceed fourteen feet at Stillwater, Oklahoma. At points in west Texas

Terzaghi[10] reports the depth of drying to exceed twenty feet.

The Permian clay may be described as follows: When wet its color varies from dark red to dark reddish brown, and when dry from a light yellowish brown to light red. It is quite often mottled gray or pale green. It quite often contains inclusions of the mineral Bentonite, a disintegrated volcanic ash. Also it may contain inclusions of highly cemented particles of sand and coarse silt. The particles range in size up to that of coarse gravel. Sand lenses of varying thickness are quite common in the deposit. In many cases the sand lenses have been indurated to rock.

In the undisturbed state the clay is hard and brittle. In many cases it shows definite jointing planes. It is tough and very plastic when wet, and it has a very high dry strength. It has been over-consolidated, or precompressed, to a high degree by overburden which has been removed by erosion, and by desiccation.

The clay in the remolded state has liquid limits and plastic indices of medium value. It has a very high dry strength.

III. TESTS AND DISCUSSIONS

It seems advisable to present a brief discussion of the physical properties of the clay which were measured in this investigation. Therefore in this section the result of each test is preceded by a discussion of the properties.

A. DESCRIPTION OF SAMPLES.

The results of tests of four samples, designated as B-1, C-1, C-2, and C-3, are reported herein. B-1 is a sample of mottled red clay removed from a sewer excavation in the west limits of the city of Stillwater, Oklahoma. It was removed from a depth of approximately seven feet below the ground surface.

C-1 is a sample of clay taken from the new Student Union site (1948) on the campus of Oklahoma A. and M. College, Stillwater, Oklahoma. It is a highly compressed dark red clay or shale. When dry, it has hardness and strength comparable to a fine grained sandstone. However it disintegrates very rapidly when immersed in water. It contains pronounced jointing planes in the horizontal direction. It is hard and brittle even when moist. It was taken from an approximate depth of fourteen feet below the ground surface.

C-2 is a sample of clay removed from the footing excavation on the site of the new Power Plant (1948) on the campus of Oklahoma A. and M. College. It is a medium soft, but brittle, mottled yellowish-red clay. Upon drying it breaks up into small cubes of dimensions up to three-quarters of an inch. Its dry strength is much lower than that of sample C-1 but it is still classed as high, and is about comparable to that of sample B-1. It was taken at a depth of nine feet below natural ground surface.

C-3 is a sample of clay taken from the Student Union site at a depth of seven feet below the natural ground surface. Although it is from the same location as sample C-1, it differs in many respects from C-1, its color is about the same, but it differs considerably in texture. Its degree of precompression is markedly lower than sample C-1. It is softer than C-1, almost of equal brittleness, and of lower dry strength.

B. <u>PLASTICITY</u>.

Plasticity is the property of a material which permits it to be deformed rapidly, without volume change, without rupture, and without apparent elastic rebound. This property of plasticity is present to different degrees in all cohesive soils through some range of water content. The range of water contents through which a soil exhibits this property can be determined only on a remolded sample.

After a cohesive soil has been remolded, its consistency can be varied at will by increasing or decreasing its water content. If the water content of a soil is increased without limit, the mass will ultimately become a suspension, at which time the mass loses its resistance to shear forces. Then, if evaporation is permitted, the consistency of the mass will decrease; and at some water content the suspension will lose its liquid properties and the mass will regain a measurable shearing resistance. Upon the loss of the liquid properties the mass takes on the property of plasticity. The water content at which the mass loses its liquid properties is called the liquid limit. Further drying is accompanied by an increase in shearing resistance, and at some water content below the liquid limit the mass loses the property of plasticity and assumes the properties of a semi-solid, then a solid.

Atterberg,(1) a Swedish agriculturist, defined the limits of the plastic range. He defined plasticity of a cohesive soil in terms of the amount of sand that could be added to a cohesive soil, with consistency at the liquid limit, causing the soil to completely lose its plasticity. He suggested that the differences in the water content at the boundaries of the plastic range be a measure of the plasticity of the soil. The difference is called the plastic index.

The liquid limit is an arbitrary value of water content determined by a method described by Atterberg. Atterberg's method was subject to considerable personal error. In 1927 A. Casagrande developed a machine and method of testing which eliminates the principal personal errors. Casagrande's liquid limit device and procedure for determination of the liquid limit was adopted by the American Association of State Highway Officials, and is described in their Standard Specifications under serial designation T 89-42.

In either the Casagrande or Atterberg method of test, the true liquid limit is at a higher value of water content than is indicated by the tests. Both tests are dependent upon impact forces overcoming the shearing resistance of the material. Therefore, the water content must be below the true liquid limit in order that the soil possess shearing resistance. The foregoing discrepancy between liquid limit defined and liquid limit determined is of little importance in that the test itself is only indicative of soil behavior under stress.

Casagrande's method of test for the liquid limit of a soil reveals more than an arbitrary value of water content of the soil as it changes from the liquid state to the plastic state. If the number of blows in the Casagrande method are plotted on a logarithmic scale against the corresponding water content on an arithmetic scale, it is found that the plot of all points in the vicinity of the liquid limit is a straight line. The equation of this straight line, called the flow line, can be written as

$$w = w_1 - I_f \log N \qquad (1)$$

in which $\underline{w}$ and $\underline{N}$ are corresponding values of water content and number of blows for any point on the line, $\underline{w_1}$ is the intercept, numerically equal to the water content at one blow, or $\underline{N = 1}$, if $\underline{I_f}$ is the slope of the line numerically equal to the difference in water content across one cycle of logarithms.

It has been observed that two soils of the same plasticity vary in the amount of resistance offered to deformation at the same water content within the plastic range, i.e., the shearing resistance is different in the two soils at the same water content. It also has been observed that all cohesive soils have the same shearing resistance at the liquid limit. Since the liquid limit is determined by overcoming the shearing resistance of a soil by the impact forces produced in the mass by the device, the number of blows must be proportional to the shearing resistance, i.e., $\underline{N} = \underline{KS}$, where $\underline{N}$ equals the number of blows, $\underline{K}$ is a constant of proportionality, and $\underline{S}$ is the shearing resistance. If $\underline{w_L}$ and $\underline{KS_L}$ are substituted in the equation of the flow line, equation 1, the equation

$$w_L = w_1 - I_f \log KS_L \qquad (2)$$

results, wherein $\underline{w_L}$ is the water content at the liquid limit and $\underline{KS_L}$ is the number of blows at the liquid limit. If $\underline{w_P}$ and $\underline{KS_P}$ are substituted in the equation of the flow line, equation 1, the equation

$$w_P = w_1 - \log KS_P \qquad (3)$$

results, wherein $\underline{w_P}$ and $\underline{KS_P}$ are the corresponding values of the water content and number of blows at the plastic limit. The second substitution assumes the proportionality between the number of blows and shearing stress holds for all values of water content down to the plastic limit. If equation 3 is subtracted from equation 2 the equation

$$w_L - w_P = I_f \log \frac{S_P}{S_L} \qquad (4)$$

results. The left hand member of equation 4 is by definition the plastic index, $\underline{I_P}$. Substituting $\underline{I_P}$ for $(\underline{w_L} - \underline{w_P})$ and dividing both sides of the expression by I_f the following expression results:

$$\log \frac{S_P}{S_L} = \frac{I_P}{I_f} \qquad (5)$$

The shearing resistance of cohesive soils at the liquid limit has been determined to be a constant value of approximately twenty-seven grams per square centimeter. The shearing resistance at the plastic limit varies for different cohesive soils, and in general is not known. However, the logarithm of ratio of the shearing resistances at the liquid and plastic limits can be evaluated in terms of two measurable quantities, I_P and I_f. The value of the logarithm is a measure of the relative toughness of a soil, and has been called the toughness index, and designated I_T.

The value of I_T for most cohesive soils varies between the values of one and three,(5) with some very fat clays having a toughness index as high as five. A soil with a toughness index less than unity is said to be friable at the plastic limit. The measurement of plasticity of such soils is meaningless.

In general, a soil having a high liquid limit and a high plastic index is subject to large volume changes upon drying out, and is also subject to high compression when superimposed loads are applied. A soil with a relatively high toughness index indicates a high dry strength.(3)

A. Casagrande(3) uses the plasticity chart as a means of classifying cohesive soils into eight major groups, i.e., inorganic clays of low, medium, and high plasticity; inorganic silts of low, medium, and high compressibility; organic silts; and organic clays.

The results of measurements of plasticity of two samples of the Permian clay are shown in figure 1. The two samples, C-1 and C-2, represent the high and the low of all the tests that were made in this

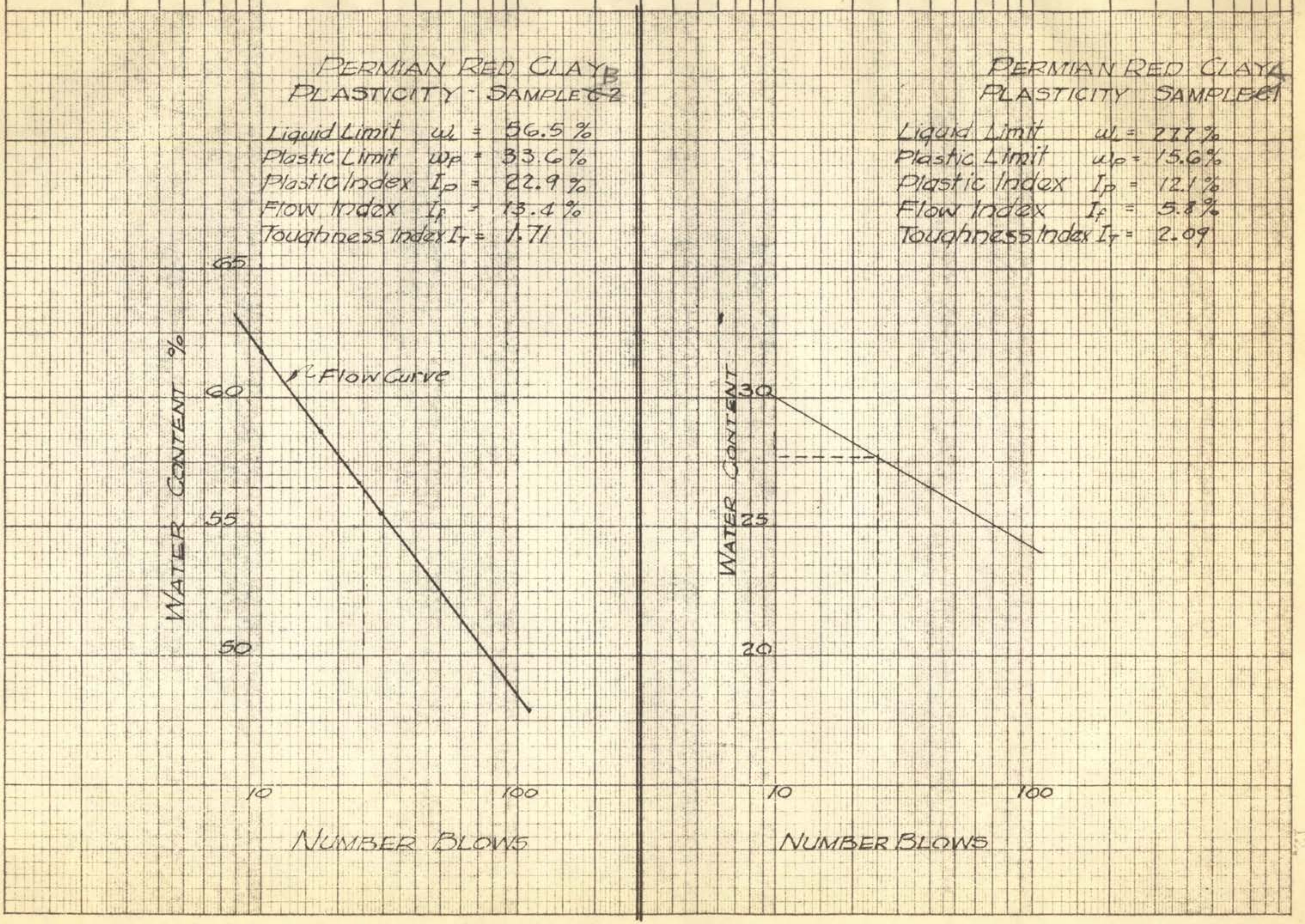

PERMIAN RED CLAY B
PLASTICITY - SAMPLE C-2
Liquid Limit w_L = 56.5 %
Plastic Limit w_P = 33.6 %
Plastic Index I_P = 22.9 %
Flow Index I_f = 13.4 %
Toughness Index I_T = 1.71
Flow Curve
WATER CONTENT %
65
60
55
50
10
100
NUMBER BLOWS
PERMIAN RED CLAY A
PLASTICITY SAMPLE C-1
Liquid Limit w_L = 27.7 %
Plastic Limit w_P = 15.6 %
Plastic Index I_P = 12.1 %
Flow Index I_f = 5.8 %
Toughness Index I_T = 2.09
WATER CONTENT
30
25
20
10
100
NUMBER BLOWS

Figure 1.

investigation. They do not represent the limits of plasticity for the Permian clays as a whole.

According to the plasticity chart, sample ~~C-1~~ A would be classified as an inorganic clay of low plasticity, and sample ~~C-2~~ B would be classified as an inorganic silt of high compressibility or an inorganic clay. However, the simultaneous values of plastic index and liquid limit for sample ~~C-2~~ B falls about on the ~~imperical~~ empirical division between the inorganic clays of high plasticity and the classification given above. It will be noted in figure 1 that both samples have medium toughness indices.

Figure 1 shows a reasonably wide variation in the plasticity characteristics of samples ~~C-1~~ A and ~~C-2~~ B. It is doubtful if either sample represents the extremes that exist in the Permian deposit as a whole.

C. <u>GRAIN SIZE AND GRAIN SIZE DISTRIBUTION.</u>

In fine grained soils a good portion of the entire mass is composed of grains smaller than .076 millimeters. The size of soil fractions smaller than .076 millimeters can be determined only by one of the methods employing the principle of continuous sedimentation.

The principle of continuous sedimentation is based upon the validity of Stokes Law. Stokes Law expresses the rate at which a spherical body will settle out of suspension in a medium of known fluid properties.

The soil grains which constitute the fine fraction of a soil are not spherical in shape, but have shapes dependent upon the crystalline structure of the minerals from which they were derived. Further the soil grains are not all of the same density, a factor which influences the settling rate of each particle. Therefore grain size as determined for clay soils in general is almost meaningless. However, in a given area a sufficient quantity of grain size analyses may lead to a method of grouping soil types.

Numerous attempts have been made to classify soil by grain size. One of the several used size classifications is shown on figure 2.

In figure 2 it will be observed that the clay consists of three size fractions, very fine sand, silt, and clay.

The presence of the sand fraction which constitutes as much as twenty percent by weight of the total mass, imparts to the clay a gritty feeling and appearance when moist or dry. The gritty appearance and feel are lost when the clay becomes saturated. Silt sizes constitute about fifty percent of the total mass, with the clay sizes representing from thirty-five to fifty percent of the total mass.

It will be observed in figure 2 that even though the two samples represented different size soils they are almost parallel in distribution of those sizes. It also will be noted that in either case eighty percent or more of the total soil is smaller than can be seen with the naked eye. The smallest size distinguishable with the naked eye is approximately .06 millimeters.

D. <u>PERMEABILITY</u>.

A material which has continuous void spaces is said to be permeable. Since a soil mass is made up of solid mineral grains surrounded by connected spaces, it exhibits the property of permeability.

If a fluid percolates through a permeable material under conditions of pressure such that there is no change in the volume of the void space, the discharge velocity of the fluid is given by the equation(10) ~~$~~

$$v = \frac{K}{n} i_p \qquad (\overset{6}{1})$$

wherein v is the discharge velocity in centimeters per second, n is the

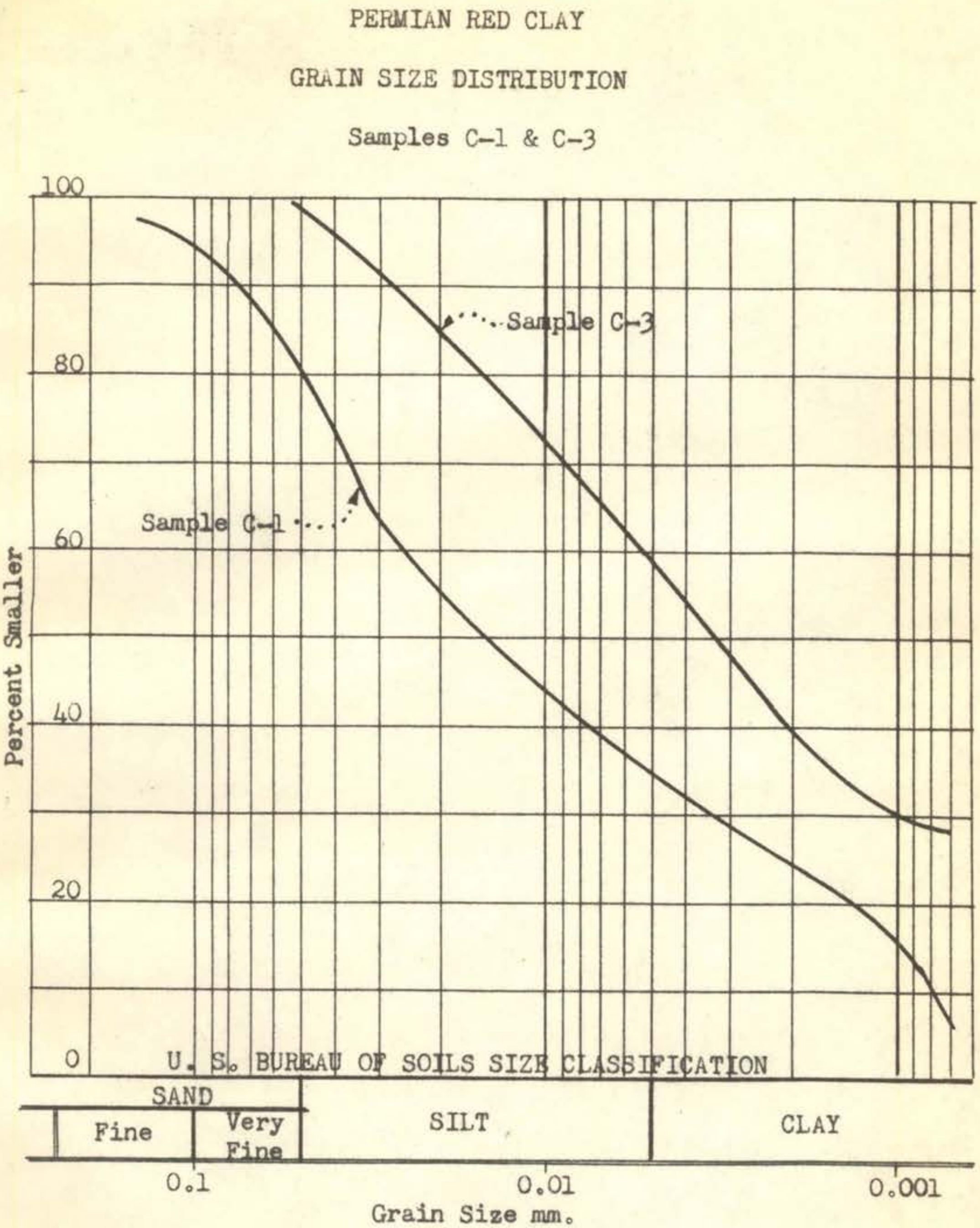

PERMIAN RED CLAY
GRAIN SIZE DISTRIBUTION
Samples C-1 & C-3
Sample C-3
Sample C-1
Percent Smaller
100
80
60
40
20
0
U. S. BUREAU OF SOILS SIZE CLASSIFICATION
SAND
Fine
Very Fine
SILT
CLAY
0.1
0.01
0.001
Grain Size mm.

Figure 2.

dynamic viscosity in gram-seconds per square centimeter, i_p is the excess hydrostatic pressure per unit of length producing the flow, and K is an imperical constant called the permeability of the material. The units of K are square centimeters. The permeability number K represents the permeability characteristics of the material and is independent of the properties of the fluid flowing.

In civil engineering practice permeability as a rule is not used directly when dealing with flow of water through soil, but the coefficient of permeability k, given by the equation

$$k = K \frac{w}{n} , \qquad (2)$$

is used. In equation 2 K is the permeability number, w is the unit weight of water, and n is the dynamic viscosity. If one works in the same system of units as set out in equation 1, the units of k are centimeters per second. It should be noted that k is dependent upon the properties of the fluid.

Darcy's Law for the flow of water through soils states that the quantity of water that will flow through a given cross section of soil at a given time under the influence of a given hydraulic gradient is proportional to the product of the area, the hydraulic gradient, and the time. When the relationship is expressed algebraically the equation becomes:

$$Q = kiAt \qquad (3)$$

where Q is the total quantity of water, i is the hydraulic gradient, A is the area, t is the time of flow, and k is the constant of proportionality. If both members of equation 3 are divided by A and t, it reduces to

$$v = ki. \qquad (4)$$

Since $\underline{i}$ is a dimensionless quantity, $\underline{k}$ has the dimensions of velocity and must be the coefficient of permeability as defined in equation 2.

A. Casagrande(5) shows that Darcy's Law is valid only when the flow is laminar. In most cases the flow through fine grained soils is laminar, therefore Darcy's Law has been used as the basis for developing the theories of flow of water through soils. footnote

The property of permeability is quite important to the civil engineer in the solution of problems encountered in dewatering excavations, in the design of earth dams, in development of ground water supplies, in the design of subsurface drainage, and in computing the probable time of settlement of clay layers.

The value of the coefficient of permeability of the Permian clays seems to be in the range 1×10^{-9} to 10×10^{-9} centimeters per second.

The permeability of a soil bears a definite relationship to the void ratio. Casagrande shows the relationship to be approximately parabolic.

The coefficient of permeability of the Permian clays was computed from the results of a limited number of consolidation tests. In one test the coefficient was computed for each consolidating pressure. The assumption was made in the computations that the coefficient of consolidation remained constant for all loads, an assumption that is not strictly true. Based on that assumption it was found that the relationship between the coefficient of permeability and the consolidating pressure can be expressed almost exactly by the equation

$$k = k_o - m \log \frac{p}{p_o} \qquad (5)$$

wherein $\underline{k}$ is the coefficient of permeability when the pressure in the soil is any value $\underline{p}$, $\underline{k_o}$ and $\underline{p_o}$ are known simultaneous values of the coefficient and pressure, and m is the slope of the curve on a semi-log plot.

One should note that equation 5 is based on so little experimental information that it cannot be taken as a reliable relationship. It is presented here only to indicate the possible relationship between the variables. However, if a sufficient number of tests were run to substantiate the reliability of the equation it could prove quite useful in the solution of many seepage problems.

E. SHRINKAGE.

The decrease in volume of a soil accompanying desiccation is called shrinkage, and the increase in volume as the soil takes up water is called swelling. These properties are present in sufficient magnitude to be of importance only in cohesive soils.

In order to describe the shrinkage process, it seems advisable to first discuss briefly capillary action.

Consider a thin elastic membrane stretched into a plane surface and submerged in a fluid which is at rest. From the principles of hydrostatics we know that the fluid pressure at any point of the membrane will remain plane after being submerged. If it were possible to increase the pressure on one surface, one would observe that the surfaces of the membrane would be transformed into curved surfaces. The curvature produced is undoubtedly the result of a pressure deficiency on the convex surface, in equilibrium with tangential stress existing in the curved surface.

The tangential stresses in the membrane are comparable to the surface tension of a fluid. It can be shown that for a condition of equilibrium the equation expressing the relationship between the pressure deficiency on the convex surface and the surface tension is

$$p - u = T_s \frac{R_1 + R_2}{R_1 R_2} \qquad (1)$$

wherein p is the pressure on the concave surface, u is the pressure on the convex surface, T_s is the intensity of surface tension, and R_1 and R_2 are the radii of two points in the surface. The left hand member of equation 1 ($p - u$) becomes the pressure deficiency on the convex surface. If the surface formed is that of a hemisphere, the radii of all points on the surface are equal, and the equation becomes

$$p - u = \frac{2T_s}{R} \qquad (2)$$

If one observes the surface of water in a small diameter clean glass tube it is seen that the surface of the liquid is hemispherical in shape. Therefore, the pressure above and below the surface film must differ by ~~an~~ a small amount equal to $\frac{2T_s}{R}$, when R is the radius of the tube. If atmospheric pressure is taken as zero it is seen that the pressure on the convex surface is negative or less than atmospheric, i.e., the water is stressed in tension.

Normally water is considered to be a substance which will take only moderate values of tensile stress, i.e., up to about one atmosphere. This weakness of tension of water as it is generally encountered is due to the growth of air bubbles which sever the water column.

If water contained in a small diameter clean glass tube has a fully developed meniscus exposed to atmospheric pressure, the tension developed in the water is inversely proportional to the radius of the tube. If there were no air in the water, it is evident that almost unlimited tension could be produced. Casagrande[5] shows that almost unlimited tension can be developed in water containing air if two conditions pertaining to the air bubbles are satisfied. The first condition is that the diameter of the air bubbles shall be equal in diameter or less than the diameter of the tube after expansion, and the second is that the internal pressure shall be equal to or less than atmospheric after expansion.

Consider now a capillary tube of uniform cross section placed in the horizontal position completely filled with water. The end surfaces will be plane. Therefore, the stress in the water will be equal to atmospheric pressure or zero. As evaporation proceeds on the surfaces, the volume of water is reduced and the surfaces attempt to recede within the tube. Adhesive forces between the molecules of water and the molecules of glass in the walls of the tube try to prevent the line of contact from moving. At the same time cohesive forces between molecules of water in the surface film try to prevent distension of that surface. Under the influence of those two sets of opposing forces a distended surface of minimum area is formed as evaporation of the water progresses. The reactions produced on the walls of the tube by both the tension in the water and the force of the meniscus attempt to shorten the tube and to reduce it in diameter.

Consider now a cohesive soil mass made up of solid grains and void spaces. The void spaces are all interconnected by pore passages of capillary sizes. Figure 3 ~~might~~ represents the soil grains found in the surface of such a soil mass, magnified many times.

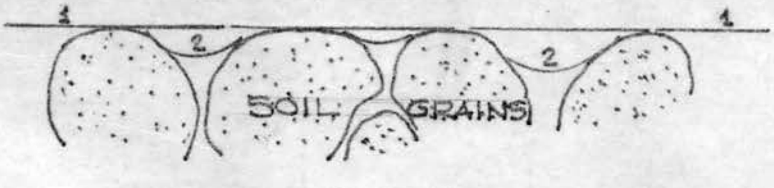


Figure 3.

Assume the pore spaces in figure 3 to be completely filled with water. When free water exists on the surface, the meniscus is a plane surface represented by line 1. If we disregard gravitational forces the stress in the water throughout the voids is zero. At some time later evaporation has reduced the water surface to point 2, and the meniscus becomes curved and is stressed in tension. The tensile stress in the water causes all the soil grains to move closer together. At the same time tension induces a flow of water from the interior of the mass to the surface where it is lost by evaporation. This process might continue for quite some time without any apparent change in the water surface. As the soil grains move closer together the pore diameters are reduced, effecting a reduction in the radius of the meniscus. The reduction in the radius of the meniscus effects an increase of tension in the pore water which is required to offset a continually increasing resistance of the mass as a whole to deformation.

The maximum compression of the mass, the maximum tension in the pore water, the minimum pore diameter, and the meniscus of minimum radius are developed almost simultaneously near the surface of the mass. At that time the change in volume concurrent with desiccation ceases, and the material is said to have reached the shrinkage limit. At the shrinkage limit the meniscus begins to recede within the mass and final compression is produced. Thus the soil passes from the semi-solid to the solid state.

The transition from the semi-solid to the solid state is evidenced by an apparent change in color of the material. As the menisci recede from the surface the material becomes lighter in color, and the surface loses its glossy appearance.

One will ask how a soil mass can be saturated and the individual grains be separated by water films and still retain its stability. The answer lies in the surface activity of the soil grains.

The surface of all soil grains is the seat of a negative charge of electricity. The water that fills the voids of the soil mass, being a bi-polar substance, is oriented by the charge on the surface of the particles. The depth of this orientation extends an appreciable distance away from the surface of the particle. The forces exerted in the orientation of the water molecules are large relative to the mass of the molecule, and as a result the water in the zone of orientation loses the characteristics of normal water and assumes the properties of a viscous liquid with increasing stiffness in the direction of the particle.

The molecules of water in contact with the soil grain exhibit the properties of a ductile solid. The layer of oriented water molecules surrounding each individual grain is called the adsorbed layer. That portion of the adsorbed layer consisting of the semi-solid and solid water seems to be approximately .005 microns in thickness.(10) The influence of the orientation may extend a distance of about 0.1 micron from the surface, i.e., the water does not assume the properties of normal water until that distance away from the particle is reached. The thickness of the adsorbed layer seems to be independent of the size or shape of the grain, but it is quite dependent upon the chemical composition of the adsorbed layer and the mineralogical composition of the grain.

If the above is true, it is evident that the percent of the total mass of a given soil occupied by the adsorbed layer increases with decreasing grain sizes. The flat scale-like particles which constitute the major portion of the very fine fraction of a soil, i.e., sizes less than two microns, have dimensions in the plane of the flat surface many times greater than in the plane perpendicular to it. Therefore a preponderence of molecules of water are oriented in one direction relative to a given soil grain, and the resultant force produced by the orientation is many times greater in magnitude than the gravitational forces acting on the particle. When water is added to a soil mass, the individual particles will be separated as water is taken into the zone of influence of the adsorbed layer. The separation of particles will produce an increase in volume of the mass, and the mass is said to have swelled. The amount of swelling in the vicinity of a single grain bears an inverse relationship to the depth below the surface of the deposit in which the grain is located. The resultant force due to the orientation of water molecules about a given soil grain is constant; while the resistance offered to the relative movement between the soil grains produced by the resultant is the integrated effect of gravitational forces on all the grains located above the grain under consideration.

Omit

The total shrinkage of a clay soil is the result of two effects. The first is the volume decrease due to relative movement between soil grains due to loss of water in the adsorbed layer. The second effect is the deflection of the minute arches built up of soil grains in the formation of the floculent structure. The latter probably contributes more to the total volume change than does the first.

Omit

Three swelling tests were run on sample C-2. The samples were air dried to constant volume, then fit into a consolidation ring. The samples were loaded into the consolidation device with different loads. After complete compression occurred in the dry state the samples were flooded and allowed to swell as they became saturated. Dial readings were taken during the period of saturation. When swelling was complete the samples were removed and their water contents checked in order to determine the degree of saturation.

The amount of swelling was computed in inches per foot of height for each of the three tests. Figure 4 is a logarithmic plot of the swelling values in inches per foot plotted against pressure in tons per square foot.

If one observes the curves it will be noted that the relationship between the swelling value and pressure is almost that of a rectangular hyperbola. There were not enough values taken in the investigation to justify an exact determination of the geometric relationship between the two variables.

F. COMPRESSIBILITY.

When a soil mass is compressed by the action of applied loads the mass undergoes a change in volume. For ordinary loads the change in volume is due to a reduction in volume of the void fraction of the entire mass; since only a very small reduction in volume, if any, is produced in the solid fraction, it is neglected. If it is assumed that all the volume change occurs in the void fraction, a relationship between void ratio, e, and vertical strain can be derived.

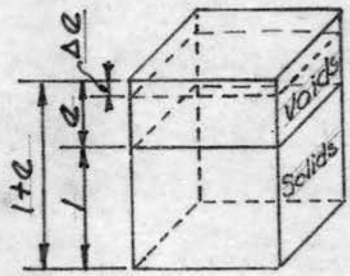
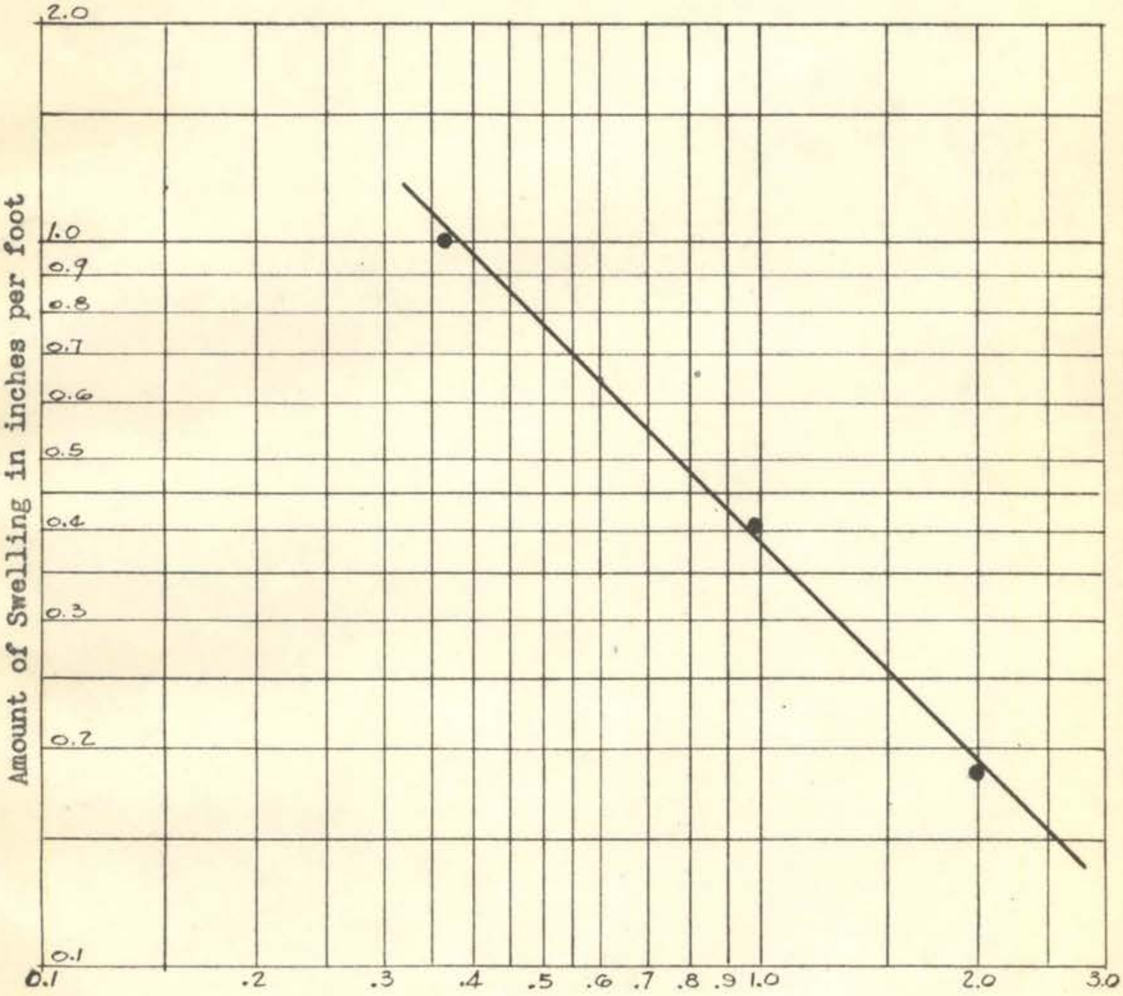


Figure ~~5~~ 2.

PERMIAN RED CLAY

SWELLING CHARACTERISTICS
UNDER VARIED LOADS

Sample C-2



Load in Tons per Square Foot

Figure 4

Let figure ~~5~~ 2 represent a volume of soil of void ratio e, chosen such that the volume of solids is unity. The total volume then will be equal to the volume of voids plus the volume of solids. Void ratio by definition is equal to the volume of voids divided by the volume of solids; therefore, as the volume of solids has been chosen as unity, the volume of voids is e and the total volume is ($1 + e$) plus. Assume that a load produces a decrease in volume equal to $\Delta V = \Delta e$, and that the volume has a horizontal cross-sectional area A, and is only one vertical element of an infinite number of such elements located under an area of infinite extent in the horizontal direction. Also, assume this area of infinite extent to be loaded with a uniform load. It is evident that the vertical deflection of all points in the same horizontal plane will be equal, and that the deformation produced in the elemental volume is one of vertical deformation only. Therefore the vertical strain is

$$\frac{\frac{\Delta e}{A}}{\frac{1 + e}{A}} = \frac{\Delta e}{1 + e} . \qquad (1)$$

Professor K. Terzaghi found by experimenting with clays that if a given sample was transformed into a thick paste by the addition of water and subsequently consolidated under an increasing load, the resultant plot of the corresponding values of void ratio and pressure to a semi-logarithmic scale was a straight line. The equation expressing the relationship could be written as

$$e = e_o - C_c \log \frac{p}{p_o} \qquad (2)$$

wherein e is any void ratio corresponding to pressure p, e_o and p_o are known simultaneous values of the void ratio and pressure, and C_c is the slope of the line equal numerically to the difference in void ratio across one cycle of the logarithmic scale. A clay which has been consolidated only under the load of its present overburden is called a normally loaded or normally consolidated clay. If a normally consolidated clay is further consolidated by increasing loads, it has been observed that the (e - log p) plot of field measurements is a straight line. Laboratory curves of the (e - log p) values for these clays are not straight lines through the entire range of loading.

The (e - log p) plot for all values of p up to and slightly in excess of the pressure corresponding to the overburden pressure is curved, beyond which the plot becomes straight and remains straight for all greater pressures. The curved portion of such a plot is actually a recompression curve, for when a so-called undisturbed sample is removed the pressure producing consolidation in the field has been relieved.

When a consolidation test (confined compression test) is run on a normally loaded clay in the laboratory, and several cycles of loading and unloading are carried out with the maximum load in each cycle being greater than in the preceding cycle; it is observed that the recompression portion of the curve in each cycle extends to a value of pressure a little in excess of the maximum in the preceding cycle. It is also observed that the straight portions of the (e - log p) plot are a series of straight parallel lines with each successive plot occupying a slightly lower position. The difference in ordinates between any two successive straight lines at a given pressure represents the permanent set produced in the material by the compression.

Sample B-1 was allowed to air dry to a water content below the shrinkage limit prior to its preparation for testing. A confined compression test was made on the material. The semi-log plot of the relationship existing between the pressure and void ratio is shown in

figure 6.

The (e - log p) plot for sample ~~B~~A-1 (figure 6) is typical in most respects of all samples, except one, which have been tested in this investigation. It will be noted that the plot is curved throughout the entire range of loading in the first cycle. The maximum load in the first cycle was approximately twenty-four tons per square foot. This indicates that the clay has been consolidated at some time in its geological history by a pressure equivalent to approximately four hundred feet of overburden. The geologists tell us that two hundred to three hundred feet of material has been removed from this area by erosion.

A definite change in curvature is noticeable on all the (e - log p) curves at pressures of three to four and a half tons per square foot. It is thought that this change in curvature is due to reconsolidation of the material many times by desiccation. A pressure of 3.6 kilograms per square centimeter was required to reduce the void ratio of sample B-1 to the same value that it had before swelling. There is an apparent change in curvature in the vicinity of that load.

Figure 6 indicates that a uniform clay deposit represented by sample ~~B~~A-1 if dried below the shrinkage limit to a depth of fifteen feet would swell when saturated, causing a rise in its surface of approximately three and a half inches. The swelling strain would be 0.23 inches per foot. Figure 7 is the (e - log p) plot of the results of a consolidation test on sample ~~C~~B-2. The sample was air dried to a water content below the shrinkage limit prior to testing. It was allowed to saturate and swell under a load approximately equal to its present overburden load, then it was consolidated under increasing loads up to a loading value of twenty-four tons per square foot.

PERMIAN RED CLAY

e-log p Curve

Sample B-1

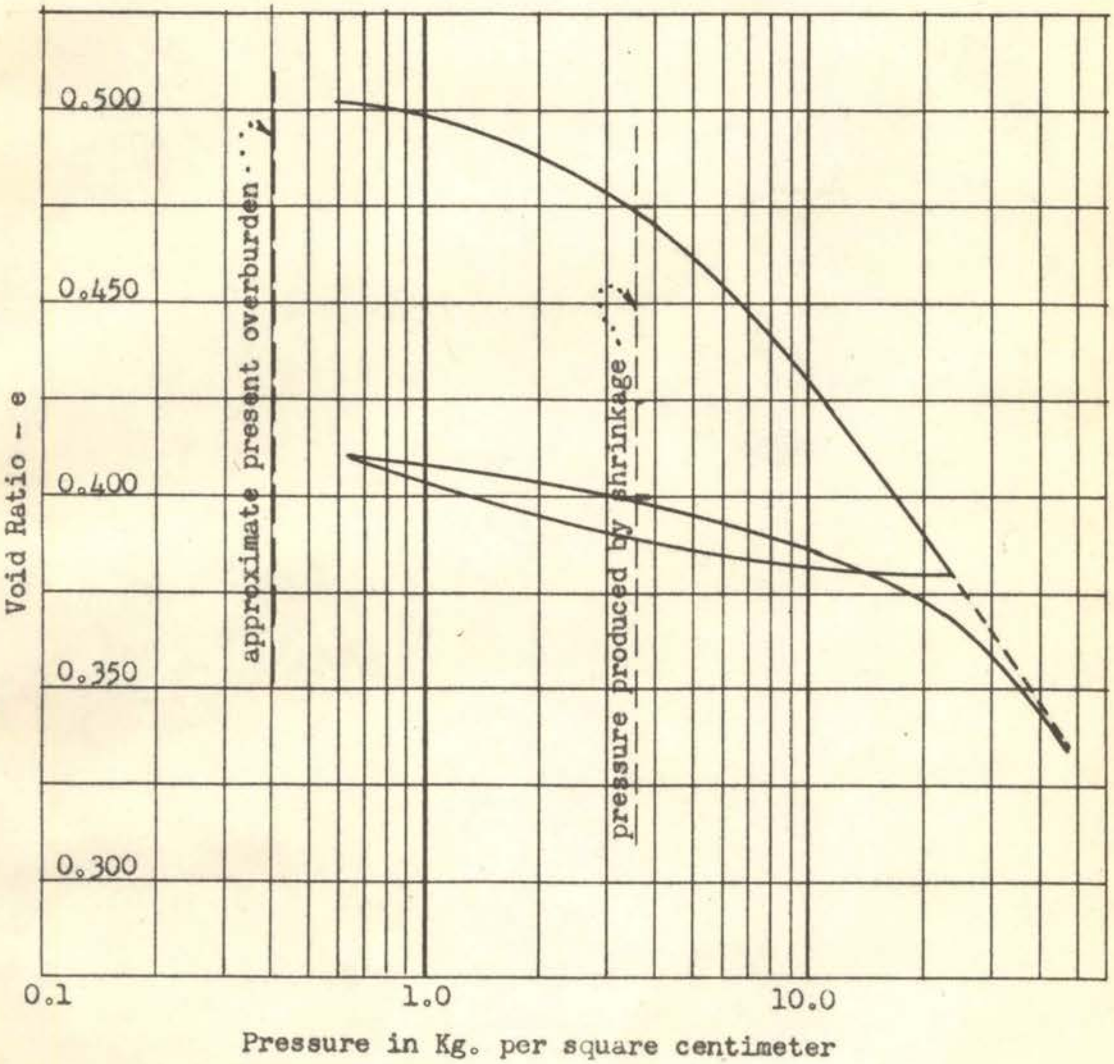


Figure 6

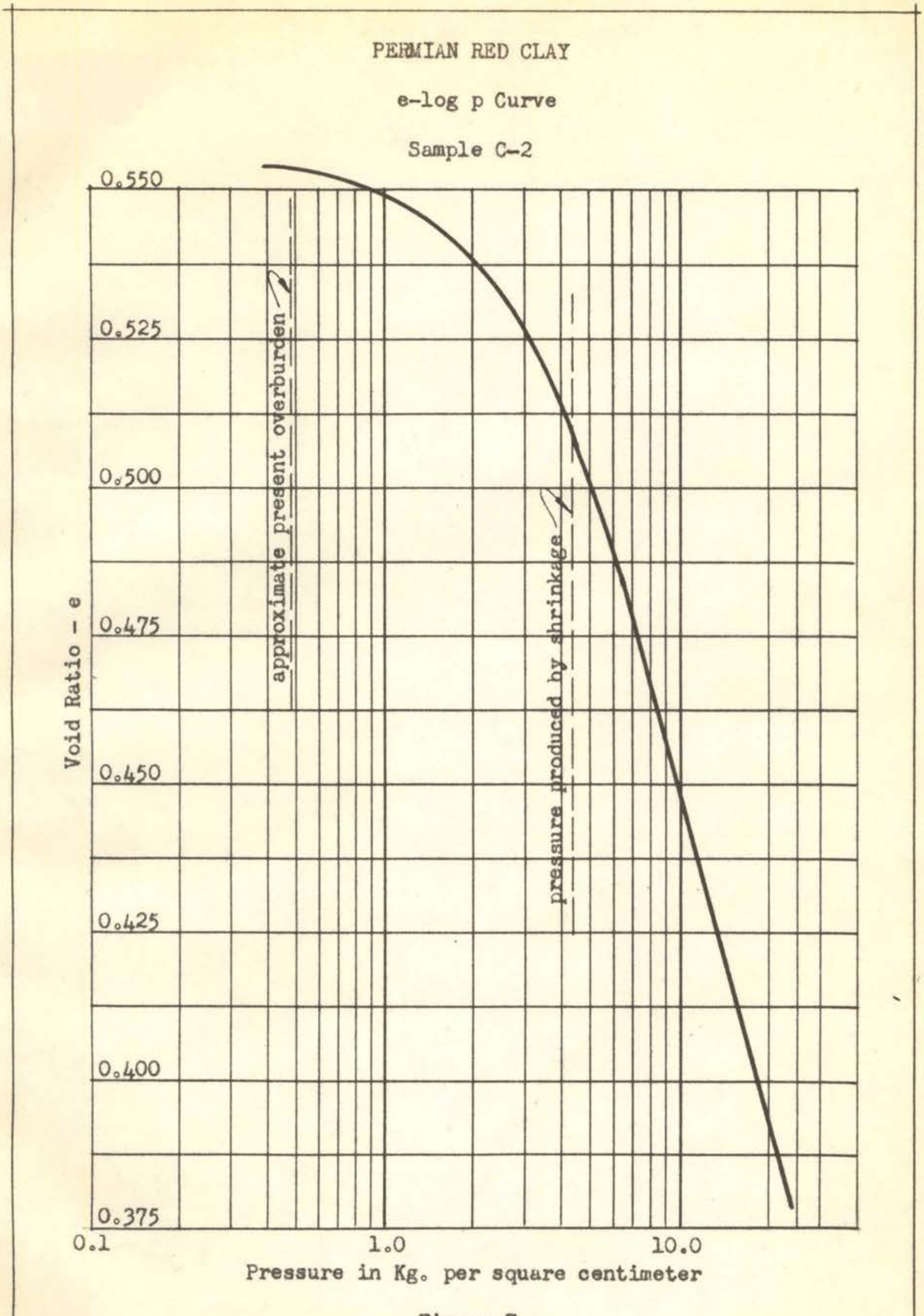

PERMIAN RED CLAY
e-log p Curve
Sample C-2
0.550
0.525
0.500
0.475
0.450
0.425
0.400
0.375
Void Ratio - e
0.1
1.0
10.0
Pressure in Kg. per square centimeter
approximate present overburden
pressure produced by shrinkage

Figure 7

The consolidation curve of sample ~~C~~B-2 differs from the curve for ~~B~~A-1 in that it becomes a straight line for pressures in excess of about six tons per square foot. If one considers the shrinkage forces to be the preconsolidation loads, the curve resembles that for a normally consolidated clay. It should be kept in mind that sample ~~C~~B-2 was taken from a depth that is well within the zone of drying, therefore the material has been subject to consolidation by desiccation many times.

The straightening out of this curve indicates that the material represented is probably the result of resedimentation in that particular area. Its compressibility is much greater than that of the other samples tested. The void ratio of this material at the shrinkage limit was 0.510 compared with sample B-1 of 0.473 and several samples taken from the Student Union Building site approximately three-eights mile south which varied between 0.392 and 0.276.

When one studies the results of the consolidation tests (confined compression tests) made on these clays, several observations can be made. For ordinary values of contact pressure used in the design of foundations of structures to be supported by these clays, the compression produced is the result of consolidation through a portion of the recompressed range of the material. The values of the total deformation produced will be small and tolerable in most cases. If the clay is dry for a considerable depth, it appears that swelling could be prevented if sole pressures equal to the shrinkage pressures were used in designs of foundations. This is not true because of the distribution of pressure with depth.

The normal pressure produced on a horizontal plane below a loaded area is dependent upon the shape and size of the loaded area, the depth below the loaded area, and the intensity of vertical stress on the plane

Omit

of loading. If a soil mass is considered to be a uniform, isotropic, elastic mass of semi-infinite extent, we find the intensity of pressure of loading at a depth of 1.5 to 2.0 times the least dimensions of the loaded area. (incomplete)

The clay under consideration does not rigorously satisfy the assumptions made. However, the difference between the assumed ideal material and the actual clay is not great enough to invalidate analysis based upon the theory.

The average load placed on a footing requires a relatively small area to produce a sole pressure equal to the stresses produced by shrinkage. The stresses produced by the sole pressure diminish at a rapidly increasing rate with depth. On the other hand stresses produced by shrinkage are uniform with depth throughout the zone of drying. It is evident then that high values of sole pressures can be only slightly efficient in preventing movements of a strata due to variations in water content.

Omit

G. SHEAR.

In the analysis of problems such as the stability of slopes, earth pressures on vertical and inclined surfaces, carrying capacity of footings and piles, etc., the inherent resistance to shearing forces or combination of forces that might produce failure becomes important. Two ideal soils are chosen in the development of theories upon which methods of analysis for such problems are based.

One ideal soil is a clean cohesionless sand. The shearing resistance of such material is dependent only on the angle of friction and normal pressure between the soil grains. The other ideal material is a fat clay. The shearing resistance of a fat clay is due entirely to cohesion between the soil grains. Cohesion is dependent upon the adsorption complex, and the amount of pore water in the soil, and it is independent of pressure.

The Permian clays fall somewhere between the two ideal soils. Their inherent shearing resistance is due both to cohesion and friction.

The specific values of cohesion and the angle of friction are measured by the triaxial compression test on cylindrical specimen.

In this investigation considerable difficulty was encountered in the preparation of the specimen for testing. The natural water content of the samples tested was very low. The material contained particles of cemented sand and silt up to one inch in their major dimension. The clay being hard and brittle and the presence of the cemented particles made it almost impossible to cut the cylindrical specimen to the required diameter. The permeability of the clay is so low that approximately two weeks is required to saturate a speciman ten centimeters high and ten square centimeters in area.

The results of the tests made in this investigation were so erratic that they will not be reported here.

The experience in this investigation for this particular test indicates that only field saturated samples should be taken, and so handled after sampling as to preserve the field moisture. Thus the specimen could be cut to form with much less difficulty, and one would be assured a higher degree of saturation at the time of the test.

IV. CONCLUSIONS

The properties of permeability, grain size, and plasticity are at most only indicative of the behavior of a soil under stress. The properties of shear, compressibility, and swelling are direct measurements of the soil behavior under stress.

This investigation indicates that for most loading conditions the property of compressibility of the Permian clays is of little importance. The additional stress induced in the clay layer by the load of ordinary structures is in general so small in comparison to the preconsolidation loads, that the settlements effected by consolidation are negligible.

The value of shearing resistance is important only in designs involving heavy surface loads on relatively small areas. If the area loaded is large or is located any appreciable distance below the ground surface the carrying capacity of the clay, a function of shearing resistance, is in general not a problem of concern. In the solution of problems involving earth pressures the shear properties assume considerable importance.

Of all the properties included in this investigation, the properties of swelling and shrinking of the clay effected by variations in water content ~~seem to be~~ are the most important. An inestimable amount of damage results from differential movement between parts of the structures caused by non-uniform variation in the water content of the clay layer below the structure.

Because of the wide variations in the physical properties of Permian clay deposits, as is the case in all soil deposits, it is doubtful if any amount of research would lead to specific evaluations of the properties. However, the author believes, as a result of this investigation, that ~~a~~ reasonable amount of statistical testing might reveal some general relationship ~~that~~ exists between the amount of volume change accompanying variations in water content, the plastic properties of the clay, and the loads on the clay. End Print the rest

The cost of an investigation to determine specific values of the soil properties pertinent to design can be absorbed in the cost of large structures, but that is not the case in the design of small structures. Since small structures represent the greater portion of all engineering construction in numbers built and a good percentage of the total cost, the author feels that general information about the behavior of the Permian clay can be of great value to the architect, the civil engineer, and the constructor. He also feels that cost of research to bring together that information can be justified, and that the research should be carried on.

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