LABORATORY INVESTIGATION OF SWELLING CHARACTERISTICS OF A REMOLDED PERMIAN CLAY

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LABORATORY INVESTIGATION OF SWELLING

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PERMIAN CLAY

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PREFACE

Soil Mechanics is a relatively new addition to the art of engineering. In the past, structural failures were eventually attributed to the use of faulty construction materials. Some fifty years ago the importance of the behavior of the underlying soils was realized. Since then soil mechanics was given a place among the various branches of engineering.

In soil problems the high indeterminacy of the variables involved makes mandatory an extensive investigation of the physical properties of the soil.

Due to an enormous amount of damage caused by volume change in the Permian clay which exists over much of the state of Oklahoma, the Engineering Research and Experiment Station of the Oklahoma Institute of Technology in cooperation with the Departments of Civil Engineering and Architecture have been conducting research programs for determining the properties of this clay under various conditions. This thesis incorporates the results of a research project conducted on remolded clays.

Of the factors entering into a problem dealing with the use of this clay in a disturbed state, the following were considered in this study: water content, superimposed load, and compactive effort. How these three factors affect the volume changes of this clay was the object of this investigation.

At first how these three factors affect fine grained materials in general, and clays in particular, and their relative importance is explained. The various processes through which these factors account for the swelling of clays are given.

Twenty-nine remolded samples of this clay were tested by the writer in the soil mechanics laboratory of the Oklahoma Institute of Technology. On the basis of the performance of these samples laboratory results are presented to show the interrelationship between these variables, and how each one of them quantitatively contributes toward swelling. Specific suggestions are included for more effective control of the expansion of this clay.

It is a pleasure to acknowledge the counsel, guidance, and co-operation of Professor R. E. Means, that made this presentation possible. Thanks also are due to Professor R. L. Flanders, Head of the school of Civil Engineering, for providing space for this investigation; to Dr. M. Herrin for his suggestions on the final reading of the manuscript; and to Dr. C. A. Dunn, executive director of the Engineering Research and Experiment Station, who was instrumental in securing funds for the research project.

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INTRODUCTION

General

In designing most structures the dimensions of the members and the strength and stress-strain relationship or modulus of elasticity of the material to be used are the primary and most essential elements governing the design. This is the case when steel, wood, and concrete are used as structural materials. Because of the relative rigidity of these materials, the deformation of the members in most cases is not a deciding factor in the design. In the case of soils, however, deformation is often the principal factor controlling the design of a structure. The deformation of most soils is accompanied by volume change of the soil.

Due to the variety of conditions under which soils can be found in nature, an exact analysis of the stress distribution in a soil mass presents a complex problem. Most soils are three-phase systems; i.e., they are composed of solid matter, water, and air. Their behavior under stress is greatly affected by their density and by the relative proportions of water and air filling their voids. These properties may vary with time and depend to a certain extent on a number of other factors. The time element becomes very important in the study of the stress-strain relationships of soils. The rate and manner of application of loads, moisture content of soils, vibrations and changes in the condition of lateral confinement of the soil are some of the factors variable in the case of soils.

The rate at which volume changes occur depends on the nature of the material, while the magnitude of these changes depends upon the overburden pressures and other variables.

Cohesionless soils, like sands and silts, are found in nature in different states. Loose sands occupy space at a high void ratio with a resultant high permeability. The weight of their relatively larger average grains causes them to settle as individual particles forming a single-grained structure. This structure might withstand the weight of any eventual overburden. With the slightest vibration the grains will tend to roll down sliding close to each other, producing a decrease in the volume and causing rather large settlements. Dense sands, on the other hand, increase in volume when deformed. Because of the way the grains in a dense sand are nested in the hollows between grains, they provide a stable support for any imposed load so long as they are laterally confined.

Shearing strength is proportional to the void ratio in cohesionless materials. The closer to each other the soil grains are the more frictional resistance they will develop against deformation under a given normal stress.

When dealing with sands and silts the primary concern is to ascertain the density of the deposits. The most satisfactory way to render them dense is by vibration. After they are properly compacted to a dense state, any superstructure will not increase their volume.

Because of their relatively high permeability, sands generally provide adequate channels for the flow of water through their mass.

Consequently, any fluctuations in climatic conditions after they are compacted to the optimum will cause no shrinkage or swelling of the mass. In the design of structures involving dense cohesionless soils, therefore, deformation due to volume change is not a problem.

Cohesive soils, like clays, are composed of very fine grains, their sizes being smaller than 0.002 mm. in diameter. Crystallographic studies revealed that most of these very fine grained soils are composed of flat scalelike particles. When these thin platy crystals are deposited at random they produce a structure with high void ratio forming soils which are compressible under static loads. Thin edges in contact with the broad surfaces tend to adjust their position to some extent under load. The broad surfaces of the adjacent crystals tend to come to rest in a more or less parallel position. But the elastic particles are deformed and bent by the load. On removal of the load the thin plates are sufficiently elastic to try to straighten out after they are bent, allowing an elastic rebound.

The movement between the crystals of the cohesive soils resembles the movement among the grains of sand, except for this partial elastic rebound which does not occur in sands.

As a result of the small size of their particles clays have a very low permeability. When a fully saturated mass of clay is loaded, at first all the stress is taken by the water. The soil grains will start sharing the load stresses with the pore water immediately after some of the water finds its way out through the pores of the soil. A considerable time will elapse for this squeezing out of the pore water, depending upon the permeability of the material. A gradual decrease in volume of the mass corresponding to the volume of the drained water will be experienced. This gradual process of decrease in volume due to squeezing out pore water by overburden pressures is called consolidation. Decrease in volume due to evaporation of the pore water and the resulting capillary pressure will be referred to in this discussion as shrinkage. Swelling will result in an expansive clay with the admission of free water through its pores. Swelling, in essence, is the opposite of shrinkage. The only difference indicated by tests is that the time for swelling is somewhat longer than the time for consolidation, butthe same relationship between time and degree of volume change holds for both shrinking and swelling.

Volume changes greatly depend upon the previous history of the soil also. The compressibility and expansibility of the soil are functions of the previous overburden and the degree of consolidation that has been achieved under this overburden. This degree of consolidation depends upon the time the overburden has been acting on the soil, with the permeability of the soil a determining factor.

When a soil is compacted with a standard compactive effort it will attain highest dry density at a water content called the optimum moisture content for this compactive effort. At this highest dry density the permeability is bound to be the least, considering that the mass of soil has its minimum void ratio. The concensus of opinion seems to be that the least volume changes, if any, in clays due to changes in water content should be experienced when the clays are compacted at the optimum.

Though compacted at optimum moisture some clays indicated a tendency for expansion. The above, then, seemingly justified assumption failed by field observations. This condition prompted investigation as to the extent of the swelling of a clay under different loading conditions, at different moisture contents, and different compactive efforts. There must be a water content at which a mass of clay of a certain density will experience no volume change under a certain load.

Purpose of Investigation

The beds of Permian clay underlying Central and Western Oklahoma, Western Texas, and Western Kansas are the subject of this investigation. Due to the wide annual range in temperatures (120° F to -20°F), and rainfall (10 in. to 60 in.) in Oklahoma, an appreciable amount of shrinking and swelling of the clay layers is experienced, with detrimental effects on structures. These unfortunate experiences will be reduced in number and eventually curtailed after a good amount of investigation is conducted and the fruitful results thereof are made known. In this thesis there has been an attempt to partly fulfill this end.

Tests were performed on disturbed samples, limiting the application of the results to compacted embankments, and fills. More specifically the investigation was designed to determine how changes in moisture content, loading condition, and compactive effort affect the consolidation and swelling of this clay.

Remolded samples, resulting from two different compactive efforts, with various water contents, were mounted on consolidation machines and subjected to compression and swelling pressures, and their corresponding volume changes measured. This was meant to lead to the establishment of relationships between the void ratio and water content against the volume change experienced, which would serve to increase the knowledge of the behavior of this clay under these conditions, so that the appropriate safeguards can be taken to avoid undesirable conditions when dealing with the clay tested.

Scope of Investigation

In an attempt to determine the magnitude and causes of volume changes in the remolded Permian clay, the following tests were performed:

1. In order to identify and classify the soil used in this investigation, Atterberg limits tests were run to determine its Liquid Limit, Plastic Limit, Plastic Index, Flow Index, and Toughness Index. (Results may be seen in Figure 4.)

2. In order to find the optimum moisture content of the soil for maximum dry density, compaction tests at various water contents were run. (Results are shown in Figure 6.)

3. The volume change under a given load, whether due to swelling (hereinafter called positive volume change) or consolidation (hereinafter called negative volume change), was obtained from a consolidation test. A sample extracted from the compaction test was subjected to an initial load. Load increments for every test had a pressure-increment ratio (ratio between the load increment and the existing load on the sample) of unity. On samples of an average area of 80.5 sq. cm. the loads applied were increased from 3 to 768 kg., this corresponding to unit pressures of 0.0375 to 9.540 kg/sq. cm. With initial loads of 3, 6, 12, and 24 kg. respectively, the number of increments per test for the consolidation part varied from eight to five and for the swelling part from four to three. All tests were run on approximately 1 in.

thick samples.

A total of twenty-nine samples were tested. This investigation was planned to cover the range of loads normally imposed upon this clay in this area according to their purported function. Therefore, maximum pressure used corresponded to 9.54 kg/sq. cm. which is equal to 9.54 tons/sq. ft. The investigation consisted of six series, five samples making up each series (with the exception of series No.4 which consisted of only four samples). Following the conventional procedure, in each series the compactive effort and loading sequence was kept the same, with moisture content being the only variable, the purpose being either to establish the water content at which a given compactive effort and a certain load would produce no volume change, or given the water content and dry density at a specific compactive effort to find out how much shrinking or swelling a certain load would cause. Using two different compactive efforts the purpose was to find out how this factor affects the total volume change experienced.

A compactive effort above the standard, as well as the standard, was tried. It was meant to try the effects of a compactive effort below the standard but, because of lack of time, this was not attempted.

The six series followed this pattern:

	Compaction			
Series No.	Number of Blows per Layer	Number of Layers	Water Content	Initial Load, kg.
I	25	3	Variable	3
II	25	3	Variable	6
III	25	3	Variable	12
IV	25	3	Variable	24
V	35	3	Variable	3
VI	35	3	Variable	24

Previous Investigations

Research on Permian clays started at the Oklahoma Institute of Technology a few years ago. Its main objective is to gain a better understanding of the behavior of these clays, both in their undisturbed and remolded states. Such an understanding is mandatory for safe design.

Both field measurements of actual movement and laboratory investigations are being conducted. This investigation of the swelling characteristics of a Permian clay forms a part of a wider program which aims at safeguarding structures against heaving of expansive clay.

Some of the physical properties of the Permian red clays were discussed by William H. Hall in 1949 (5)*. R. E. Means, W. H. Hall, and J. V. Parcher (14) extended the investigation to cover building foundations. Some suggestions on preventing damages to buildings are given therein.

Expansive clays have been a matter of concern to soil engineers in Texas. Chester McDowell (12) has been investigating the cause of damages to highway pavements by the heaving action of clays in Texas. He suggests a way of approximating the potential vertical rise of any light-weight structure resting on clay strata based upon the percent volume change. Results of his method checked closely with actual field measurements.

Actual time-movement measurements for a period of four years on a building project in Texas lead Lawrence A. DuBose (4) to contend that

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

heaving of Texas clay can be controlled by proper compaction. These measurements showed that a very negligible expansion should be expected after careful compaction control of the fill operations.

CHAPTER II

PHYSICAL PROPERTIES OF CLAYS

Structure of Soil

An understanding of the structure of soil is a prerequisite for the control or alteration of soil properties.

The equilibrium forces that come into play are the forces between the soil particles themselves and between soil and water.

In general there are two kinds of bonds: (a) atoms can bond to other atoms to form molecules, which bonds are called intramolecular forces; and (b) atoms in one molecule can also bond to atoms in another molecule through the intermolecular forces. The first kind of bonds are very strong forces and are not likely to be broken by the forces normally applied to soil; hence the discussion will be confined to the intermolecular cohesive forces. No detailed description of the forces as such will be given, but only as they affect the soil structure as a whole.

Among the three clay mineral groups: kaolins, illites, and montmorillonoids the latter constitutes the major part of expansive clays. Minerals of that group are exceedingly fine grained crystalline aggregates with an unusually wide range in composition. They have a comparatively weak intermolecular bond and bear a negative surface charge. The strength of this charge, which is a function of the size of the particle, its shape, and its chemical composition, determines how closely the mineral particles are grouped.

In nature there are some crystals electrically neutral, called nonpolar, their layers being held together by stray electrical forces. The electrical charges of the atoms in a water molecule do not balance each other; thus water molecules are considered to be bipolar. In a soil-water suspension, water molecules are adsorbed to soil grains, (some of them may even penetrate into the interior of the grain), forming water films on the surfaces of the soil grains. The thickness of these films depends on the intensity of the electric charge at the grain surface. This adsorbed water film has different properties than normal bulk water due to strong intermolecular attractive forces compressing the water at the solid surface into a dense form of abnormally high viscosity. Investigators have estimated the average adsorption pressure of water to be of the order of 20,000 tons/sq. ft. It is, therefore, believed that close to the solid grains of clay the adsorbed water film has the properties of a solid. Other experiments indicate that treatment of clays with various chemical compounds resulted in changes in their shearing strength, plasticity, and permeability. These phenomena can only be explained by the presence of thin, strongly viscous water films adsorbed by the soil grains. It is reasonable to assume, then, that with increasing distance away from the soil grain the water film assumes more of the physical properties of natural water. When these water films are relatively thick (thickness of adsorbed layer consisting of the semi-solid and solid water estimated at about 0.005 microns) the soil is more plastic.

Mattson (11), in order to give an explanation to the great distance between the particles in a plastic clay suspension, developed the theory that the particles besides being surrounded by their adsorption films are also surrounded by a film of "osmotically imbibed" water. These osmotically imbibed water films account mostly for the plasticity of clays. While they are much thicker than the adsorbed water films, they are very much less strongly held, and their mechanism of attraction, therefore, is entirely different. Osmotic imbibition requires the presence of a surface which can dissociate diffusible ions and is therefore confined to clays. Adsorption, on the other hand, is dependent only upon an intermolecular attraction between solid and liquid and is common to all solids which wet. Adsorbed films can, however, like osmotically imbibed films, be responsible for swelling and shrinking and even for plasticity, but to a very much smaller degree, and only if the solid particles are so small that the film thickness becomes appreciable in comparison. The various water layers surrounding a soil particle are shown in Figure 1.

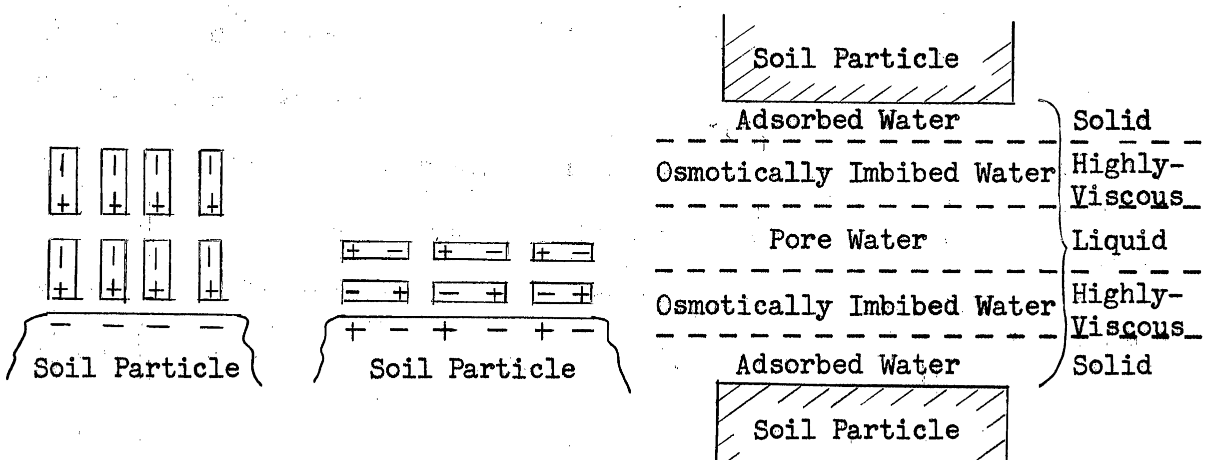


Figure 1

Soil water, in general, is divided into adsorbed or highly viscous water, which is attracted by forces from the soil strong enough to influence its path, and pore water which is water free of any soil attractive forces. Comparatively speaking nearly all of the water in a sandy soil is pore water, but less than half of the water in a saturated cohesive soil may be pore water.

There is likely to be a difference in potential between the adsorbed water or water films and the bulk of the soil water or pore water. Changes in the physical properties of the soil can be brought about by changes in the thickness of the immobilized water. Trace chemicals, or the leaching action of percolating water can bring about some changes. Colloidal particles allow changes of the cation adsorbed on their surface according to their chemical affinity; for example, a hydrogen clay (colloid with adsorbed H cations) can be changed to a sodium clay by percolation of water containing sodium salts. Occasionally these changes decrease the permeability of a soil, as was intentionally done to seal the lagoon lining at Treasure Island with salt in 1940 (9).

The water films coating the soil particles, because of their relative immobility, can be considered as part of the particle as far as permeability is concerned.

There are four kinds of stresses acting in a clay-water system: the hydrostatic pressure of the water; the friction on the surface of the grains caused by the motion of water; the intergranular stresses acting at the points of contact with the adjoining grains; and stresses inside the particle to counterbalance these three stresses acting on the surface of the particle for the existence of mechanical equilibrium. All these stresses act in their own region, and they do not exist outside their respective region of action. In soil mechanics a simplification of this state of stresses is introduced. Two groups of stresses are assumed to act, whose members are in equilibrium with each other; the intergranular stresses are in equilibrium with the friction on the surface of the grains, while the stresses inside the particles are in equilibrium with the hydrostatic pressure. In a state of equilibrium the increase of one stress due to any gravity forces would automatically cause an increase in the corresponding stress.

Sedimentation

Physical agents - water, ice, wind, and gravity - cause weathering of rocks. The products of this weathering are successively deposited in decreasing order of size, proportional to the velocity of the agent. Nearly all clay particles being colloids are most likely deposited at the bottom of lakes or oceans.

The molecules of a liquid, being in constant vibration, strike against the clay particles. Due to the like charges they carry, the clay particles move around in suspension. Their relatively light weight prevents them from settling out of suspension. The vibrating water molecules impart to the clay particles an irregular motion known as Brownian movement. In the presence of a coagulating agent the particles are discharged, and the repulsion between them ceases. The small colloidal particles, then, form flocs large enough not to be greatly affected by the weight of the grains. Coming in contact with each other they gradually form larger and heavier flocs until miniature arches are formed, which slowly settle at random and a flocculent structure results with relatively large void spaces. This flocculent structure can support reasonably heavy loads up to the point of breakdown of the intermolecular bonds.

It is known that forces between particles depend not only upon the distance between them, the force between two atoms varies inversely as seventh power of the distance between them, but also on the medium separating them. Once the medium is changed, especially when water is added to fill the voids of the soil mass, the intergranular forces decrease, loosening the bond between the grains and allowing swelling to occur. The rate of swelling, like the rate of consolidation, depends upon the ease with which water can percolate; i.e., the permeability of the material. The smaller the pores forming void spaces in the soil mass, the lower its coefficient of permeability. Permeability, in turn, depends on the degree of saturation of the material. If the soil grains are already well coated with water films and are partially saturated, the incoming water will meet with greater difficulty finding its way through the reduced pore spaces than when the soil grains are in a relatively dry state; thus increasing the time required for volume change to take place.

In marine deposits sedimentation takes place at a faster rate due to the salt concentration. In fresh water deposits due to the lack of electrical attraction the grains need more contact surfaces for equilibrium purposes; thus a deposit of higher density results.

Consolidation

The gradual deposition of layers upon layer increases the pressure imposed upon the original strata. These pressures are at first carried by the pore water as hydrostatic excess pressures. As the pore water drains because of the excess pressure, the additional pressure is transferred to the soil, consolidating it by a volume change equal to the volume of drained water. During consolidation the particles come closer to each other, and the intergranular forces increase, which means increase in cohesion.

Even after the excess hydrostatic pressures in the pore water are dissipated the particles continue to approach each other. During this very slow process the adsorbed water is extruded from between the particles and, to some extent, becomes pore water. This results in strength increase, since the particles are closer together at points of contact, and in permeability decrease, since the volume of flow channels tends to decrease. Volume changes due to this process might be negligible as compared to the ones due to secondary compression. Shifting of particles and the breakdown of aggregates are two of the effects of secondary compression. The denser the soil the less volume change will be produced due to breakdown of its structure; consequently, secondary compression effects on remolded soil can be considered negligible. In any event these effects require considerable lapse of time to produce a noticeable volume change.

The strength of clays, therefore, depends upon the time under load, as well as the magnitude of the load and the void ratio of the clay. The effect of time is more pronounced in remolded clays than in undisturbed which have already had some of their adsorbed water squeezed from between the contact surfaces. Desiccation, on the other hand, can accomplish in a short time what extremely large pressures will accomplish in a long time on a saturated soil.

Remolding Effects

Clays lose their strength with remolding. Remolding pushes particles apart at points of contact and destroys some of the inaccessible zones in the soil; thereby making the pore water in the system more nearly continuous. This separation of particles reduces cohesion, but increases the effective surface area of the soil particles and the amount of adsorbed water in the system. In other words, the previously inaccessible particle surfaces now adsorb pore water. Experiments (1) showed that a soil has more adsorbed water when remolded than it did before remolding.

Remolding an undisturbed clay (a) increases the amount of adsorbed water at the expense of the pore water, (b) renders closer contact between soil particles, and (c) destroys the largest voids, decreasing the void ratio of the mass. Figure 2 is a schematic presentation of an undisturbed and remolded mass of clay.

Figure 2

The differences between the undisturbed and remolded states may be summarized as follows: The undisturbed clay has, generally speaking, higher strength because of the smaller distance between particles at points of contact which resulted from the action of heavy loads for a long period of time or by drying. The permeability of a remolded clay is smaller than that of an undisturbed clay. This is explained by the reduced amount of pore water, by the closer contact between particles, and by the destruction of the largest pores, or flow channels.

Increase of compressive strength as a result of remolding may be due to: (a) the breaking down of weak aggregates exposing more mineral surfaces which can adsorb the available water, changing it from free to more viscous or plastic film water, (b) the interlocking of particles, especially in the case of certain flocculated sedimentary clays, (c) the breaking up of planes of weakness and lubrication, and (d) the closing of fissures and consequent elimination of planes of weakness.

From void ratio versus pressure curves it is observed that the curve for the remolded state has lower void ratio values corresponding to the same pressures; and the curve for the remolded state is less steep than the virgin part of the curve for the undisturbed sample.

Several experiments performed (15) showed that some of the tested soils regained 100% or more of their undisturbed strength, but most of them did not. The importance of the thousands of years of confining pressure to which the in situ deposit has been subjected cannot be outweighted by the closer distance between particles in regaining the original undisturbed strength of a clay after it is remolded.

Compaction

Compaction is the artificial compression of a soil by mechanical means, as contrasted to consolidation which is the densification of a mass of soil due to volume decrease brought about by the gradual squeezing out of the water from the pores of the soil mass. Proper compaction of fills is of greatest importance, since upon compaction depends the shearing strength of the fill and hence the stability of earth dams, embankments, and highway and airport base courses.

A saturated mass of compressible soil will be considered. On remolding the internal structure of the soil is destroyed. Not only the water films around the individual grains become distorted when the stress applied is greater than their bond strength, but also the grains themselves are displaced. This process is the result of compactive effort on the soil. After the impact stresses come into equilibrium with the effective residual excess hydrostatic stresses the soil mass achieves its densest state; i.e., adsorbed water at points of contact is driven into the void spaces. This means a lower void ratio with higher unit dry density.

The indirect result of compaction, then, is to decrease the bond between the soil grains. The heavier and more consistent the compactive effort is the less the thickness of the water films that remain coating the solid grains, and the greater the shearing strength of the soil mass.

Once it is remolded the structure never returns to its original condition. On submersion in water the gradual settling of solid particles in flocs cannot be duplicated; the shearing resistance due to adhesion between the grains that was inherent to their original structure can never be attained after remolding and compaction.

If a given soil in an air-dry condition is placed in a container and submitted to a definite compactive effort, a certain density (usually measured in pounds per cubic foot) will be obtained. As water is progressively added and the same compactive effort used, the optimum moisture content will be reached at which the density of the soil is maximum. If a greater compactive effort is used, a higher maximum density will be obtained at a lower optimum moisture content. On the other hand if a smaller compactive effort is used a lower maximum density will be attained at a higher optimum moisture content.

As far as compaction requirements are concerned, the American Association of State Highway Officials (AASHO) makes the following recommendations regarding the material to be used in fills.

For fills not exceeding 10 ft. in height and not made on steep slopes or subject to long inundation, the Liquid Limit of the soil should not exceed 65, the Plasticity Index should not be less than $0.6w_L - 9.0$, and the compaction test results should be:

Maximum Dry Weight lb/cu. ft.	Rating	Minimum Field Compaction Requirements % of Maximum Dry Weight
90.0 and below	Very Poor	95
100.0 - 109.9	Poor	95
110.0 - 119.9	Fair	90
120.0 - 129.9	Good	90
130.0 and above	Excellent	90

For fills exceeding 10 ft. in height or placed on steep slopes or subject to inundation, the Liquid Limit of the soil should not exceed 50, and standard compaction tests should show:

Maximum Dry Weight lb/cu. ft.	Rating	Minimum Field Compaction Requirements % of Maximum Dry Weight
99.9 and below	Unsatisfactory	.....
100.0 - 109.9	Very Poor	100
110.0 - 119.9	Poor	95
120.0 - 129.9	Fair	90
130.0 and above	Good	90

Exemplifying the effect of compaction, excavated earth will expand beyond its original volume in the transporting vehicle, but it will shrink below the excavated volume when placed and compacted in the fill. To illustrate, 1 cu. yd. of earth in the cut may use 1.25 cu. yd. of space in the transporting vehicle, and finally occupy only 0.85 to 0.65 cu. yd. in the embankment, depending on its original density and the amount of compaction applied.

There are three ways of compacting a soil: by impact, rolling, and vibration. Rolling is the most effective way of compaction when large masses of earth are to be handled. For the compaction of clay fills a sheeps-foot roller is most commonly used. Compaction by impact is achieved with a gasoline rammer usually weighing 210 lb., which, because of its small contact area (0.492 sq. ft.), is suitable for the compaction of backfill in trenches or quite close to concrete structures. Noncohesive soils can most effectively be compacted by vibration.

Shrinkage

There are two kinds of forces regulating intermolecular action: the attraction and repulsion forces. The repulsion forces are in general attributed to electrokinetic charges; the opposing attraction forces are ascribed to the compression exerted on soil grains by the surface tension of the water films. The latter forces predominate in the drying process of a soil system. They will be explained in terms of a saturated compressible clay.

Pore spaces between the very fine clay grains are so minute that they will be referred to as capillary. These capillary tubes are interconnected throughout the soil mass and are variable in size. When the pore spaces are filled with water and there is some free water at the surface there is no meniscus formed or the meniscus is a plane surface, with no tension in the water, as shown by Position 1 on Figure 3(magnified many times). Evaporation of the pore water will cause a meniscus to form in each of the pores, as shown at Position 2 with a resulting tension in the water. The fully developed meniscus will be in the largest pore. There will be hydrostatic forces exerted on the soil grains due to tension in the pore water. This force varies inversely with the radius of the meniscus and directly with the surface tension of the water. The tension of the water exerts a hydrostatic pressure (compression) on the soil grains producing an elastic deformation of the particles and a relative decrease in volume. With further evaporation the meniscus will recede to Position 3 with a smaller

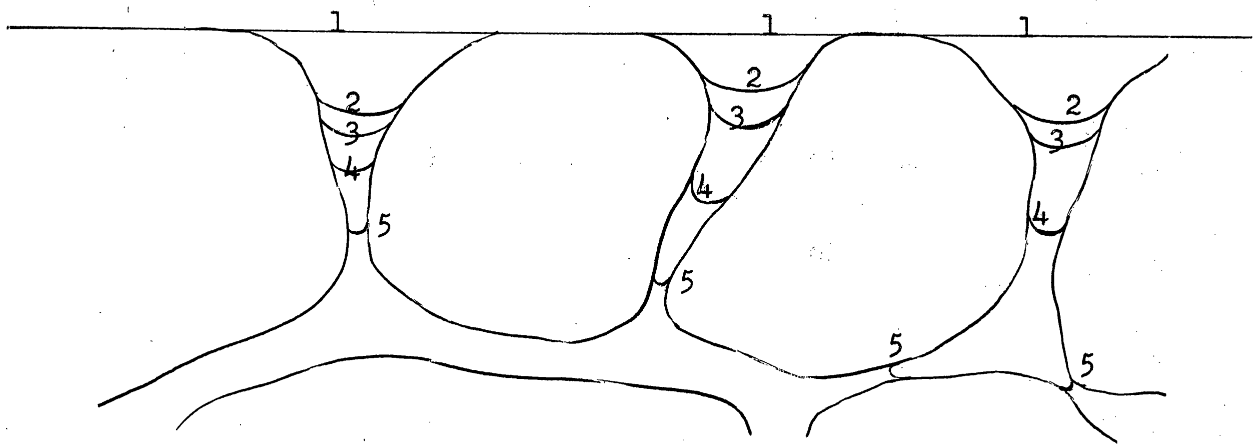


Figure 3. Drying Stages in a Soil Mass

radius. Due to the continuity of pore spaces all other menisci will have the same radius. The stress in the water increases as the radius of meniscus decreases, causing an increased compression between the soil grains, and further reduction in the volume of the mass.

This shrinkage process will continue until the menisci reach the smallest diameter in the largest capillary tubes, as shown by Position 5. The radius at this point will be the smallest of a fully developed meniscus, and the tension in the water will be a maximum.

After this point, called the shrinkage limit, any additional reduction in the moisture content will produce no increase in tension in the pore water and, therefore, no further decrease in volume. This is considered the transition point of the soil mass from the semi-solid to the solid state which is evidenced by the surface of the material becoming lighter in color.

The void ratio of the soil mass assumes its lowest value at the shrinkage limit, since there will be no further reduction in volume after the shrinkage limit is reached. The total volume reduction corresponds to the volume of the water evaporated during the shrinkage process up to the shrinkage limit.

At a given moisture content the thickness of the water films will be that required to give an equilibrium value of capillary pressure. As drying proceeds, the capillary pressure increases as the soil grains move closer together. At the shrinkage limit, when the water content decreases without any appreciable volume change, the capillary pressure reaches a maximum value. For the dry surfaces in contact the attraction forces predominate and hold the particles together. After the shrinkage limit is reached the water vaporizes in the interior and is transferred to the surface as vapor.

The compressive force, therefore, depends upon the surface tension of water. Water, having a high surface tension (72 dynes per cm.), compresses highly the soil grains. In some cases there is a slight contraction following the shrinkage limit, since the capillary forces apparently compress the clay particles slightly at the points of contact. With lower surface tension the capillary pressure is decreased. As the pore water moves through the clay after the shrinkage limit stage, the remaining water films are removed and a small secondary shrinkage occurs which increases in magnitude as the surface tension is lowered. A smaller particle size will increase drying shrinkage since the number of water films increases as particle size decreases. The major factors, therefore, affecting the drying behavior are: (a) particle size, (b) surface tension, and (c) water content.

When free water is admitted to the pore spaces of a compressible mass of soil after having been shrunk by evaporation, the menisci will be destroyed, the surface tension will become zero, and the pressure between the grains will be relieved. Under these conditions the clay mass will start swelling due to elastic rebound of the particles.

Swelling

Water added to a disturbed mass of soil will tend to thicken the moisture films, if such films already exist, around the grains thus imparting a relative motion among the grains pushing each other apart; ultimately the water will fill the voids of the soil mass. The relative incompressibility of water coupled with its affinity to the electric charges on the soil grains and helped by the passive strength of the grains which have lost their bond, all these factors serve for the swelling of the soil mass. Another factor mostly conducive to swelling is the elasticity of the clay particles. On removal of the load which induced a certain deformation to the particles, the soil grains will rebound and tend to assume their former shape. This rebound causes certain voids in the volume which available water rushes to fill. Gravitational and capillary forces also push the water through the pores of the soil; the former depend on the head or pressure of water whereas the latter depend on the pore diameter. The smaller the effective diameter of the pores the greater is the distance that the water will rise in them. Consequently the rate and degree of swelling actually depend upon the hydrostatic head, the size of the grains, the amount of elastic deformation of the particles, and the available moisture.

Swelling of a mass of soil can be explained in terms of energy. When work is done by compacting a soil at whatever moisture content an amount of energy is stored in the soil mass equivalent to the compactive effort expended. The soil grains possess an amount of internal energy by virtue of their configuration and motion. If the soil mass is relieved of its overburden, its stored potential energy is transferred into kinetic energy due to the swelling pressures brought into the pores by the free water; thus the soil mass rebounds or swells upon the removal of the load. Hence, theoretically speaking, no rebound or swell should occur when, with constant moisture content, pressures are imposed upon the soil causing stresses equivalent to the compaction stresses.

It is generally conceded that consolidation and swelling are two fundamental phenomena which must be understood by everyone who attempts to gain a knowledge of soil behavior in engineering problems (7).

CHAPTER III

DESCRIPTION OF MATERIAL

Geologic Origin

From geological investigations it is believed that the area today covered by Western Oklahoma, Western Texas, and Western Kansas during the Permian period was an inland sea surrounded by relatively high mountains. From the steep slopes of these mountains through physical and chemical weathering the disintegrated products of rocks were deposited in this area. Due to shifting of the tributaries with the resultant erosion and the resedimentation it is not surprising to find so much variety in the physical properties of these clays.

The fact that three layers of gypsum lie between thick layers of clay and sandstone strata prompted the geologists to infer that there must have been three severe drought seasons with three ensuing periods of deposition.

At present the climate of this area is variable. With the annual rainfall ranging between ten to sixty inches most of the rain comes during two of the spring months; the rest of the rain can be scattered indiscriminately throughout the rest of the year. Drought periods are not infrequent in this region. These factors affect the soil properties; for example, the depth of the zone of drying approximates fifteen to twenty feet depending on the geographic region. In this area more rain falls in the Eastern part than in the Western.

Because of the rather great percentage of particles of sand size making up this clay it is reasoned that the clay under consideration is a marine clay. Based on this reasoning and knowing that flocculation of marine clays takes place at such a rapid rate that the flocs include many of the heavier silt and fine sand grains, it can be implied that these clays are of flocculent structure; i.e., a structure consisting of a system of large honeycombs which incorporates the coarser grains, silts, and of a honeycomb structure of second order, consisting of small clay grains and flocs of coagulated colloidal clay. These flocs in themselves represent a honeycomb structure of a still smaller order.

These Permian deposits of clay and sandstone were covered by later deposits of the Mesozoic and Cenozoic eras to a depth of several hundred feet. These later deposits have since been eroded to again uncover the Permian deposits in the area. Such a heavy overburden produced very strong, hard, overconsolidated clays which can support extremely heavy loads in their natural state. Because of their high content of minerals of the montmorillonite group they tend to swell considerably when water is admitted to their pore spaces and shrink when desiccated.

The soil under investigation was a dark reddish brown, jointed clay, with occasional black specks, disturbed, taken from a ditch three feet below the surface on the north side of the new (1956) federal agricultural offices building on the Oklahoma A. & M. College campus. It constitutes a representative specimen of the clay common to much of this area. This clay was very stiff when dry and quite plastic when saturated.

Main Physical Properties

Specific gravity and Atterberg limits tests gave the following characteristic values:

G_s	w_L %	w_P %	I_p	I_F	I_T
2.74	36.3	14.6	21.7	12.5	1.74

The graphical determination of the liquid limit, w_L, is shown on Figure 4.

The rather high plastic index (21.7) indicates a soil of considerable volume change variation with fluctuations of its water content; its liquid limit of 36.3% reveals a soil of moderate compressibility and a considerable amount of cohesionless material.

Compaction tests (see Figure 6) run in conjuction with the consolidation tests gave the following results:

Designation	Compaction Number of Blows per Layer	Compaction Number of Layers	Optimum Moisture Content %	Void Ratio, e	Maximum Dry Density γ_{dry} lb/cu. ft.
A	25	3	15.1	0.524	112.21
B	35	3	13.2	0.522	112.32

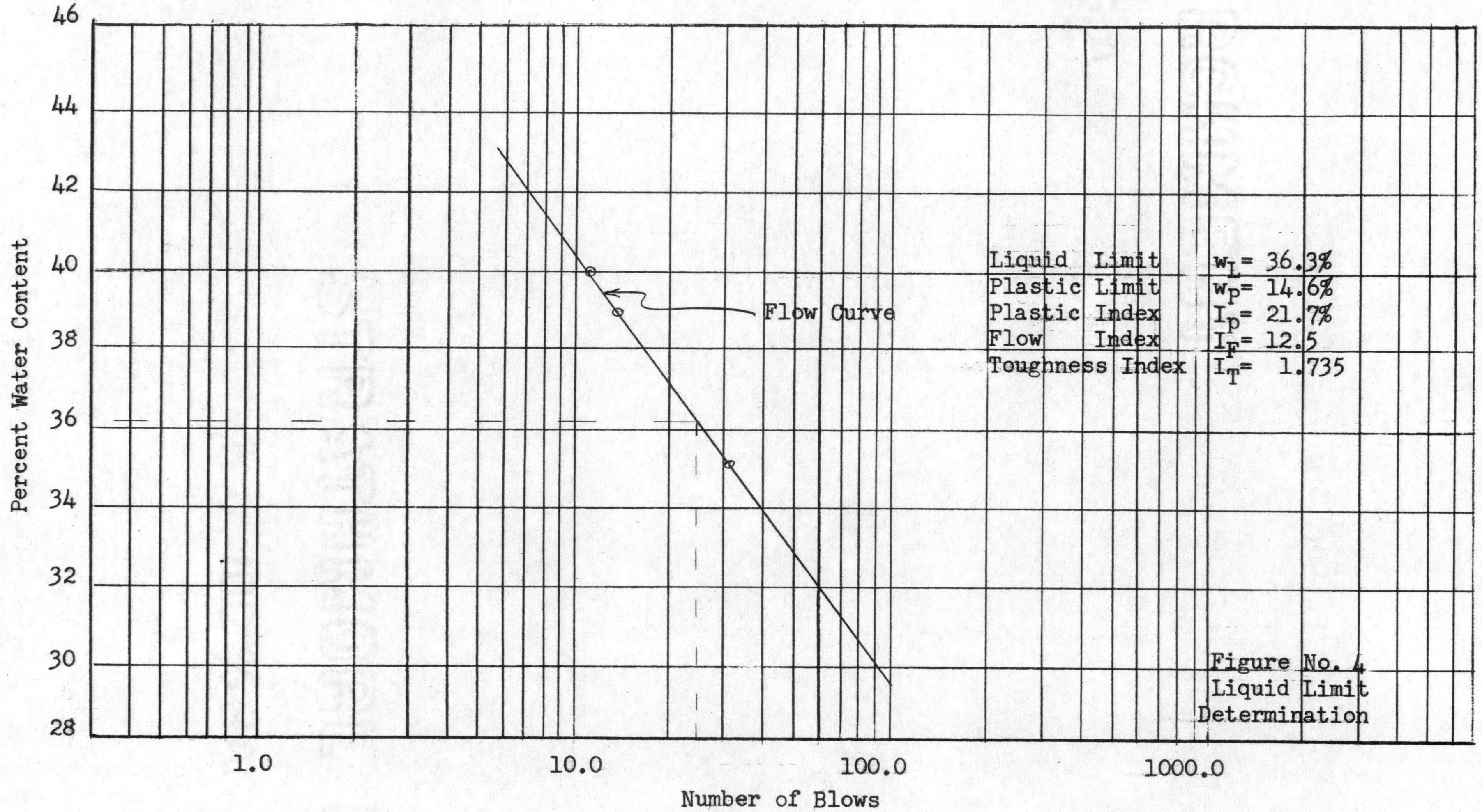


Figure No. 4
Liquid Limit
Determination

CHAPTER IV

TESTS

General

As of today three complex petrographic tests are available to determine if expansive material is present in a soil and in what quantities; the microscopic examination, X-ray diffraction, and differential thermal analysis. For a complete investigation of the mineralogical composition and influence of texture and structure of a fine-grained material the combination of the results of all three methods is necessary.

There are other simplified identification tests to determine possible expansive characteristics of clays. The colloid content of a soil and the Atterberg limits considered together serve the purpose.

A grain size analysis was not included in this investigation. The Atterberg limits tests were run for the purpose of indentifying the soil used in this investigation in order to compare our results with previous investigations, and as a guide to future investigations for the research program under way at this institution.

Load-expansion tests were performed to provide factual data for predicting the amounts of expansion. They are a form of the standard one-dimensional consolidation test.

Equipment Used

Atterberg Limits. For the determination of the liquid limit of the material the standard liquid limit device, developed by Arthur Casagrande (2) was used.

The plastic limit determination followed the procedure set out by T. William Lambe (7).

Results of both tests appear on Figure 4.

<u>Compaction</u> <u>and</u> <u>Consolidation</u> <u>Tests</u>. The special equipment used was a frame with ten consolidation units, five on each side, using light weights and a multiplying lever loading system. Adjustment for balance is provided through jacks at the loading yoke end. As soil containers (consolidometers) the floating-ring type was used. Advantages of the machine system used, on the one hand, are that it is more compact than a jack with platform scales system, and assures constancy of loads applied. The floating-ring type of container, on the other hand minimizes the effect of friction between the container wall and the soil specimen, as contrasted to the fixed-ring container. The use of the latter would have been mandatory if permeability measurements were to be made. The diameter of the clay sample was much greater than its thickness; hence the friction along the sides of the sample has practically no influence on the volume of voids. This side friction is an undesirable factor since it actually does not occur in nature. A comparative diagrammatic sketch of both types of containers, indicating the effect of side friction is shown in Figure 5.

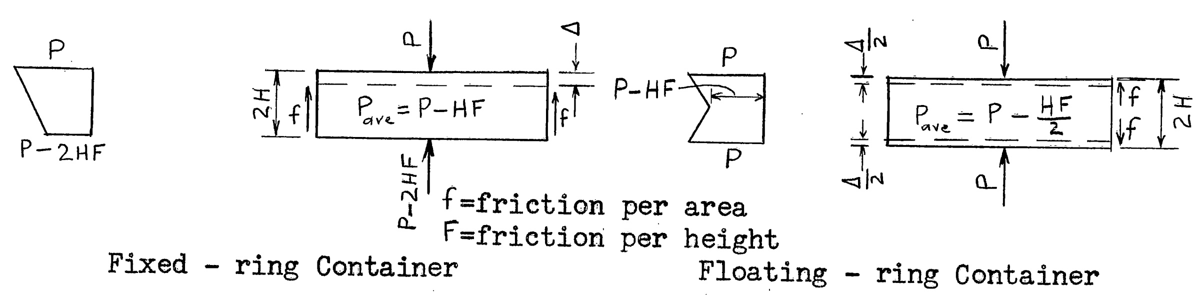


Figure 5. Side-Friction

Research on the consolidation of the Boston blue clay at the Massachusetts Institute of Technology (16) evaluated the side friction effects in the order of 10 - 20% of the vertical load. While important, these effects can be neglected with the assumption of one-dimensional consolidation.

The samples tested were taken from a compaction test. The consolidation ring was mounted between two split rings to form the compaction mold. The two split rings were both laterally and vertically fastened by a bolt and nut arrangement, thus insuring no leakage or extrusion of soil through their connections. A metal plate formed the base on which the cylinder was placed when running the compaction test. The total volume of the cylinder for the compaction test, therefore, consisted of the volume of the two split rings plus the volume of the consolidation ring to be used. This total volume ranged for the different rings between 930 to 982 cubic centimeters.

Variations of compactive effort were brought about by variations in the number of blows for each of the three layers of soil in the compaction cylinder. The same tamper was used throughout. It weighted 5.5 pounds and had a plane face 2 inches in diameter at the striking surface. An arrangement provided for each blow to have a free fall of 12 inches.

Consolidation ring diameters ranged between 10.127 to 10.1260 centimeters, with heights in the range of 2.48 to 2.57 centimeters.

The cylindrical porous stones placed above and below the sample serving to permit drainage or saturation had diameters of 10 centimeters, and weighted dry about 780 grams and saturated about 887 grams.

Sample Preparation

Two porous stones, which had been immersed in water for several hours, were removed from the water and allowed to stand at room temperature for at least one hour. After the dimensions and tare weight of the ring to be used had been taken, it was placed between the split rings and fastened in place. The weight and dimensions of the cylinder (two split rings and soil container) were recorded.

The proper quantity of soil to be tested, adequate to fill the compaction cylinder, passing U. S. sieve No.10, was thoroughly mixed with the appropriate quantity of water according to the desired water content. This mixture was placed in three layers in the compaction cylinder, each layer being tamped with as many evenly distributed blows as the specific compactive effort of the series called for. After weighing the compacted soil and cylinder, the top and bottom split rings were removed and the excess compacted material removed to prepare the consolidation sample. Samples for water content determinations were taken from both the top and bottom of the compaction sample. To insure no loss of moisture, immediately after very carefully trimming the top and bottom faces of the consolidation specimen flush with the container, taken from the middle of the cylinder, it was weighted and the weight recorded as tare plus soil wet at the beginning of test. This constitutes the soil consolidation sample to be tested. It was then mounted on a consolidation unit between the two porous stones, making sure that both the porous stones were properly centered on the sample surface. The loading unit was adjusted and lowered until it just made contact with the porous stones, and consequently with the sample itself. Volume changes during the test were registered by a vertical-deflection Ames dial. Immediately after contact of the machine with the sample was assured, an Ames dial was mounted, adjusted and the first reading recorded. The initial load, according to the series performed, was applied immediately thereafter. The sample thus loaded was left to set for at least twelve hours. This gave ample time for the determination of the actual water content of the sample before the preparation of the next one. Then the sample was flooded and readings of time and vertical deflection were taken from the instant the sample was completely immersed in water.

Compression Test Procedure

In order to get a good spread of points even at the early portion of the stress-strain curve - though experience proved that this clay has been subjected to large loads before - the initial load was as low as 3 kg., which corresponds to an average unit pressure of 0.0375 kg/sq. cm.

For this not very compressible clay and the thickness of the specimens used it has been established that the time intervals for readings to give a good spacing of points on the five-cycle plot of dial readings versus time should be taken at total elapsed times of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240 minutes etc... Thus the thickness of the sample could be determined at every elapsed time at which a reading was taken.

The same load was left on the sample for approximately forty-eight hours - a preestablished period during which specimens of the same clay passed the 100% theoretical consolidation and compression practically ceased. At the end of forty-eight hours a reading was taken before the next load in the loading sequence was applied. After the application of the new load, readings at the elapsed times previously specified were taken and recorded.

The loading program was fixed to follow the following sequence: 0, 3, 6, 12, 24, 48, 96, 192, 384, 768, 192, 48, 12, 3, 0 kg. except for ommission of the loads lighter than the initial load. It will be noticed that the consecutive loads follow a geometric series, being doubled each time, up to the maximum load of 768 kg., after which two of the initial load increments were removed at a time so that the stages for the swelling part of the test were half as many as those of the consolidation process, depending on the initial load. This loading sequence was followed throughout the project, irrespective of the starting load for the respective series.

Though the apparatus used could take loads greater than 768 kg., data for larger pressures were considered of no practical importance. Since swelling information counted as much as compression effects, the same sequence of time readings and data was taken as for the consolidation process. Throughout the test the water level was maintained above the top of the sample so as to prevent evaporation of the water and annul any surface tension resulting therefrom.

At the completion of the test the sample was removed, and its weight determined before it was placed in a drying oven. The weight of dry soil was needed for the water content determination. After that, each consolidation unit was calibrated with the porous stonesflooded and loading procedure similar to the one followed before the removal of the sample. Difference in dial readings between two successive loads was used to correct for the machine deformation.

Variables Involved

Compaction Test. The portion of the sample of soil that was left over from one compaction trial was reused to obtain all the density-water content measurements, as well as the next consolidation ring sample. The soil was left to air-dry between trials for ease of sieving, and each time the appropriate amount of water was added so that a mix with higher water content than the previous one would result, and the whole compacted immediately after thorough mixing. Thus samples were prepared every other day. Research has shown that closer agreement with field conditions can be obtained if the water is allowed to soak for some time (e.g. overnight), and a fresh sample of soil is used for each determination (7). For example, it was found that densities as much as 8 lb/cu. ft. smaller were obtained by using fresh samples. Allowing the mixing water to soak in before compaction gives higher densities for many soils, particularly for those with porous particles. Since the samples for this research project were not soaked prior to compaction and the same soil sample was reused, part of the low densities arrived at may be explained.

Consolidation Test. The duration for each load should be the same for an accurate void ratio - pressure curve. If it is not uniform, secondary consolidation effects are more pronounced. For the clay under investigation each loading increment was allowed to act for approximately forty-eight hours (more exactly, elapsed times ranged between 2740 to 3050 min.) If any one increment were allowed to run for a longer time a larger value for the difference in void ratios would be obtained, and effects in time and compression values of the next increment would be noticeable. However, if each increment were allowed to stand for a longer interval, for one week say, the resulting curve would be merely displaced vertically without appreciable change of slope. The values of void ratio change and compressibility coefficient are, therefore, independent of the time allowed for each loading increment.

The results of laboratory tests depend on the size of the specimen used. A series of tests on five widely different clays, with the same specimens, indicated that the pressure - void ratio curve was essentially independent of size (7). The rate of compression, though, was greatly dependent on size; higher coefficients of consolidation were obtained on the larger specimens. A ratio of specimen diameter to thickness of about three to four is recommended. Diameters greater than $2\frac{1}{2}$ to 2 3/4 inches are desirable. Both stipulations were satisfied in this investigation.

The water content sample from the compaction test instead of being made up of specimens from the top, middle, and bottom of the compacted soil, only two samples one from the top split ring and another from the bottom were taken. This practice eliminated excessive evaporation from the sample due to uncontrolled room temperatures; also it afforded a check on how uniformly the added water was mixed with the soil. Satisfactory values were obtained which checked with the initial water content of the consolidation sample, from the middle portion of the compaction mold. An average value of the water content was used. This average value affected the unit dry density and void ratio values in the calculations.

The moisture of the room was kept as low as possible, varying with the prevailing weather.

No special care was taken to avoid entrapped air bubbles in the water.

Test Limitations

Whenever the lever arm of a consolidation unit was about to touch the machine due to the last applied load increment, before the next increment was applied, the machine was occasionally jacked as far up as it would make the application of at least one more loading increment possible. This action, needless to say, imposed a momentary stress upon the sample causing a corresponding decrease in its void ratio which should have been brought about gradually. The effect of this action is noticeable on some time curves. Since swelling characteristics of this clay was the main feature of this study, such a minor disturbance has no direct bearing on the results obtained.

After the samples had swelled under the initial load they were left in the consolidation unit and were subjected to a regular consolidation test. Thus the other consolidation constants were derived, without materially affecting the initial swelling values.

Test Observations

Test procedure provided for a consolidation time for every load increment of approximately forty-eight hours. This period of time was established for this clay because compression or swelling of the sample was observed to be negligible beyond this point. This, however, was not the case at the end of each test when the sample was unloaded. Swelling was occuring then at a very slowly decelerating rate and would continue for a few days. This phenomenon is an indication of the expansive capabilities of the Permian clay at an unloaded condition.

In general under all initial pressures applied - up to 0.2963 kg/sq. cm. - all samples swelled from their initial water content to saturation.

CHAPTER V

PRESENTATION AND INTERPRETATION OF RESULTS

Compaction test results appear in Figure 6 in the form of an optimum moisture content determination; i.e., a plot of unit dry density and void ratio against percent water content. From this figure it can be seen that approximately 85% saturation was the maximum accomplished.

Time curves from the consolidation test data were plotted to furnish the necessary data for the void ratio - log pressure curves for each initial loading condition. From these curves it is apparent that the coefficient of consolidation varies widely with pressure. This fact was also established by previous investigators. The log fitting method was found most suitable for plotting the pressures imposed upon this clay. Such time curves are not shown in this report because they have no direct bearing on the conclusions arrived at.

From the void ratio - log pressure curves the coefficient of compressibility, C_c; the coefficient of swelling, C_s; the precompression unit pressure, P_p; the pressure on curve corresponding to void ratio at P_p, p_{e_s}; and the void ratio after swelling under p_s, e_s were obtained. In addition to the above Table 1 shows the initial unit pressure imposed, p_s; the initial void ratio of the sample, e_o; total swelling under the initial load, and unit deformation due to swelling for each sample. These values were tabulated in their respective series in order of increasing initial water contents.

The ultimate purpose of this investigation was to find the water content at which the clay tested should be compacted so that it will swell the least when loaded with a certain load. In other words, this study was based on the belief that compaction control can minimize possible volume changes. This belief was substantiated in Texas with actual measurements on a warehouse floor resting on a fill of their expansive clay (4).

With this scope in mind the experimental findings of twenty-nine specimens are presented in the following sequence:

The water content is plotted against the void ratio for both compactive efforts, A and B, on Figure 6. The curve for each compactive effort is the average drawn between the values obtained for the different series. The relationship shown makes possible a direct interpretation of the water content of a sample, compacted with a certain compactive effort, to its void ratio. Results conform to established principles; i.e., a heavier compaction gave a smaller optimum moisture content with a resulting higher density. Figure 6 shows also a very small difference in the void ratio values that resulted from two different compactive efforts. Later on the reason is given why no further use of the void ratio values is made in this report.

Water content versus the precompression by compaction load was plotted for all series. All curves followed the same trend. A representative curve for series No.2 is shown in Figure 7. As expected, with the water content approaching the optimum the pressure imparted on the sample as a whole by the compactive effort increases. A correlation to the behavior of the water content - void ratio curve is evident.

Figure 8 shows a plot of the coefficient of compressibility versus water content for compaction A, and Figure 9 for compaction B. It is remarkable to find out that even the coefficient of compressibility of a sample is related to the moisture content during compaction. The rate of compressibility of this clay is reduced with increasing water content up to the optimum, where the coefficient of compressibility is smallest. Above the optimum moisture content the coefficient of compressibility keeps increasing at nearly the same rate it was decreasing before it reached the optimum condition. The same effect is exhibited by the curves for both compactive efforts.

From the tabulated data it can be seen that there is no relation between the coefficient of compressibility and the coefficient of swelling. The relationship, if any, between an average value of the water content for each sample and its coefficient of swelling was not investigated.

Two void ratio - log pressure curves for the same initial unit pressure (0.0375 kg/sq. cm., the least used) are shown in Figures 10 and 11 in order to illustrate the effect of compactive effort on the consolidation constants. Figure 10 represents compactive effort A with an initial water content of 13.80%; Figure 11 represents compactive effort B with an initial water content of 14.55%. Considering the difference in initial water contents to be negligible, the following deductions can be made for the effect of two different compactive efforts: Though the values of the precompression load are graphically derived they show close proximity, 0.60 kg/sq. cm. for compaction A and 0.675 kg/sq. cm. for compaction B. The void ratio values 0.565 .555 and 0.545 respectively point to the effect of the increased compactive effort in the second case. Unit pressure values at the void ratios of the precompression loads are 0.45 .56 and ~~0.56~~ .45 kg/sq. cm. corresponding to the above void ratios. They substantiate the fact that more overburden pressure was required to impart more density to the material. These two specific samples after they had been compressed under the same cycle swelled at the same rate, though derived from different compactive efforts; i.e., they have identical coefficient of swelling values, 0.024.

The same comparison of values will be attempted for two samples with the same initial unit pressure which is the greatest used in these tests. Figures 12 and 13 are meant to serve for this purpose. Again the difference in the water contents of these samples, 15.70% for compaction A sample and 15.93% for compaction B sample, can be neglected. It is observed that the void ratio - log pressure curve corresponding to compaction A is much steeper before the precompression load was reached than the one under compaction B. This effect is due to the increased density of the second sample brought about by a heavier compactive effort. The same relationship exists between P_p (0.59 and 0.95 kg/sq. cm.), e_n (0.617 and 0.560), and p_{e_s} (0.55 and 0.~~86~~68 kg/sq. cm.) referring to compaction A and B respectively. The values for the coefficient of swelling in this case are different. Compactive effort A gave a C_s = 0.0188 and effort B gave 0.0238. This relationship holds true for all other samples. The samples subjected to an increased compactive effort, after they have consolidated under certain loads, on the gradual removal of these loads, swelled at a faster rate than the ones subjected to less compactive effort. This fact serves to indicate the possibility that closer contact between the soil grains, i.e. stronger bond, is achieved by a gradual adjustment of position of the grains due to successive loading increments rather than the adjustment being brought about by dynamic loads.

From Table 1 it is observed that of two samples of nearly the same initial void ratio, compacted the same, the one with a higher initial water content swelled the least.

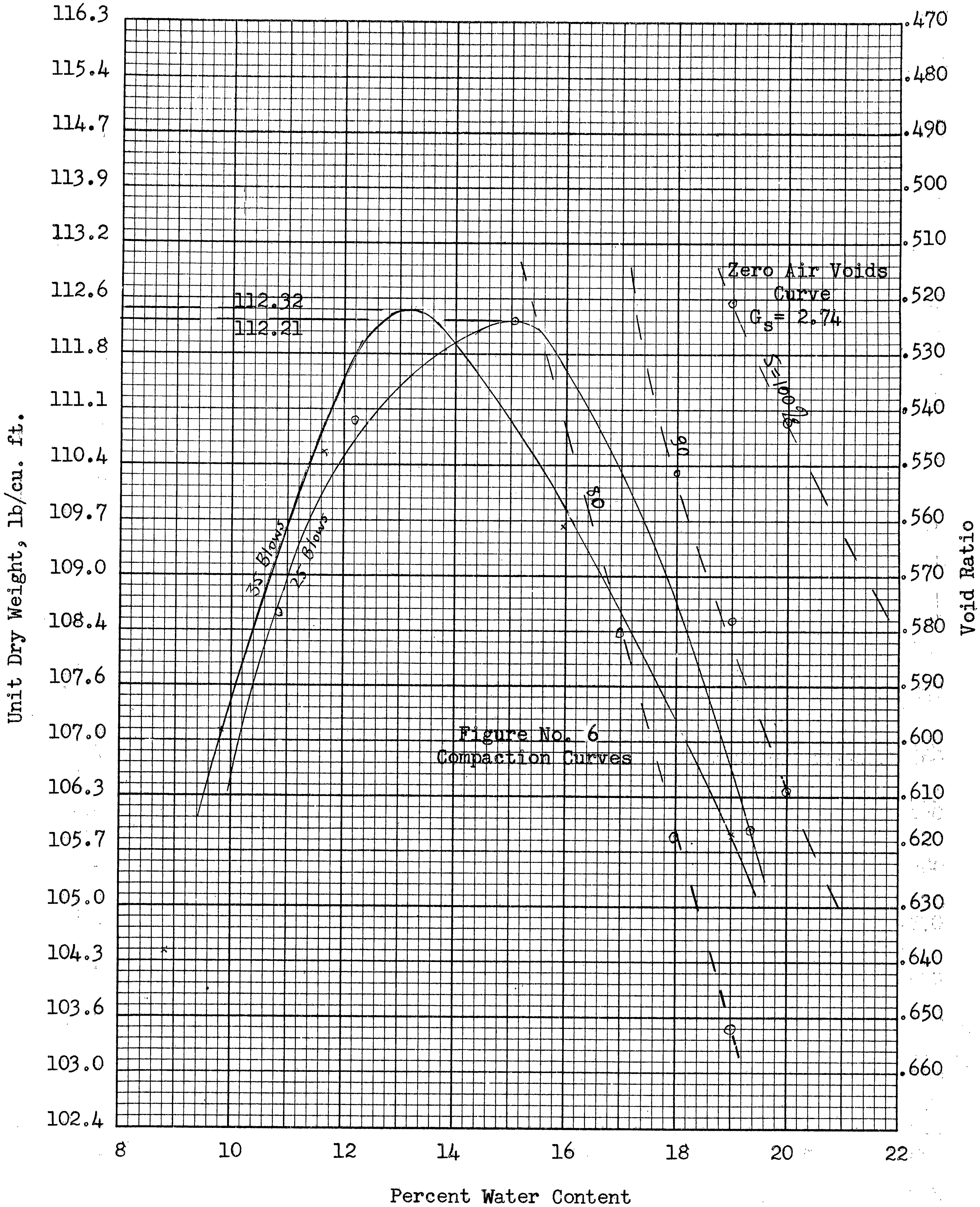


Figure No. 6
Compaction Curves

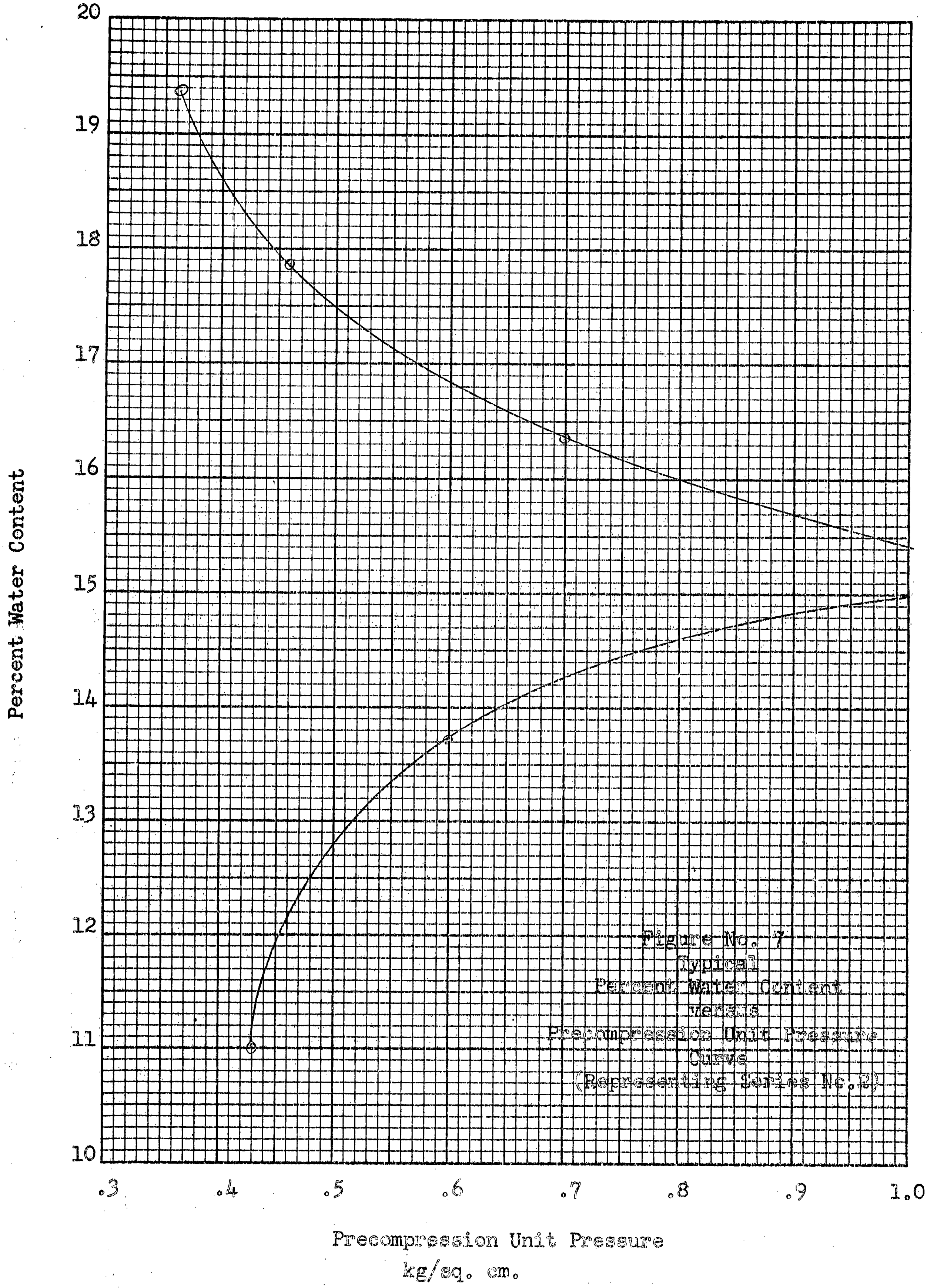
TABLE I

BASIC EXPERIMENTAL DATA

Sample No.	Compactive Effort	Initial Water Content, w_o	Initial Void Ratio, e_o	Initial Unit Pressure, p_s kg/sq. cm.	Void Ratio Under p_s, e_s
1 - 2	25 blows	9.13	0.624	0.0373	0.6598
1 - 1	" "	10.96	0.594	"	0.6721
1 - 3	" "	13.80	0.555	"	0.5709
1 - 4	" "	16.35	0.588	0.0745	0.5948
1 - 5	" "	17.20	0.588	"	0.6029
2 - 1	" "	11.02	0.560	0.0745	0.5912
2 - 2	" "	13.71	0.534	"	0.5519
2 - 3	" "	18.53	0.602	"	0.6221
2 - 4	" "	17.87	0.656	"	0.6658
2 - 5	" "	19.38	0.616	"	0.6124
3 - 1	" "	10.90	0.577	0.1496	0.5851
3 - 2	" "	11.80	0.617	"	0.6211
3 - 3	" "	15.10	0.524	"	0.5240
3 - 4	" "	18.80	0.626	"	0.6217
3 - 5	" "	21.40	0.707	"	0.6927
4 - 2	" "	10.80	0.627	0.2963	0.6265
4 - 1	" "	12.20	0.542	"	0.5535
4 - 3	" "	14.20	0.616	"	0.6025
4 - 4	" "	15.70	0.645	"	0.6313
5 - 1	35 blows	8.88	0.638	0.0373	0.6940
5 - 2	" "	11.65	0.548	"	0.5619
5 - 4	" "	14.55	0.545	"	0.5599
5 - 3	" "	15.97	0.561	"	0.5683
5 - 5	" "	20.18	0.658	"	0.6624
6 - 1	" "	9.86	0.598	0.2963	0.5909
6 - 3	" "	14.05	0.564	"	0.5642
6 - 2	" "	15.93	0.569	"	0.5717
6 - 4	" "	19.00	0.617	"	0.6151
6 - 5	" "	20.00	0.711	"	0.6856

TABLE I (Continued)

Coefficient of Compressibility, C_c	Coefficient of Swelling, C_s	Precompression Unit Pressure, casagrande style P_p kg/sq. cm.	Unit Pressure Corresponding to e_s, p_{e_s} kg/sq. cm.	Total Deformation Under Initial Load +=Swelling cm.	Unit Positive Volume Change, ε_s cm/cm
0.190	0.0150	0.300	0.285	+ 0.0779	0.03050
0.180	0.0280	0.290	0.280	+ 0.1051	0.04120
0.156	0.0240	0.600	0.450 560	+ 0.0266	0.01043
0.163	0.0240	0.700	0.570	+ 0.0164	0.00662
0.157	0.0170	0.510	0.390	+ 0.0056	0.00219
0.164	0.0260	0.430	0.326	+ 0.0088	0.00345
0.139	0.0220	0.600	0.470	+ 0.0112	0.00436
0.172	0.0280	0.850	0.720	+ 0.0016	0.00063
0.183	0.0300	0.460	0.400	+ 0.0057	0.00228
0.136	0.0273	0.364	0.330	+ 0.0010	0.00039
0.1560	0.0315	0.940	0.840	+ 0.0056	0.00221
0.1747	0.0278	0.430	0.370	+ 0.0061	0.00239
0.1373	0.0206	1.080	0.680	+ 0.0044	0.00172
0.1708	0.0235	0.790	0.580	- 0.0001	0.00000
0.1938	0.0265	0.415	0.327	+ 0.0005	0.00020
0.1707	0.0218	0.510	0.480	+ 0.0019	0.00074
0.1460	0.0207	0.870	0.670	+ 0.0044	0.00172
0.1667	0.0164	0.670	0.560	+ 0.0020	0.00078
0.1653	0.0188	0.590	0.550	0.0000	0.00000
0.1680	0.0270	0.260	0.240	+ 0.0118	0.00463
0.1480	0.0305	0.490	0.440	+ 0.0239	0.00937
0.1317	0.0240	0.675	0.560, 450	+ 0.0104	0.00419
0.1560	0.0242	0.910	0.820	+ 0.0066	0.00258
0.1762	0.0320	0.620	0.510	+ 0.0071	0.00277
0.1625	0.0208	0.650	0.630	+ 0.0042	0.00164
0.1431	0.0215	0.890	0.825	+ 0.0044	0.00172
0.1470	0.0238	0.950	0.860 .68	+ 0.0031	0.00121
0.1605	0.0160	0.760	0.640	+ 0.0002	0.00008
0.1855	0.0240			+ 0.0001	0.00004



20
19
18
17
16
15
14
13
12
11
10
Percent Water Content
.3
.4
.5
.6
.7
.8
.9
1.0
Precompression Unit Pressure
kg/sq. cm.
Figure No. 7
Typical
Percent Water Content
versus
Precompression Unit Pressure
Curve
(Representing Series No.2)

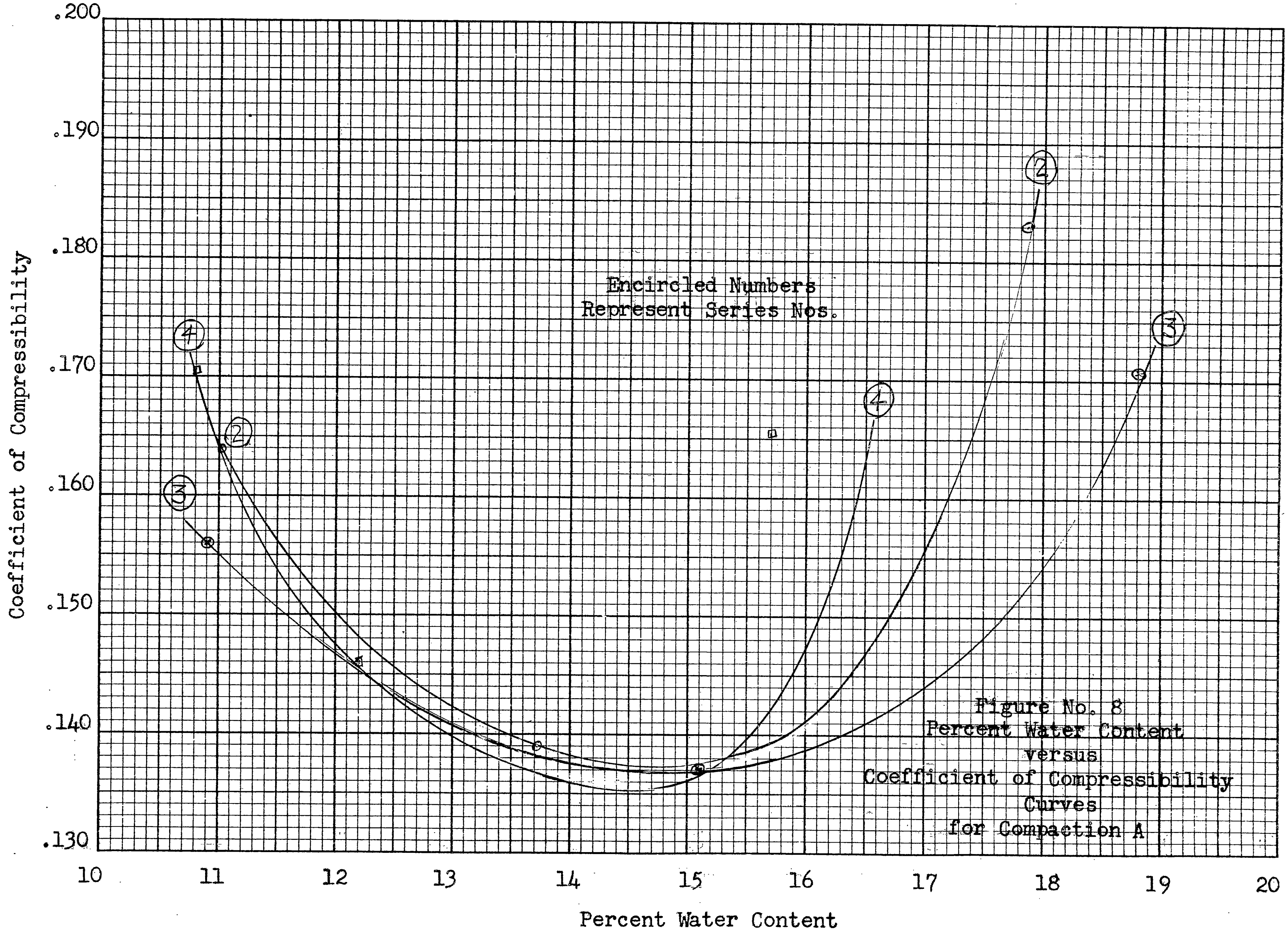


Figure No. 8
Percent Water Content
versus
Coefficient of Compressibility
Curves
for Compaction A

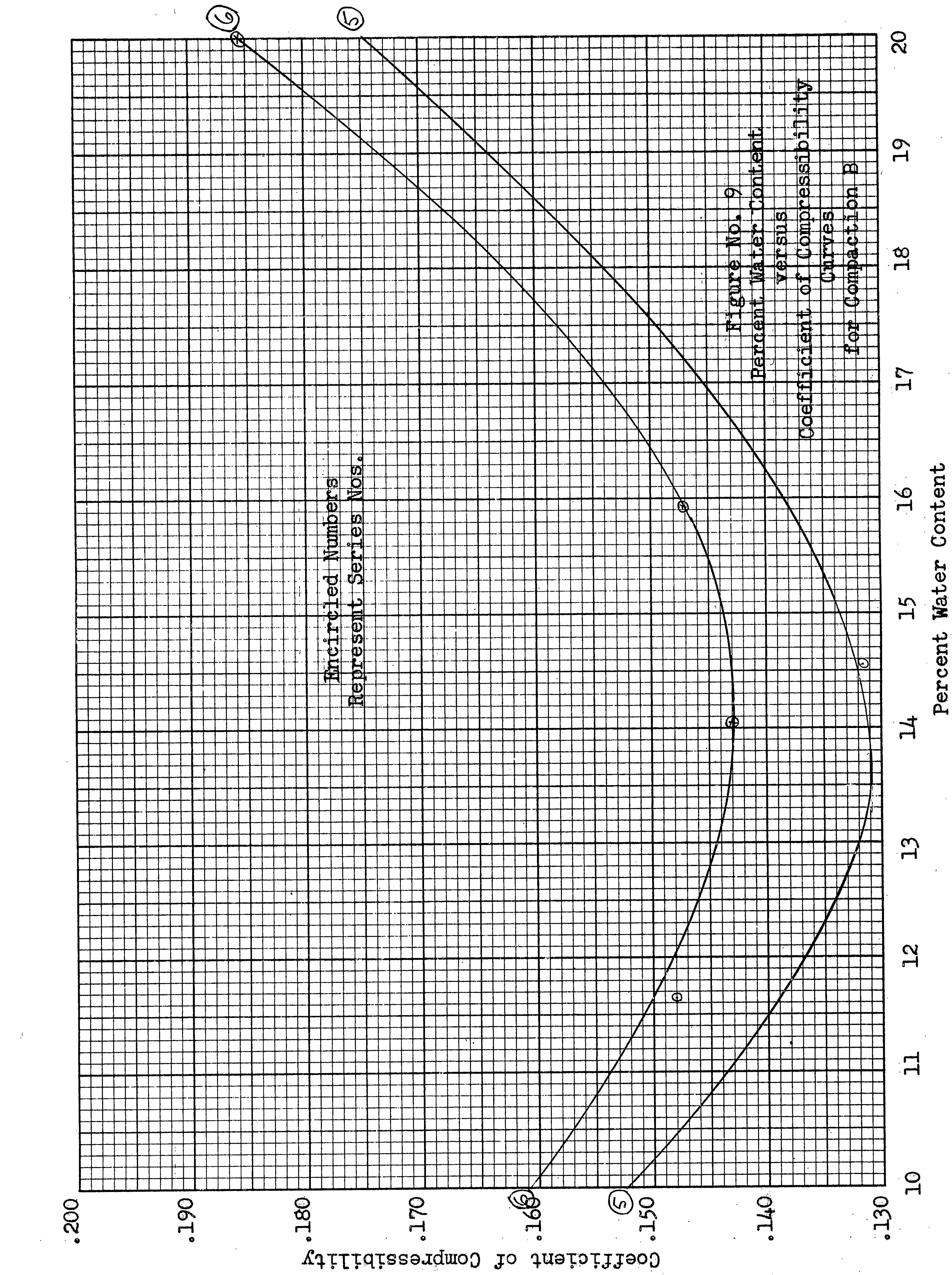


Figure No. 9
Percent Water Content
versus
Coefficient of Compressibility
Curves
for Compaction B
Encircled Numbers
Represent Series Nos.
6
5
Coefficient of Compressibility
.200
.190
.180
.170
.160
.150
.140
.130
Percent Water Content
10
11
12
13
14
15
16
17
18
19
20

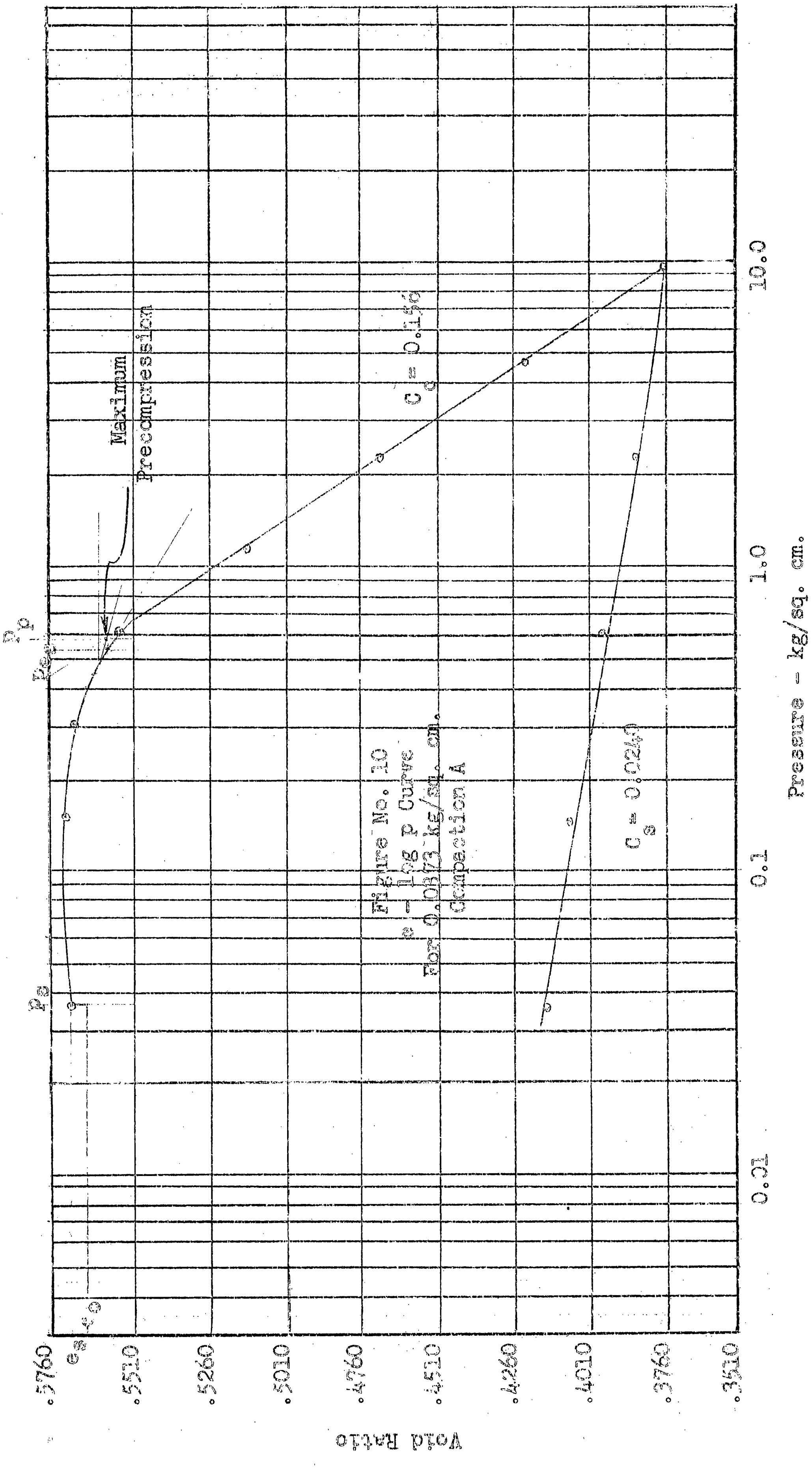


Figure No. 10
e - log p Curve
For 0.0873 kg/sq. cm.
Compaction A
Maximum
Precompression
C_c = 0.156
C_s = 0.0240
P_p
P_e
P_s
e_s
e_o
Pressure - kg/sq. cm.
Void Ratio
0.01
0.1
1.0
10.0
.5760
.5510
.5260
.5010
.4760
.4510
.4260
.4010
.3760
.3510

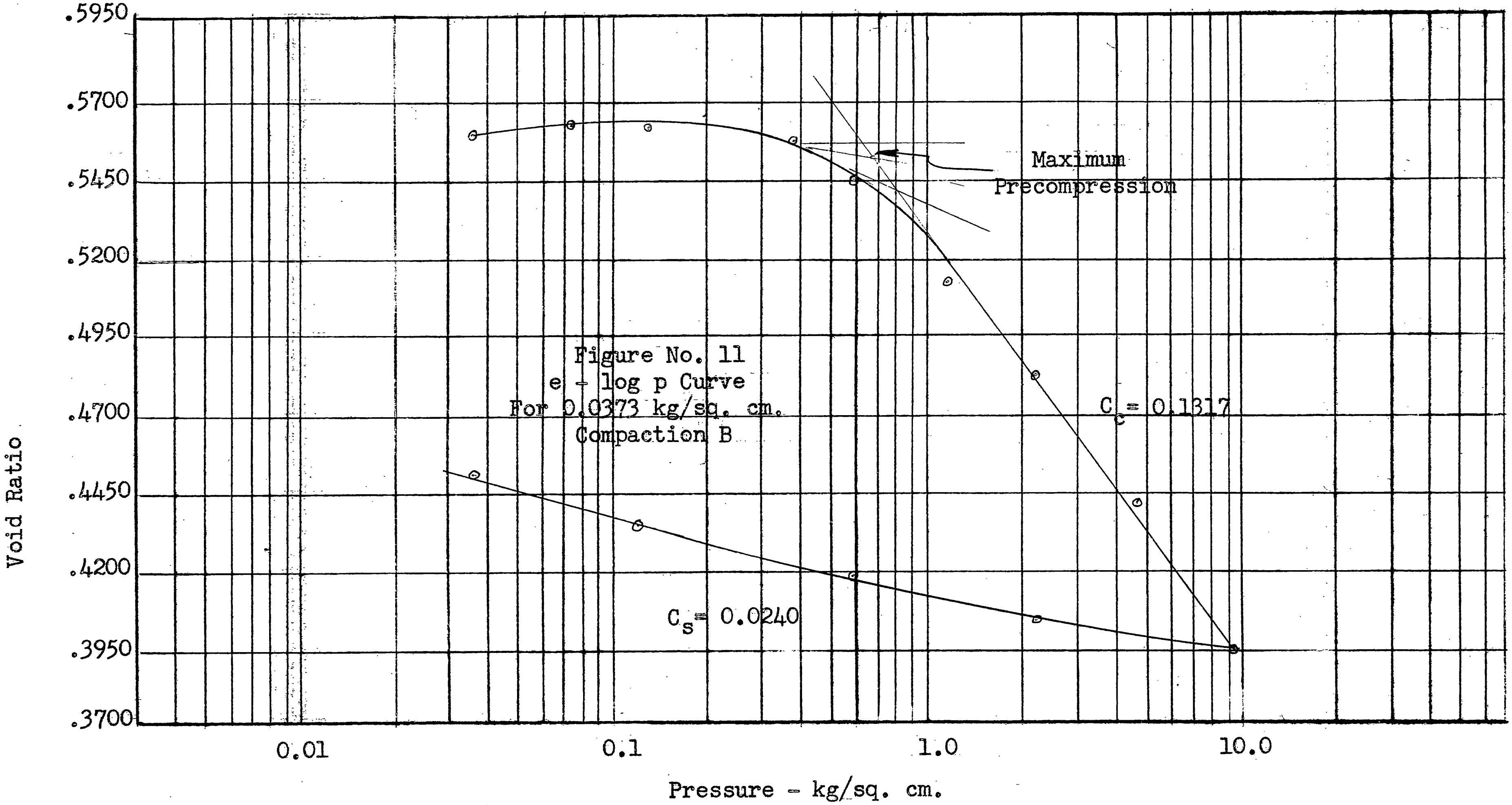


Figure No. 11
e - log p Curve
For 0.0373 kg/sq. cm.
Compaction B

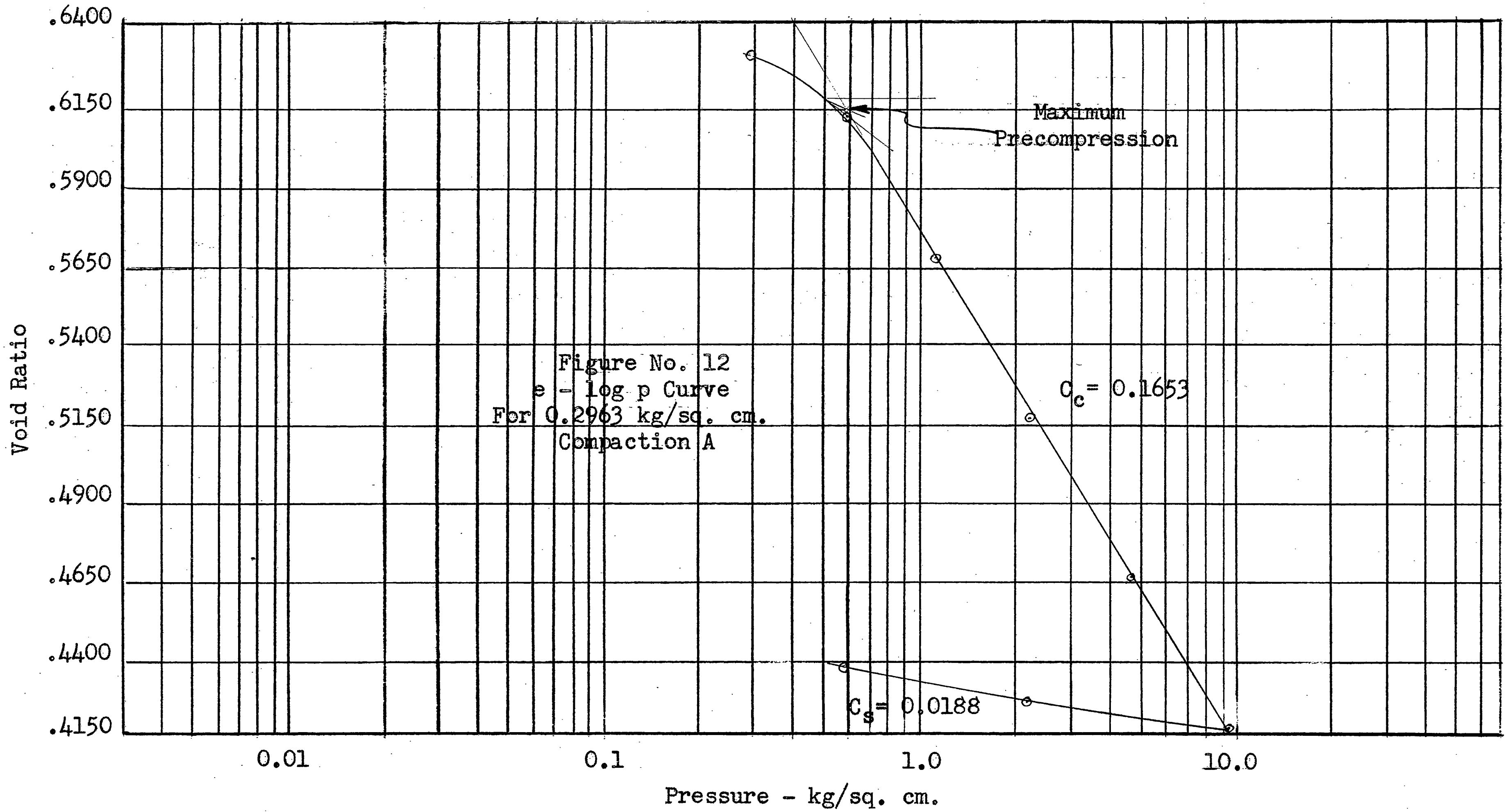


Figure No. 12
e - log p Curve
For 0.2963 kg/sq. cm.
Compaction A

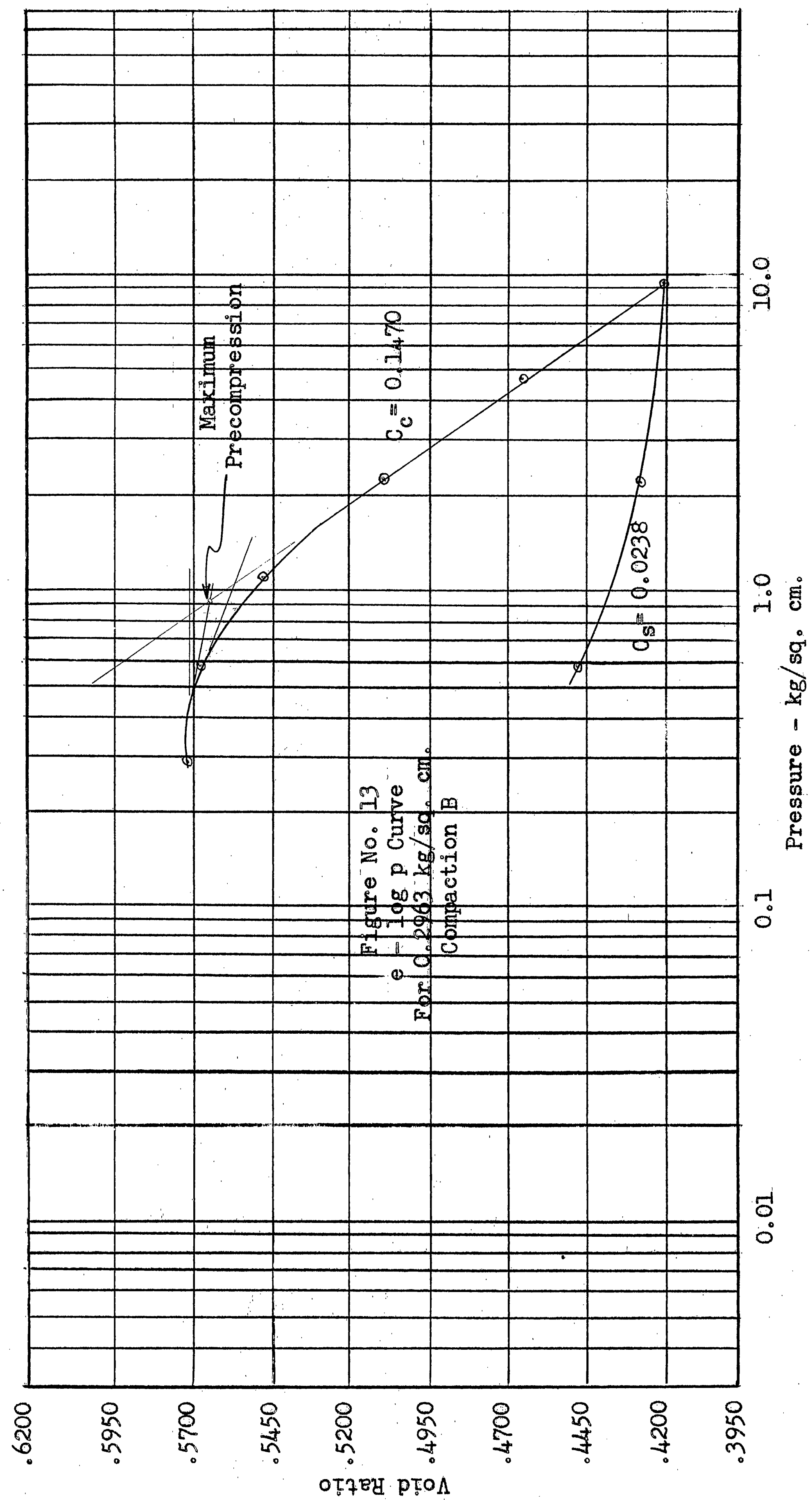


Figure No. 13
e - log p Curve
For 0.2963 kg/sq. cm.
Compaction B

After plotting the unit positive volume change versus the initial void ratio of the sample it was established that there is no relationship between the two. A layer of this clay can swell irrespective of the initial void ratio to which it was compacted. Consequently, no use of the void ratio values with respect to the swelling characteristics of this clay will be made in this report.

Plots of percent water content versus unit deformation (swelling) appear for compaction A in Figure 14 and for compaction B in Figure 15. From the curves of the different pressures imposed upon the samples the following characteristic relationships are deduced:

For both compaction efforts the greatest unit deformation occurred under the smallest pressure; with greater pressures much smaller deformations were recorded.

The least swelling that occurred related to the optimum moisture content is tabulated in Table II for both compactive efforts. It will be noticed that in order to record the least swelling the samples should have been compacted well above the optimum. The greater the pressures the clay is to carry the closer to the optimum it should be compacted. ?

Table III shows the unit positive volume change recorded under the pressures used in these tests for a range of water content between 11% and 18% with compactive effort A. The same data referring to compaction B are presented in Table IV. Table V indicates the maximum percent swelling for the two compactive efforts used. This table reveals that with compactive effort B the swelling experienced was greater in magnitude for the higher pressures than is the case for the smaller compactive effort A. This is not the case when smaller overburden pressures are involved. Swelling under light loads was less for compactive effort B. interesting

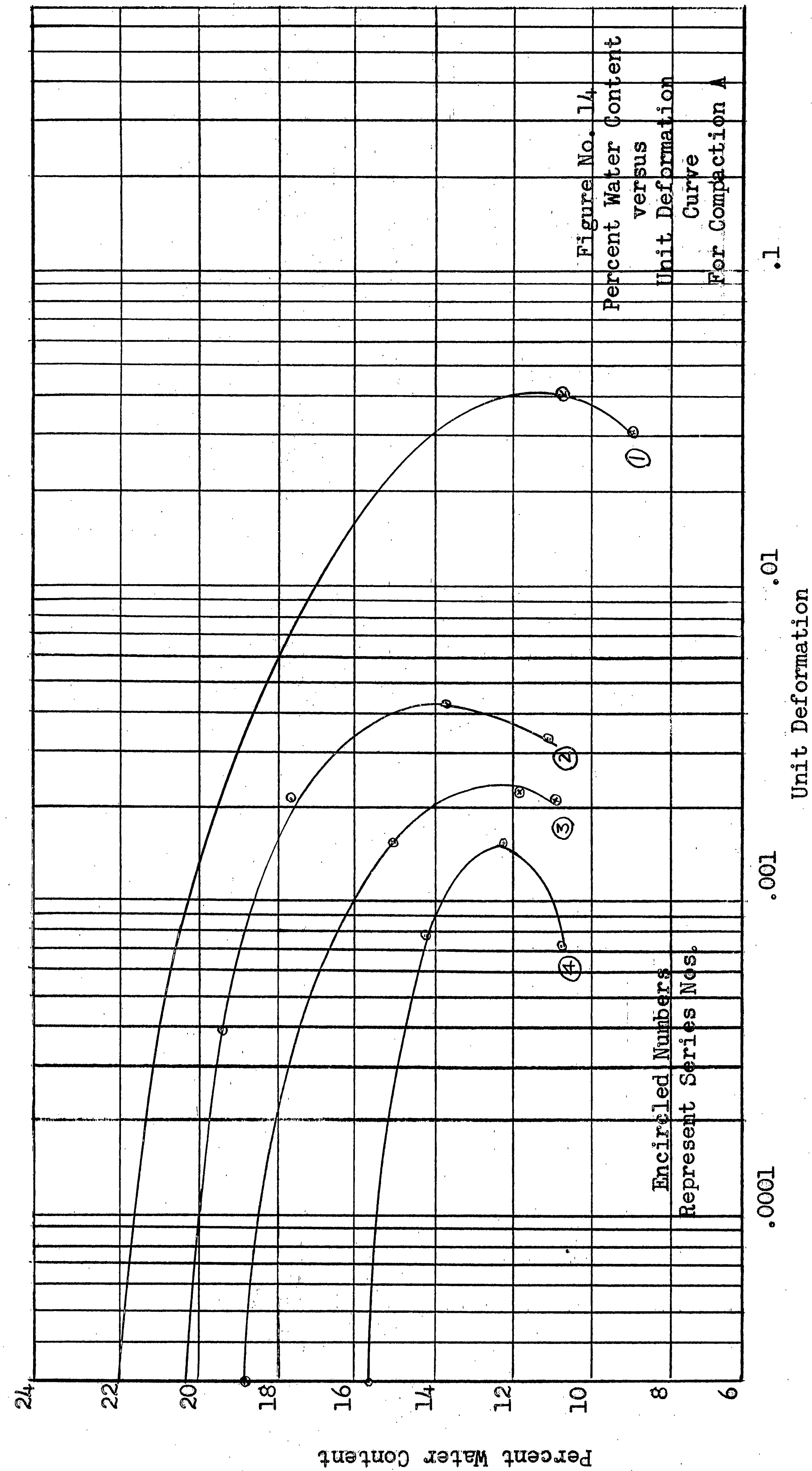


Figure No. 14
Percent Water Content
versus
Unit Deformation
Curve
For Compaction A

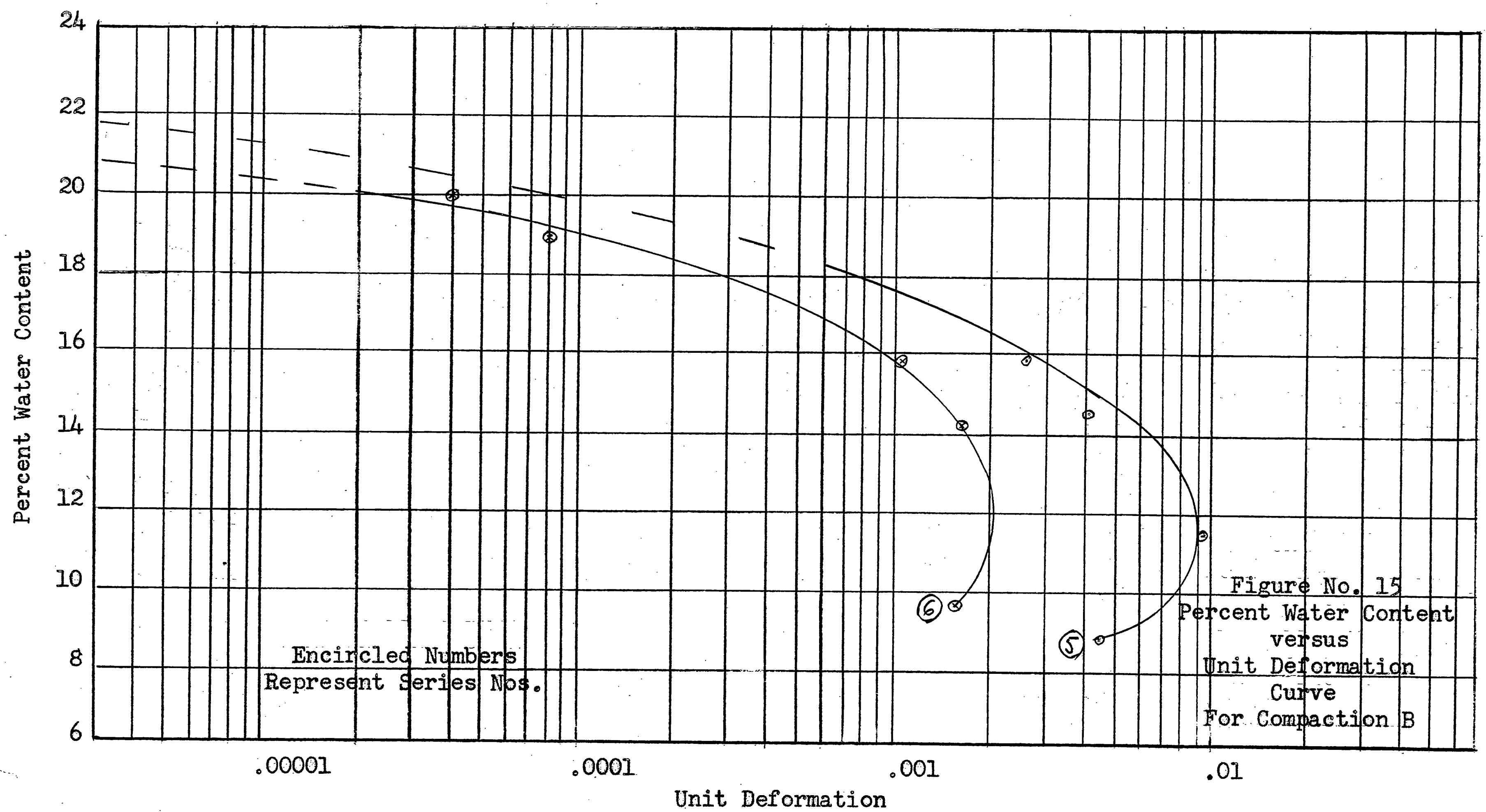


Figure No. 15
Percent Water Content
versus
Unit Deformation
Curve
For Compaction B

TABLE II

COMPACTION FOR LEAST SWELLING RELATED TO OPTIMUM MOISTURE CONTENT

Unit Pressure Imposed, tons/sq. ft.	Water Content for Least Swelling, %		Percent Above Optimum Moisture Content	
	Compaction A	Compaction B	Compaction A	Compaction B
0.2963	15.7	20.3	4	45
0.1496	18.8		25	
0.0743	20.1		34	
0.0375	22.0	21.1	46	50

TABLE V

MAXIMUM PERCENT SWELLING UNDER VARIOUS INITIAL LOADS

Compaction	Initial Load			
	3 kg.	6 kg.	12 kg.	24 kg.
A	4.0	0.44	0.24	0.16
B	0.9			0.19

TABLE III

SWELLING UNDER VARIOUS INITIAL LOADS
WITH DIFFERENT WATER CONTENTS
FOR COMPACTION A

Water Content %	Initial Load 3 kg. cm.	6 kg. cm.	12 kg. cm.	24 kg. cm.
11	0.0400	0.00340	0.00226	0.00091
12	0.0310	0.00390	0.00238	0.00166
13	0.0185	0.00425	0.00242	0.00145
14	0.0290	0.00435	0.00230	0.00089
15	0.0220	0.00427	0.00185	0.00036
16	0.0155	0.00385	0.00105	
17	0.0098	0.00310	0.00050	
18	0.0055	0.00195	0.00021	

TABLE IV

SWELLING UNDER VARIOUS INITIAL LOADS
WITH DIFFERENT WATER CONTENTS
FOR COMPACTION B

Water Content %	Initial Load 3 kg. cm.	24 kg. cm.
11	0.00900	0.00184
12	0.00880	0.00190
13	0.00760	0.00187
14	0.00570	0.00174
15	0.00395	0.00153
16	0.00240	0.00115
17	0.00127	0.00068
18	0.00060	0.00033

It can be seen from Table I that in the case of the smallest unit pressure, 0.0375 kg/sq. cm., smaller unit deformation values as a whole were obtained with an increased compactive effort; whereas where greater unit pressures are involved increased compactive effort gave greater values for unit positive volume change.

As a result of this investigation the following figures are presented for practical considerations:

Figure 16 representing a plot of unit pressure - unit deformation for compactive effort A is shown on page 59. Given a certain overburden pressure and with that much compaction the water content can be readily determined which will give the least swelling. Figure 17 corresponds to the same variables as Figure 16 but applies to compaction effort B.

As illustrated in Figure 17 there is no overlap of the water content curves as is shown in Figure 16. Knowing the optimum moisture content of a Permian clay and the unit load to be carried by this clay, with the aid of Figure 18 one can determine, according to the compaction to be used, the percentage of water content above the optimum which should be used for compaction for the least swelling.

As already explained, the physical properties of clays are affected by their mineral content; so, the results and conclusions offered in this report are valid only for clays having physical properties nearly the same as the ones of the clay tested.

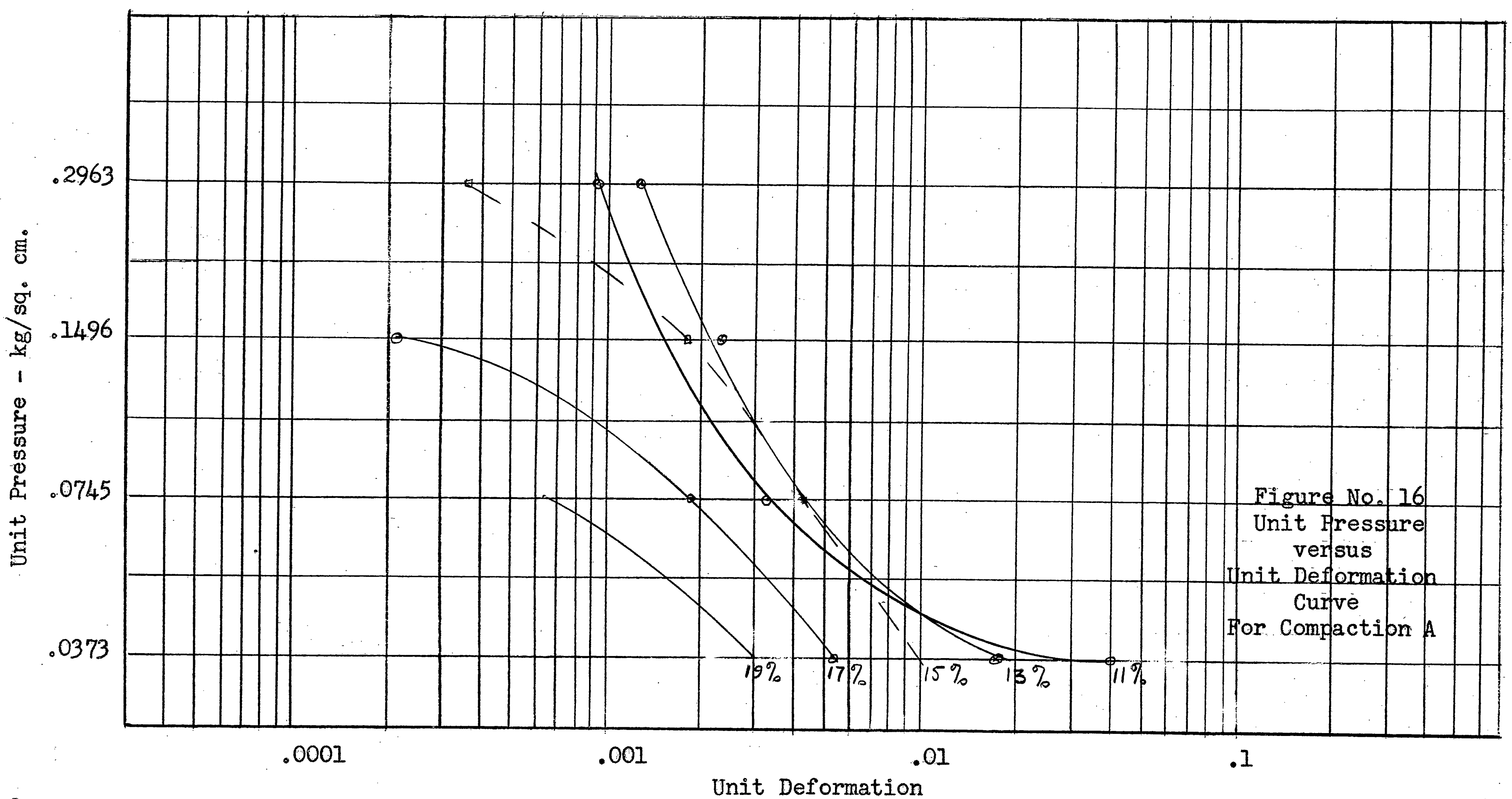


Figure No. 16
Unit Pressure
versus
Unit Deformation
Curve
For Compaction A

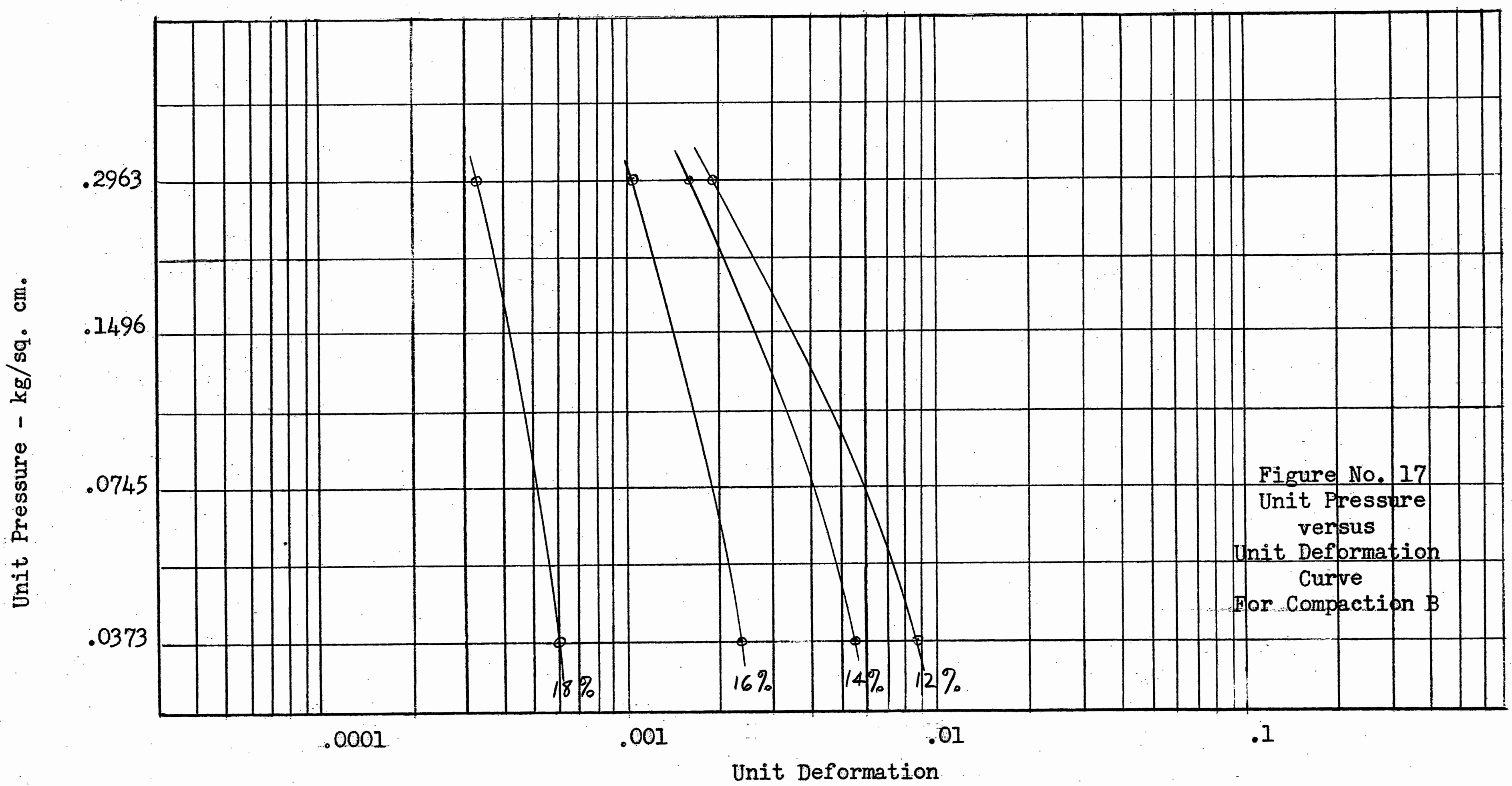


Figure No. 17
Unit Pressure
versus
Unit Deformation
Curve
For Compaction B

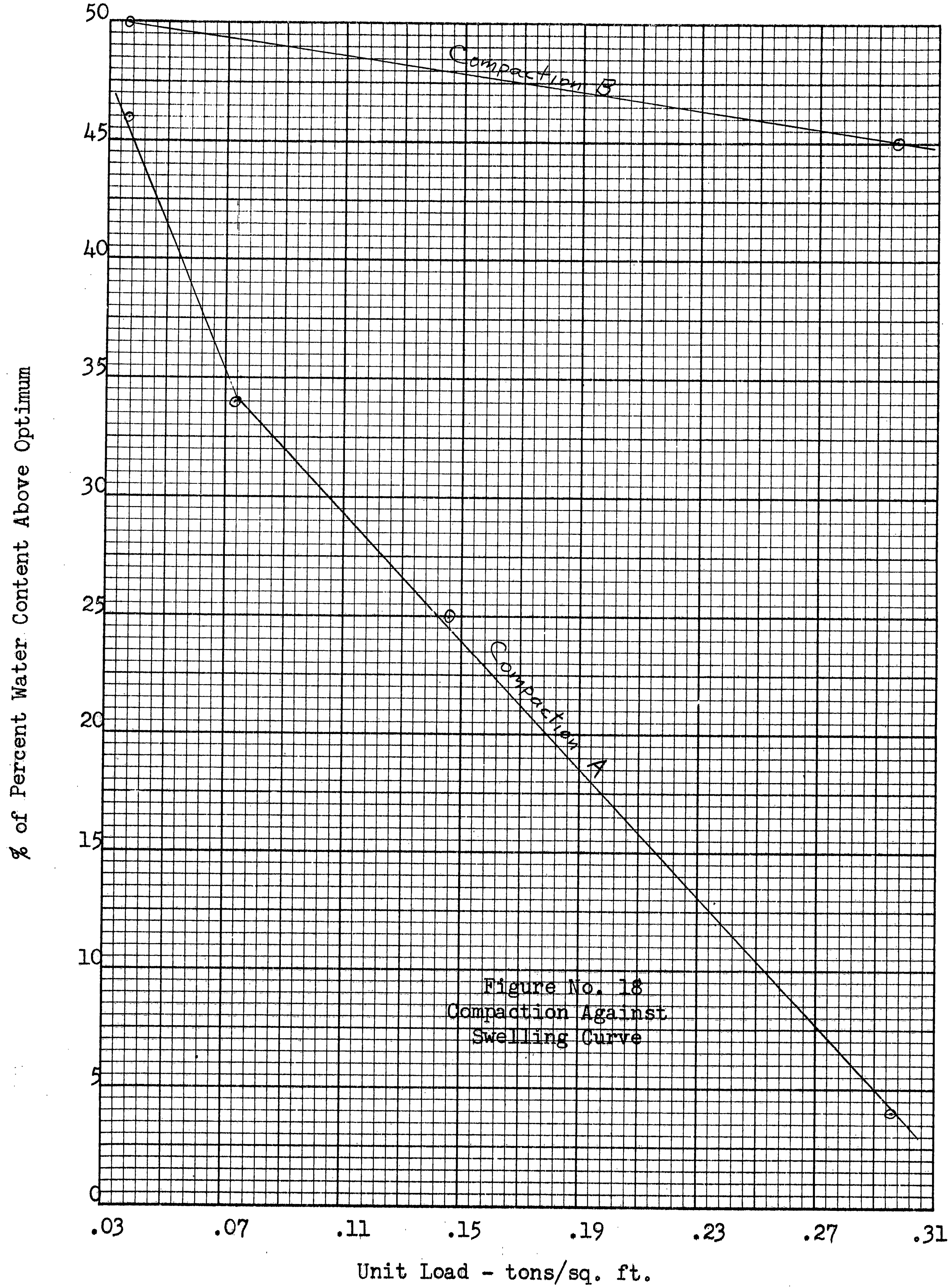


Figure No. 18
Compaction Against
Swelling Curve

CHAPTER VI

SUMMARY AND CONCLUSIONS

An attempt will be made to compare conditions affecting laboratory samples and field conditions.

Assume a certain job for which this Permian clay is to be used in a remolded state. It is known that this clay is preconsolidated with loads much greater than the one it is intended to support. For compaction purposes the clay is sprinkled if it is in a relatively dry state; otherwise it should be dried to around the optimum condition. The question is at what moisture content this clay should be compacted to safeguard the superstructure against swelling. The only constant involved is the design unit pressure that this clay is meant to support.

In the laboratory various consolidation samples were compacted at different initial water contents. These water contents represent the percent moisture at which this material will be compacted in the field. Immediately thereafter the sample was loaded with a certain load and was let to set for approximately twelve hours. This period was not uniform for all samples. Also the degree to which the porous stones touching the sample were impregnated with water was not controlled. But both these variable conditions, it is felt, rendered no sizable effects in the whole process. If some moisture should be transferred from the wet porous stones to the sample, the time allowed for each sample before flooding was too short for the moisture to penetrate any measurable distance into the sample.

The degree of this penetration, of course, depends on the relative initial moisture of the porous stones and the sample. This effect would partly be minimized by an initial pressure on the sample when contact was attempted between the consolidation machine and the porous stones, before the sample was loaded. These variables do not exist in the field.

In certain cases in the field the compacted soil might not be loaded immediately after compaction. Following this condition one sample compacted at above the optimum moisture content started swelling. This one case cannot be generalized, however. More research is needed in this connection.

The swelling would occur after water finds its way through the soil. Thus the sample was flooded, and, under the already existing load, it started swelling. In the field a fill after compaction might lose some of its moisture before it will be affected by the swelling factor, the percolating water. This constitutes a serious discrepancy as to the application of the results of this study for the direct control of swelling on compacted fills. The extent to which this discrepancy affects the suggestions to be offered in this report depends on the thickness of the fill, the permeability of the material, and the climatic conditions of the place. Imagination and judgement should be exercised in this case to alleviate the results of this deviation.

Another possibility is that a fill of this clay not fully saturated might settle under its overburden before any swelling occurs. This condition is not investigated in this study. In other words, the flooding of the sample in the laboratory might not be exactly duplicated in nature. This is one variable to be counted for in the application of the conclusions of this study.

Two compactive efforts were used on various samples. Compaction A involving twenty-five blows per compaction layer, and compaction B with thirty-five blows. In the field the corresponding heavier compaction can be brought about either by the use of heavier equipment or by more passes of standard equipment.

From the measurements of the experimental positive volume changes (swelling) resulting from load-expansion tests on various samples of the Permian clay with water content, initial load, and compaction effort as variables, the following results are offered:

1. All samples swelled when compacted at optimum and under the precompression load.
2. Swelling decreased with increased initial water content under any load.
3. Swelling was greatest around the optimum moisture content and was minimum at a water content greater than the optimum.
4. For heavy loads (approximately 0.30 tons/sq. ft.) there was more swelling with greater compaction, and less swelling with greater compaction for small loads (approximately 0.04 tons/sq. ft.).
5. When compacted with same water content swelling was greater under the lighter loads.
6. Samples with nearly the same void ratio swelled less at greater water content.
7. Precompression load increased around the optimum moisture content.
8. Water content and not the void ratio affects swelling.
9. The coefficient of compressibility depends on the optimum moisture content.
10. The densest state does not produce the least swelling.

In general compacted fills of this clay under light loads, less than the precompression load (about 0.3 to 0.9 tons/sq. ft.), should be compacted at a moisture content well above the optimum. Figure 18 might prove to be useful in this respect. The maximum unit pressure used in this ivestigation was equivalent to 0.2963 tons/sq. ft.

The conclusions arrived at for the Permian clay agree in a sense to some of the conclusions from compaction studies of clayey sands, as far as highway construction is concerned, made at Vicksburg, Mississipi (3). The latter are quoted below:

- For this soil, at a given water content, the highest CBR is produced by the compactive effort for which the given water content is the optimum. Over-compaction reduced the CBR value.
- Increasing roller weights did not result in increased maximum densities.
- The maximum CBR of soaked statically compacted specimens occurred at molding water contents 1 to 3% higher than optimum.

The compaction requirements of fill material put out by the American Association of State Highway Officials, quoted on p.20, are very rigid. No specific restrictions for the compaction of fill material of this clay are included in the Oklahoma State Highway Specifications. According to the results of this investigation, for least swelling, fills of this clay should be compacted at above the optimum according to design loads, as indicated on Figure 18.

Pore pressures in thick layers of clay with poor drainage might prove detrimental to the stability of the structure involved by decreasing the shearing strength of the material. This happens in the case of earth dams. The great weight of the superimposed compacted clay fill causes considerable pore pressures due to the excess water that cannot escape rapidly from a large mass of fine-grained fill. In such cases due consideration should be given to the water content at which the fill material is compacted. If the clay investigated is to be used, it is suggested that at least the lower layers be compacted at optimum, if not on the "dry side" of the optimum, i.e. at a water content smaller by some two per cent than the optimum, and the upper layers can be compacted according to the recommendations offered in this report in order to eliminate excessive swelling of the clay.

This study also indicates the usefulness of the optimum moisture content of this clay. Its determination is an easy field operation. Compaction control, then, can be based on the optimum moisture content determination.

In the data presented the precompression unit pressure at a certain moisture content or void ratio that would produce no volume change under certain load was always greater than the imposed load. Compaction has produced a greater preconsolidation load than any used in these series; this is a fact established also by previous investigators.

Unfortunately this writing is confined to laboratory experimentations and only time will prove the accuracy of the conclusions drawn. It is understood by the author that Mr. Raymond E. Means, Professor of Architectural Engineering and his thesis adviser, has had already some experiences with fills used to support light weight structures. The compaction in those cases complied roughly with the conclusions derived in this report. But this investigation is the first to give statistical data on the controllable variables of this Permian clay for specific conditions.

Suggestions for Future Investigations

The need for more research would not cease to exist as long as this clay is causing inestimable damages to structures. As F. A. Marston puts it: The more experience one has in the examination and testing of clay soils, the more impressed one is with the need for research regarding the characteristics and behavior of those soils, composed entirely of very fine grains.(10)

The discussion of the conclusions made by this study makes this need more evident. Research should be channelled along the lines of various field conditions. For more and better applicability, research on Permian clay can be branched out to cover:

a. Volume changes under specific conditions

b. Effect of compaction below standard proctor

c. Effect of modified proctor compaction

d. Behavior under a number of cycles

e. Potential vertical rise by triaxial tests

f. Effect of loading increment on consolidation characteristics

g. Effects of size and thickness of samples

h. Secondary compression effects on undisturbed samples.

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VITA

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