SWELLING PRESSURES
OF A
PERMIAN CLAY

By

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PREFACE

This investigation is one of a series of investigations to establish swelling properties for Permian Clay.

The author wishes to express his indebtedness and gratitude to the following individuals and organizations:

To Professor J. V. Parcher for his valuable guidance and assistance in the preparation of this thesis and for acting as the author's major advisor;

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CHAPTER I

INTRODUCTION

GENERAL

The predominant geological soil formations of Oklahoma are the Permian and the Pennsylvanian (Figure 1). The Permian formation, often referred to as the "Permian Red Beds", is the one with which the soil investigations of this thesis are concerned. The clay from this formation, hereafter referred to as Permian Clay, is a marine clay. This clay was once covered by several hundred feet of overburden, since removed by erosion, and thus is also an overconsolidated clay. The mineral makeup of the Permian Clay varies due to differences in the relative proportions of illite, kaolinite and montmorillonite. The minerals illite and montmorillonite, both of which exibit high expansive properties, combine with heavy overconsolidation pressures to give Permian Clay in its undisturbed state a high swelling potential. The semi-arid climate of Oklahoma is another factor which aggravates the effects of swelling of the clay by providing cyclic periods of wetting and drying.

The economic importance of investigations aimed at determining the swelling properties of Permian Clay is evident to those who use this clay as a foundation for costly buildings and highways. If, for instance, a building is erected on Permian Clay without any thought given to the swelling potential of the clay, damage in future years to the building's walls and floors will be almost inevitable. If, however, the properties of swelling of Permian Clay are known and the building

Figure 3*

Geologic Map of Oklahoma

*Sheerar, Leonard Francis, The Clays and Shales of Oklahoma, p.77 , Oklahoma Engineering Experiment Station Publication No. 17, Oklahoma State University, Stillwater, 1932.

designed to allow for, or to keep to a minimum, differential swelling, the building may remain undamaged from swelling over its life span. It can be seen then that investigations similar to the one conducted in this thesis are not only of importance in the realm of basic knowledge but also in the realm of engineering economy.

PURPOSE OF INVESTIGATION

The general purpose of this thesis is to extend the information and data existing on the swelling properties of Permian Clay. The specific objectives are to:

1. Determine the validity of an assumption which has been made in order to predict the swelling pressure which an undisturbed desiccated clay will exert.
2. Determine the amount of swelling which will occur under different loads or confining pressures.
3. Determine the relationship between molding water content and swelling potential for remolded Permian Clay.

The data for this study is accrued from results of previous investigations carried out at Oklahoma State University (Table No. 1) and from the two test series of this investigation. Each of the test series for this investigation was composed of seven (7) soil samples. Series (A) is a test series on undistrubed clay and Series (B) is a test series on remolded clay.

CHAPTER II

THEORIES EXPLAINING VOLUME CHANGE OF CLAY

GENERAL

The physical properties of clay have through the years demanded the interest of man. Two general areas in which this can be observed are agriculture and soils engineering. The interest of the agriculturist in clay comes as a result of the desire to know why some soils can support vegetation abundantly and others can support little or none. The soils engineer on the other hand is interested in clay as a construction material. As a result of the different interests exhibited by these groups, different concepts of the properties of clay have been developed. The engineer, for example, has in the past developed his concepts of the properties of clay from physical observations. The agriculturist, however, aided by his research arm, the soil physicist, has formulated concepts from the physico-chemical nature of clay. Today, then, as background for the study of a particular property of clay such as swelling, concepts from both the physical and physico-chemical investigations of clay need to be reviewed. This is the purpose of this chapter.

The two physical concepts of clay expansion presented in this study as review are by A. Casagrande [(5)] and R. E. Means [(6)]. It should be noted that these concepts were developed for the purpose of providing mental tools with which to analyze the volume change of clay. The paper written by Casagrande was one of the first efforts made by an engineer to explain the volume change of marine clays.

The fact then that it is a relatively old concept does not mean that it is not still a useful concept. One can still use this concept to develop a feeling for the volume change which occurs in clay. The "Spring Analogy" used by R. E. Means to describe the effect of desiccation on clay also allows an engineer to do some mental reasoning about clays volumetric changes in semi-arid regions. At the time which each of these concepts were formulated, undoubtedly there was information about the physico-chemical properties of clay available, but these men as engineers were interested in a concept which could be of use in practical building problems. In fact, it has been in just recent years that soils engineers have attempted to explain the behavior of clay in terms of its physico-chemical properties.

The two physico-chemical concepts of clay expansion presented in this study as review are by G. H. Bolt (7) and T. W. Lambe (8). Bolt, a soil physicist, has developed a concept of swelling by making use of recent developments in investigational apparatus such as the electron microscope, etc. He explains the actual swelling forces in terms of differential osmotic pressure potentials. Lambe has explained the swelling of clay in terms of the interactions between clay particles and mineral ions. The difference between the two concepts is basically in the explanation of the forces which cause clay to swell. Both concepts use the ionic concentrations surrounding the clay minerals as the starting point of the explanation.

A. CASAGRANDE'S EXPLANATION

During the process of sedimentation in fresh water, the silt and large clay particles settle out individually, while clay particles of colloidal size first form flocks which are large enough to overcome the effect of Brownian Movement and then to settle out. In sea water the coagulating effect of the salt is so strong that it causes, also, clay particles of larger size and even silt and small sand grains to form flocks during their downward movement.

When these flocks and individual grains reach the bottom they form a very loose honeycomb structure. They do not settle into a more stable position, but usually remain in that position in which they first touched the bottom. This behavior is due to molecular attraction at the points of contact, which, for grains of this small size, is of the same order of magnitude as the weight of the grains themselves.

Under the steadily growing weight of the overlying sediment consolidation takes place, which means that water is being squeezed out, and the structure becomes more dense. The flocculent structure of the clay in the space where the silt grains are relatively near together should for geometrical reasons undergo a larger compression per unit of volume than the clay contained in the spaces between the grains. A sudden increase in pressure would cause lateral flow and produce the same degree of compression of the clay in the larger voids as in the narrow spaces between the larger grains. However, a very slow increase in pressure allows sufficient time for a higher degree of consolidation to occur in the narrow space without lateral flow of the clay. Consequently, the void ratio of the clay in the narrow spaces will decrease more than in the larger voids, and the pressure will be carried mainly by the structure of the larger grains, being transferred through the highly consolidated clay in the narrow spaces between the grains. That part of the clay which fills the space within the individual arches of the strucutre will carry only a very small fraction of the pressure and therefore will remain in a very soft state, while the consolidated "bond clay" between the grains will carry a unit pressure much higher than the average pressure over the whole area.

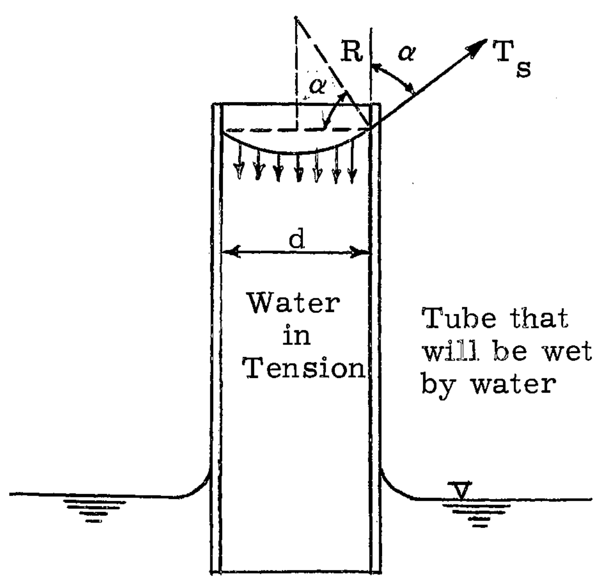
On the basis of the above explanation, the difference in the physical properties between the undisturbed and remolded states is easily understood. Remolding destroys the bond of highly consolidated clay between the silt grains and causes the soft clay filler to surround the larger grains which have formed the structure. As a result, the consistency in the remolded state will be very soft - in extreme cases even about the liquid limit.

From the standpoint of the sampling and testing operations, it is important to know what happens within the clay when the external load is removed. Just as a spring or a sponge which is compressed will expand again when the load is removed, clay will slowly expand or swell when it is taken from the ground and immersed in water. However, when kept away from water it cannot expand because the surface tension of the water in the microscopic voids is large enough to prevent any expansion. Thus, we should expect that the internal stresses in an undisturbed sample, immediately after it is taken from the drill-hole, do not change as long as there is no volume increase due to swelling. But even then, if no external water is available which the clay could absorb, following its tendency to swell, there is one possibility of a gradual reduction of the internal stresses by "internal swelling." The highly consolidated "bond clay" tends to expand with a greater force than the soft clay filler within the structure. In other words, there is a tendency for the structure to deform in such a manner that the total volume of the material remains unchanged, the "bond clay" expanding by drawing water from the soft filler. Such a deformation within narrow limits is physically very well possible, and there appears no reason why this internal swelling should not take place.

R. E. MEAN'S EXPLANATION

Capillarity. Since a portion of the deformation of clay is elastic, there is a partial rebound after a load which is applied for the first time is removed. When the same pressure is applied producing a recompression of the clay, the deformation is almost entirely elastic and the full elastic recovery is attained after removal of the pressure.

When a tube of small diameter of a material that will be wet by water and is open at both ends is inserted vertically into a pool of water, the water will form a meniscus inside the tube and climb the sides of the tube as illustrated in Figure 2. The water is attracted by the material in the tube in the same manner as the water is attached to a soil particle and tries to wet as much of the surface as possible. A pull is exerted at the junction of the surface of the water and the wall of the tube. The molecules of water also attract each other like tiny magnets so that they hang together to form a sort of



Capillary Tube
Figure No. 2

inverted dome supported by the walls of the tube. The water in the tube is suspended by this attraction from the inverted dome or meniscus. If the attraction between the material of the tube and the water is greater than the attraction between molecules of water, a semispherical meniscus forms and the maximum pull or lift is exerted. The relation between the tension in the water and the diameter of the tube or the radius of the meniscus can be determined by equating the vertical component of the force at the junction of the tube and the meniscus and the weight of the column of water hanging from the meniscus.

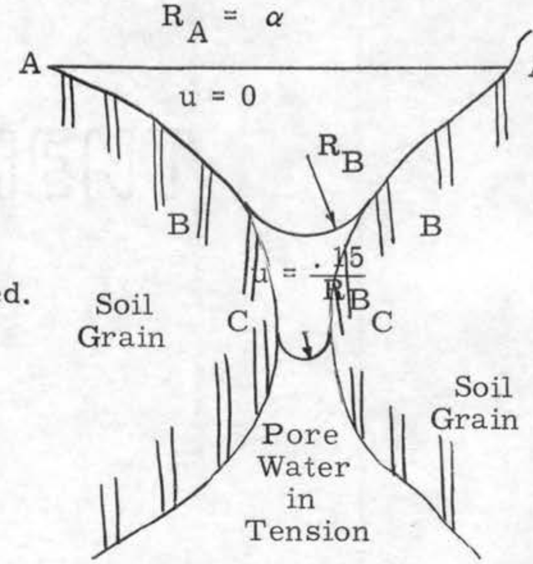
$$\pi d T_s \cos\alpha = \frac{\mu \pi d^2}{4} \quad : \quad \frac{d}{2} = R\cos\alpha$$

$\mu = \frac{2T_s}{R}$ = tension in water at the top of the column.

T_s = surface tension = 0.076 gm. cm.$^{-1}$.

The maximum tension in a tube of diameter (d) will be produced when the meniscus is fully developed and $R = \frac{d}{2}$.

The spaces between the small clay particles form capillary tubes. When the pores of clay are completely filled with water and there is free water on the surface as at (A) in Figure 3, there exists no tension in the water and no compression in the soil grains. Evaporation forms menisci in the pores near the surface as at (B) in the figure and tension in the pore water becomes $\mu = \frac{0.152}{R_B}$. This tension is transmitted throughout the connected pores to all the water in the soil mass making the radius of the menisci in all pores at the surface equal to R_B. The pores at the surface are subjected to a compressive stress equal to the total stress produced by the stressed pore water. This load on the soil grains produces a deformation or shrinkage of the clay in the same manner as though this stress were applied from a superimposed load. Further evaporation causes a receding of the menisci to the smallest portion of the largest connecting pore as at (C) in Figure 3. At this stage the greatest pore water tension and corresponding compression in the soil will be developed. The clay will have attained its maximum shrinkage. At this shrinkage limit the clay will be saturated.

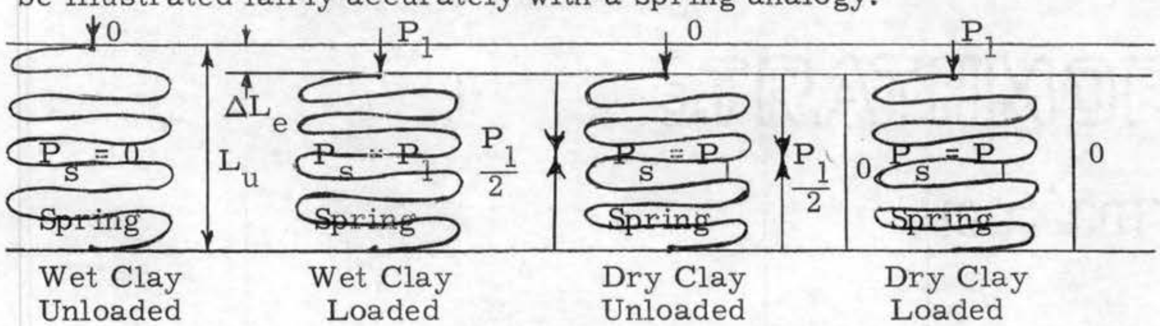


Capillary Tube in Soil

Figure No. 3

When free water is supplied to the surface of the saturated clay at the shrinkage limit, the menisci on the surface are destroyed, the tension in the pore water is relieved, and the clay is free to swell through the elastic portion of the deformation caused by the pore water stress.

<u>Spring Analogy.</u> Whether the principle contribution to shrinkage is the evaporation of attached water, or forcing out the attached water by pressure produced by capillary water, or to bending of elastic particles by pressure, probably the action of desiccated clay can be illustrated fairly accurately with a spring analogy.



Elastic spring loaded with superimposed load
and with tie downs

Figure No. 4

Assume an elastic spring of unloaded length (Lu) in which a superimposed load (P_ℓ) produces a deformation (ΔL_e) as shown in Figure 4. If the spring be tied down with tension members having a modulus of elasticity many times that of the spring of length (Lu) such that the tension induced in the tension members due to ΔL_e of the spring causes no elongation, loading the tied down spring with a superimposed load less than (P_ℓ) causes practically no additional deformation of the spring. The tension in the wires is reduced and the compression in the spring remains approximately (P_ℓ). If the tie wires are cut when there is no load on the spring, it will expand through the deformation (ΔL_e). If a load (P_ℓ) is on the spring when the tie wires are cut, no rebound of the spring will occur. The greater the load on the spring when the wires are cut the less will be the elastic rebound.

Clay at the shrinkage limit may be considered as an elastic spring tied down by tensile stress in the capillary pore water. Applying free water to the surface of the clay eliminates the tension in the pore water and allows the clay to expand as water is drawn into the pores.

G. H. BOLT'S EXPLANATION

A relatively new theory concerning the swelling of clay has been presented by Dr. G. H. Bolt in a paper published in June 1956. In this paper, Bolt discusses and explains the swelling of clay. He points out the presence of intermolecular forces caused by the atomic structure of clay and water and also the presence of osmotic forces which are indicated to be the main cause of the swelling phenomenon. To describe Bolt's theory of swelling two conditions existing in a soil solution are discussed. One is the ionic attraction and distribution of ions around clay particles and the other is the osmotic potential caused by the concentration of ions near the clay particles.

Due to ionic substitutions in the crystal lattice, clay particles carry, in general, a negative charge. The exchangeable cations which accompany the clay particles in the liquid phase of the soil solution are attracted to the particle as a result of this negative charge. This attraction is opposed by the osmotic tendency of the ions to distribute themselves evenly throughout the system. As a result, a diffused type of distribution of ions is formed. Since the negative charge of the particle and the positive charge of the counter ion are separated, this system can be regarded as an electric double layer. In order to indicate the diffused character of the counter charge, the term "diffused double layer" is used. Although the osmotic pressure within the double layer decreases continuously with the distance from the solid surface of the clay particle, it can be proved that the effective osmotic pressure of the system is determined by the concentration of the ions in the central plane between the plates.* (The plates in this case being clay particles and the assumption being made that the clay particles are arranged in a parallel manner.) This follows from the consideration that the hydrostatic pressure-gradient force in the double layer, arising as a result of the gradient in the osmotic pressure, is balanced by the electric forces acting on the space charge formed by the counter ions. In the central plane the potential gradient (or electric field strength) is zero and, therefore, no electric forces are acting in this location. This implies that in a clay system, in equilibrium with an electrolyte solution situated outside the range of influence of the clay

* Verwey and Overbeek - 1948

plates, the hydrostatic pressure in the central plane acts as a net pressure pushing the plates apart. The "swelling pressure", therefore, equals the difference between the osmotic pressure in the central plane and the osmotic pressure in the equilibrium solution. In practice, such an equilibrium solution is provided by the solution which is pressed out upon consolidation of the clay specimen. Thus, the swelling pressure of the system is caused by the tendency of the liquid phase to re-enter the system.

T. W. LAMBE'S EXPLANATION

If a clay is subjected to an "all around" pressure with drainage permitted, it compresses; upon load release, it usually expands. These volume changes in clay are due to one or more of the five following components:

1) Gas-change of volume or amount
2) Particle Diminution
3) Particle Deformation
4) Particle Rearrangement
5) Change in Size of Micelle.

Gas can contribute to a volume decrease in a soil either by compressing, because of an increase in pressure, or by going into solution in the pore water because of a greater pressure or a lower temperature. On the other hand, it can contribute to a volume increase by expanding under a load release or by coming out of solution of the pore fluid when the pressure is reduced or the temperature raised. Volume changes of fine-grained soils caused by gas are complex and not well understood. This component will be excluded from the following discussion of volume change.

While Components 2 and 3 (particle diminution and particle deformation) undoubtedly occur in certain soils, they are probably insignificant in clays. The contact area in granular soils is so small that the usual stresses are large enough to crush unsound mineral (or rock) aggregates or to deform them elastically. The particles of such friable materials as caliche or weathered granite often decrease in size during laboratory or field compactions. The amount and type of contact area in clays, however, is such that particle crushing or elastic deformation from the stresses which the soil engineer would normally apply to the soil is probably not measurable.

Particle deformations can, however, make a significant contribution to volume changes in a soil composed of large plate-like particles. Loading and unloading a dry mass of sand-size mica flakes demonstrates this. To use this experiment to deduce the mechanism of clay compression is not wise. If a test sample of dry mica (muscovite) having silt- and clay-size particles was loaded and unloaded, leached with water, and finally loaded and unloaded again, the results would show 1) a marked decrease in sample volume upon contact with water and, 2) a larger rebound in the mica-water system than in the mica-air system. Expressed as a percentage of compression, the rebound of the mica-water is more than twice that of the mica-air.

Both of these actions are characteristics of colloidal behavior. The water permits the mica flakes to pack more densely than the air does. Because it is more polar than air, the water results in greater electrical repulsions between adjacent mica particles, i.e., lubricates the particles. Since there are greater interparticle repulsions when water is in the mica pores than when air is, the greater expansion upon load release is to be expected.

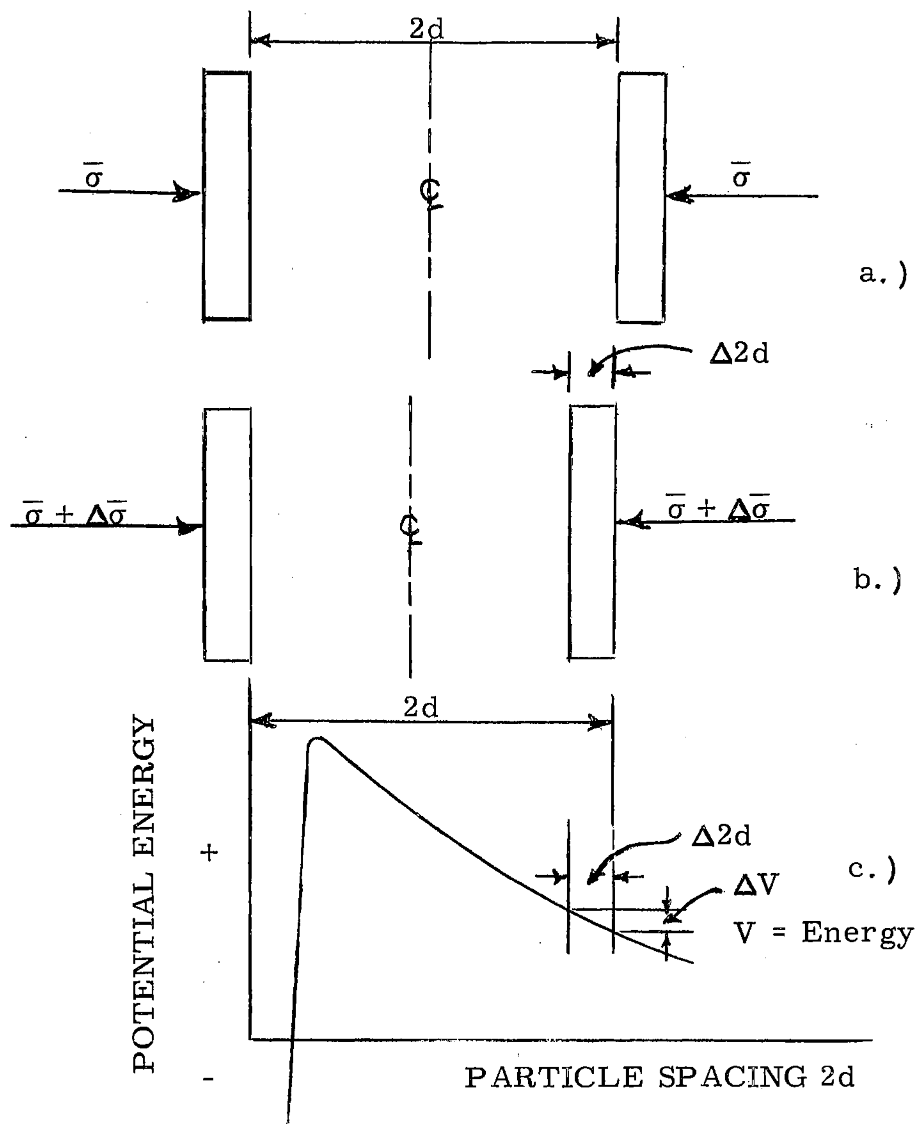
This compression test shows that merely reducing the particle size of the mica introduces colloidal phenomena. With clay particles (which have relatively large electrical surface potentials) colloidal phenomena are much more important than with the mica flakes (which have low potentials.)

The rearrangement of soil particles from a random array to a more orderly array contributes to compression. There are convincing data (Hvorslev, 1938; Mitchell, 1956) that one dimensional compression tends to align particles. The sudden decrease in volume, when the water permeated the mica sample is due to a rearrangement of mica particles. Volume decreases caused by the rearrangement of particles are essentially non-recoverable. When the volume increases from other causes, some minor changes, of course, occur in particle orientation.

In most clays Component 5, changes in size of soil micelle - i.e., soil mineral particle plus double layer water (including adsorbed water) is an important contributor to compression upon pressure application and is almost the sole cause of rebound upon load reduction. The influence of this component is illustrated by Figure 5, showing two adjacent soil colloids and the relationship (Figure 5c) between spacing and total potential energy.

At equilibrium the externally derived intergranular pressure (consider the colloids as having unit areas) $\overline{\sigma}$ plus the electrical attraction between the particles equals the electrostatic repulsion. The spacing 2d is the one that brings about equilibrium for this force system.

If we add an increment of intergranular pressure, $\Delta\bar{\sigma}$, as shown in Figure 5b the forces are not in equilibrium until the inter-colloid spacing is reduced by $\Delta 2d$. Figure 5c shows that reducing the spacing $\Delta 2d$ requires an input of work equal to ΔV; in other words, the net repulsive force between the colloids have increased $\Delta\bar{\sigma}$. If $\Delta\bar{\sigma}$ is removed the colloids will move apart a distance of $\Delta 2d$, since force and spacing are uniquely related.



Interparticle Spacing as Function of Forces

Figure No. 5

CHAPTER III

LABORATORY INVESTIGATIONS

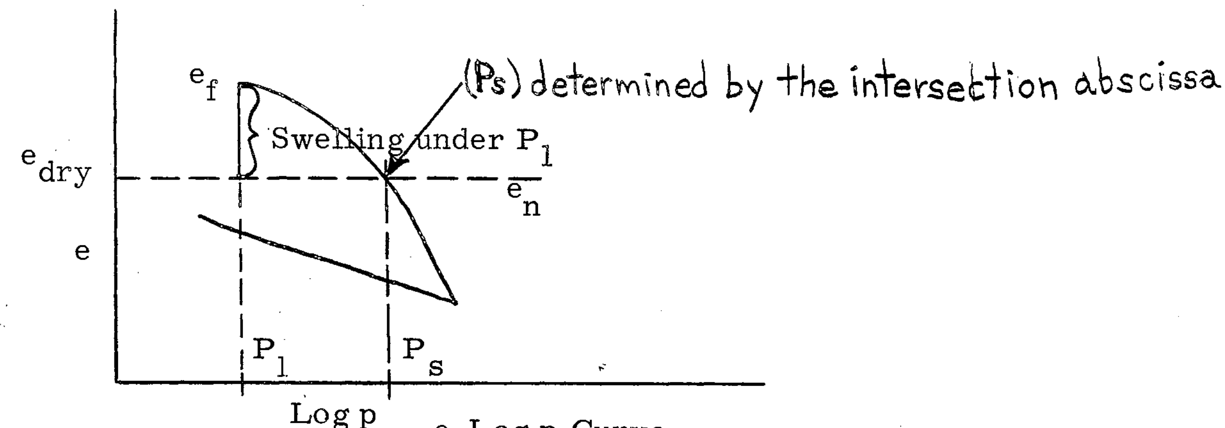
THE PLAN OF INVESTIGATION

The specific objectives of this thesis were outlined in the introduction. The purpose of this section is to expand that outline and describe the laboratory tests which were used to attain each objective.

Relationships Between Load and Swelling. The pressure applied to the clay by pore water in tension is dependent upon the effective pore diameter of the clay. As seen from the spring analogy, if the clay is dry and loaded to the same pressure as the pressure that produced the shrinkage, the clay cannot swell against this pressure when free water is applied to the soil.

One method for determining the pressure required to prevent the swelling of clay that has been assumed valid is explained as follows:

> A sample of clay with a water content below the shrinkage limit is placed in a consolidation ring between porous stones as for a consolidation test with a light load applied. The sample is flooded and allowed to swell completely after which it is subjected to a standard consolidation test. The resulting void ratio-pressure (e-logP) curve which indicates the void ratio after complete consolidation under the indicated pressure will appear as shown in Figure 6. In this figure the intersection of the horizontal line indicating the natural void ratio, e_n, with the void ratio pressure curve indicates the pressure required to recompress the clay to the same



e-Log p Curve
Figure No. 6

void ratio as produced by the capillary water. According to reasoning used in the spring analogy, dry clay loaded to a pressure equal to P_s will neither swell when the clay is saturated nor decrease in volume due to load. Pressures greater than P_s will produce settlement when the soil becomes saturated. Pressures smaller than P_s will allow swelling of the clay.

In order to check this assumption and to get a relationship between pressure and amount of swelling, Test Series (A) composed of seven test samples was conducted. Six of these test samples were loaded under different pressures, allowed to swell, then loaded in increments as a consolidation test. One test sample was prevented from either swelling or consolidating by varying the load which confined it. The details of this testing procedure mentioned briefly here is given in full later.

Relationships Between Molding Water Content and Swelling. In order to determine the effect of molding water content upon the swelling potential of clay, Test Series (B) composed of seven soil samples was conducted. Each of the samples was molded with a Standard Proctor compactive effort but the molding water content was varied. After the samples were molded they were fitted into a consolidation ring and placed between porous stones as for a consolidation test. The samples were then allowed to dry out in the air. Six of these samples were confined under the same load and flooded with water. The samples were allowed to swell completely after which they were subjected to a standard consolidation test. One sample was also prevented from either swelling or consolidation by varying its confining load. The details of this testing procedure are also included in full in the Lab. Procedure of this section.

DESCRIPTION OF CLAY TESTED

The clay tested in this investigation was obtained from the excavation for the Public Health Building on the campus of Oklahoma State University. It represents the typical reddish brown clay of this region, a clay that is heavily fissured and contains some sand and some rock fragments distributed throughout its mass. The specific characteristics of the soil are as follows:

Specific Gravity		Gs = 2.70
Atterberg Limits and Indices:	Liquid Limit	38.4
	Plastic Limit	15.7
	Plastic Index	22.7
	Flow Index	6.6
	Toughness Index	3.44
	Shrinkage Limit	10.72
	Natural Water Cont.	16.47
Grain Size Distribution:	D_{60} = 0.023 mm	
	D_{10} = 0.0005 mm	
	C_u = 46	
	General Description: Inorganic Clay of Medimum Plasticity	
	Colloid Content % minus 0.001 mm	26%

Compaction Test	γ_{dry}	$w_{opt.}$
Standard Proctor	116 #/ft^3	15 %
Miniature Harvard	108 #/ft^3	16.5%

LABORATORY PROCEDURE

UNDISTURBED CLAY SPECIMENS (SERIES A)

The clay used in this series of tests was obtained from the excavation for the foundation of the Public Health Building at a depth of 10 ft. It was removed in blocks about eight inches cube which were carved from the open foundation bank. These blocks were wrapped with a plastic covering immediately upon removal and later at the soils laboratory with cheese cloth impregnated with micro-crystalline wax. Prior to the time that these covered blocks of clay were used for testing purposes they were stored in a room subject to climatic temperature variations for approximately 6 months. The water content at the time they were used was about 16.5 % which is an average natural water content for Permian Clay.

The undisturbed clay in the shape of rough blocks was trimmed into a cylindrical shape the dimensions of which are convenient and adequate for testing. A sharpened ring 6.0 cm. in diameter and 12.0 cm. in length was used to accomplish this. It was placed upon a block of the clay and by alternately trimming the clay from around this sharpened ring with a paring knife and gently pressing, a cylinder of clay was pressed into the ring. The clay cylinder was then removed from the sharpened ring and cut into thinner cylinders approximately 1.9 cm. in thickness. During this procedure every effort was made to form cylinders of consistent dimensions, but due to rock fragments contained in the clay some edge irregularities resulted. In this series

20 such cylinders were carefully prepared, from which the best 7 were used as test specimens. The thin cylinders were allowed to air dry for approximately 3 weeks to assure that the water content of the clay specimens was below the shrinkage limit. From measured dimensions, the average diameter and thickness of each specimen was then calculated and recorded.

Each of the soil samples now in the shape of a thin cylinder was confined laterally and prepared for testing. Lateral confinement was provided by an iron ring and leadite, a caulking compound. In preparation for this confinement the clay cylinders were wrapped about their circumferences with aluminum foil to prevent contact of the clay and leadite. The wrapped clay cylinders were then centered in iron rings each 2.54 cm. thick and 9 cm. in diameter. The void space between each iron ring and clay cylinder was filled with molden leadite. After a few minutes the leadite hardened and the clay cylinders were were confined. The iron rings containing the clay and leadite were each placed on a porous stone about the diameter of the iron ring. The clay was then covered on top by another porous stone disc, the diameter of which was slightly smaller than that of the specimen. This apparatus was placed into an aluminum bowl in order to supply an unlimited amount of water to the specimen during the progress of the test.

The swelling and consolidation tests were conducted by placing the bowls containing the soil specimens and porous stones in a consolidation machine. This machine employs two decimal beams in series, so that a lever arm ratio of 100:1 is achieved. A movable fulcrum allows adjustment for deformations. The loads achieved by each of these machines are transmitted through a head to the top porous stone covering the clay specimen. An Ames dial is placed against the head to allow measurement of deflection.

Once the clay specimens were all confined and assembled on the consolidation machine the loading schedule was begun. This schedule provided for six of the specimens to completely swell or consolidate under the loads indicated below and then undergo a standard consolidation test. One specimen was to be confined under whatever pressure was required to prevent volume change when the specimen was flooded.

A-1	0.394 Ton/ft.^{2}
A-2	1.190
A-3	2.479
A-4	3.938
A-5	6.202
A-6	9.483
A-7	Variable Pressure

The specimens A-1 thru A-6 were loaded and immediately flooded with water. Dial readings were taken at 2 day intervals for a period of three weeks when, for practical purposes, deformation had ceased. The pressure on each specimen was then doubled at intervals of three days up to a maximum of about 50 T/ft^2, the capacity of the machine. It was then reduced successively by one-half down to approximately 6 T/ft^2. From 6 T/ft^2 it was reduced to 1.5 T/ft.^{2} and then to zero. Readings for each load taken 3 days (72 hrs.) after each load was applied, at which time primary consolidation had ceased, were used to compute the void ratio. An exception to this was the final zero load period when the period of time was 2 weeks. These values were used to plot an e-log p curve for each specimen (Figures 7-12). In order to determine the initial void ratio of the clay specimens, the clay was carefully carved from each confining iron ring leadite combination. These carvings, along with the water used to make sure all the clay in each specimen was removed, were deposited in an evaporating dish and dried out in an oven. The weight of dry clay in each of the specimens was thus determined. Knowing the weight of a clay specimen, its thickness and cross-sectional area, along with the specific gravity of the clay, the initial void ratio e_n was determined.

Specimen A-7 was confined under 1.5 T/ft^2 and flooded with water. The Ames dial was then observed at hourly intervals and any indicated deformations were counteracted by either increasing or decreasing the specimen load. The volume change was thus prevented and the load which accomplished this determined. The results of this test are shown on the $\frac{\Delta e}{1+e}$ vs p curve of Figure (20) as the Range of Zero Δe.

REMOLDED CLAY SPECIMENS (SERIES B)

The clay used for this series of tests was the same as used in Series A. The scraps that resulted from trimming the clay blocks into cylinders for Series A were air dried and pulverized. The soil was remolded into six soil cylinders having different molding water contents. Each cylinder was compacted with the same effort (Standard Proctor). The actual molding water contents were found by oven drying trimmings from each clay cylinder immediately after compaction and are as follows:

B-1	14.66 %
B-2	15.74
B-3	18.85
B-4	20.52
B-5	22.42
B-6	23.72
B-7	20.52 %

Clay specimens used for testing were then obtained from the remolded clay cylinders. The sharpened ring used to obtain the undisturbed soil specimens was placed upon each clay cylinder and by alternately trimming the clay from around the ring's cutting edge and gently pressing the ring, a cylinder of remolded clay was pressed into the sharpened ring. The cylinder of clay was removed from the ring and trimmed into thinner cylinders. These cylinders were likewise allowed to air dry for approximately three weeks or until attainment of the shrinkage limit was assured. Each cylinder's average dimensions were then determined and recorded. The cylinders were approximately 6 cm. in diameter and 1.9 cm. thick.

The preparation of the specimens for testing, or lateral confinement, was the same as was used for the undisturbed clay specimens with one exception. The aluminum foil which was wrapped only around the circumference of the undisturbed clay specimen was also used to cover the bottom cross-sectional area of the remolded clay specimens. This was done in order to prevent any possible contact of the clay and leadite. It was observed, after the undisturbed test series was completed, that the bottom of the clay specimens were darker in color than the tops of the specimens indicating some contact between the clay and leadite. As a result of this observation the remolded clay specimens were covered on the bottom to prevent any possibility of contact. It should be noted here, however, that this procedure causes the theoretical time for consolidation to increase four fold, thus causing the time for primary consolidation to increase from the usual 2 or 3 days to a week or ten days. In this particular test series it was determined that a loading period of one week (168 hrs.) was sufficient time for primary consolidation to occur.

Once the remolded clay specimens were confined and assembled on the consolidation machine, loading was begun. Six specimens, each compacted at a different molding water content, were loaded to the same pressure, 1.0 $T/ft.^2$ and flooded with water. The specimens were then allowed to completely swell or consolidate. This process required about four weeks time, during which dial readings were recorded every two days. Following the initial phase the loads on the specimens were then doubled at intervals of 7 days up to a maximum of about 50 $T/ft.^2$. They were then reduced by one-half down to 6 $T/ft.^2$, and then to zero. Readings for each load taken 7 days (168 hrs.) after each load was applied, at which time primary consolidation had ceased, were used to compute the void ratio. An exception to this period of time was the final unloaded period which was two weeks. These values were used to plot the e-log p curves for each specimen Figure (13-18). The initial void ratio was determined in the same manner as for the undisturbed soil specimens.

The seventh soil specimen was used to determine what load would cause the clay specimen compacted at a molding water content of 20.52% neither to swell nor consolidate. This specimen was loaded initially to 1.5 $T/ft.^2$, flooded with water, and the load adjusted as required to prevent change in the dial reading. This process was continued throughout the duration of the test. The results of this test are presented in Chapter IV.

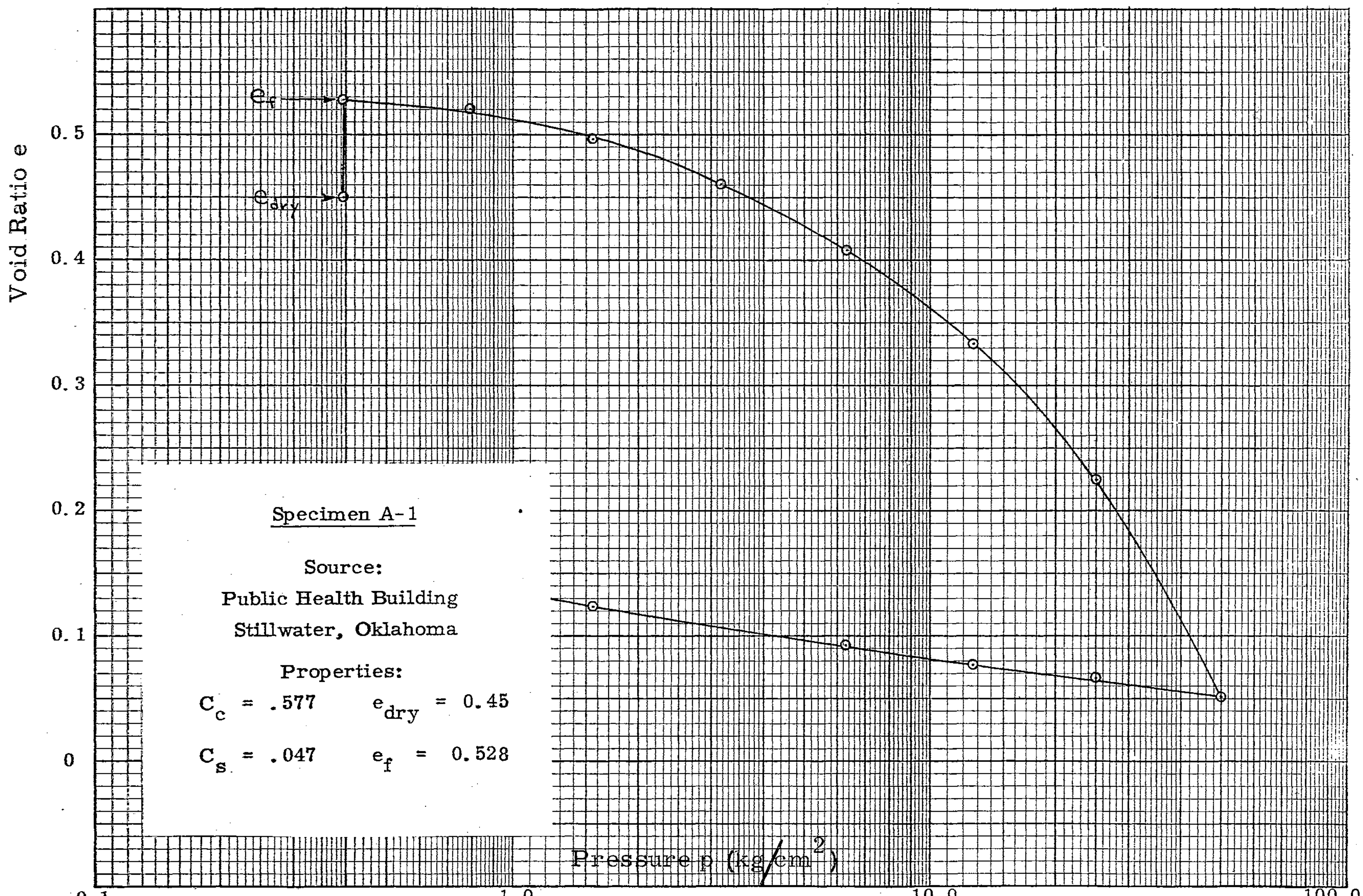


Figure 7

Consolidation Test (e - log p Curve).

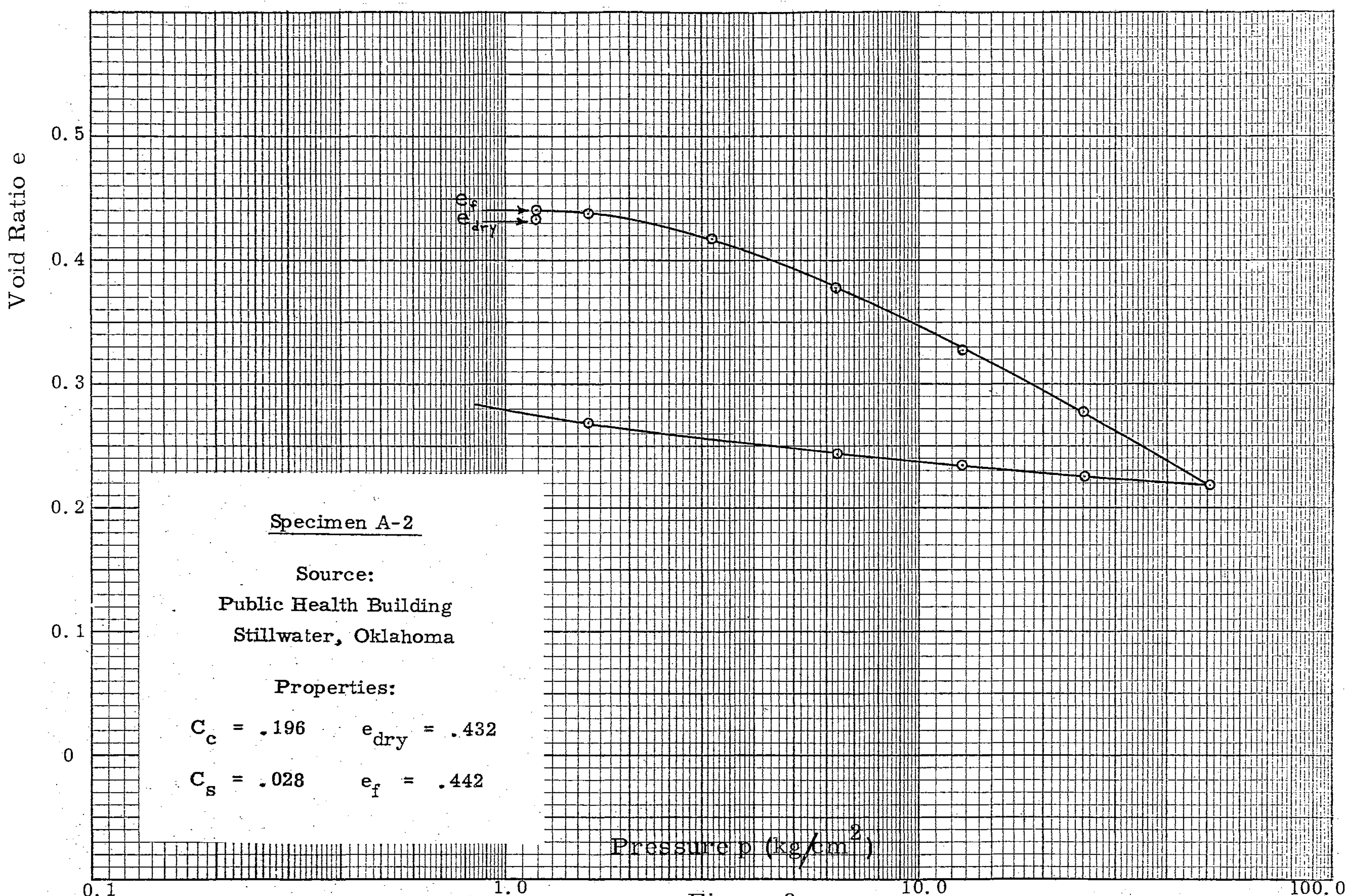


Figure 8

Consolidation Test (e - log p Curve).

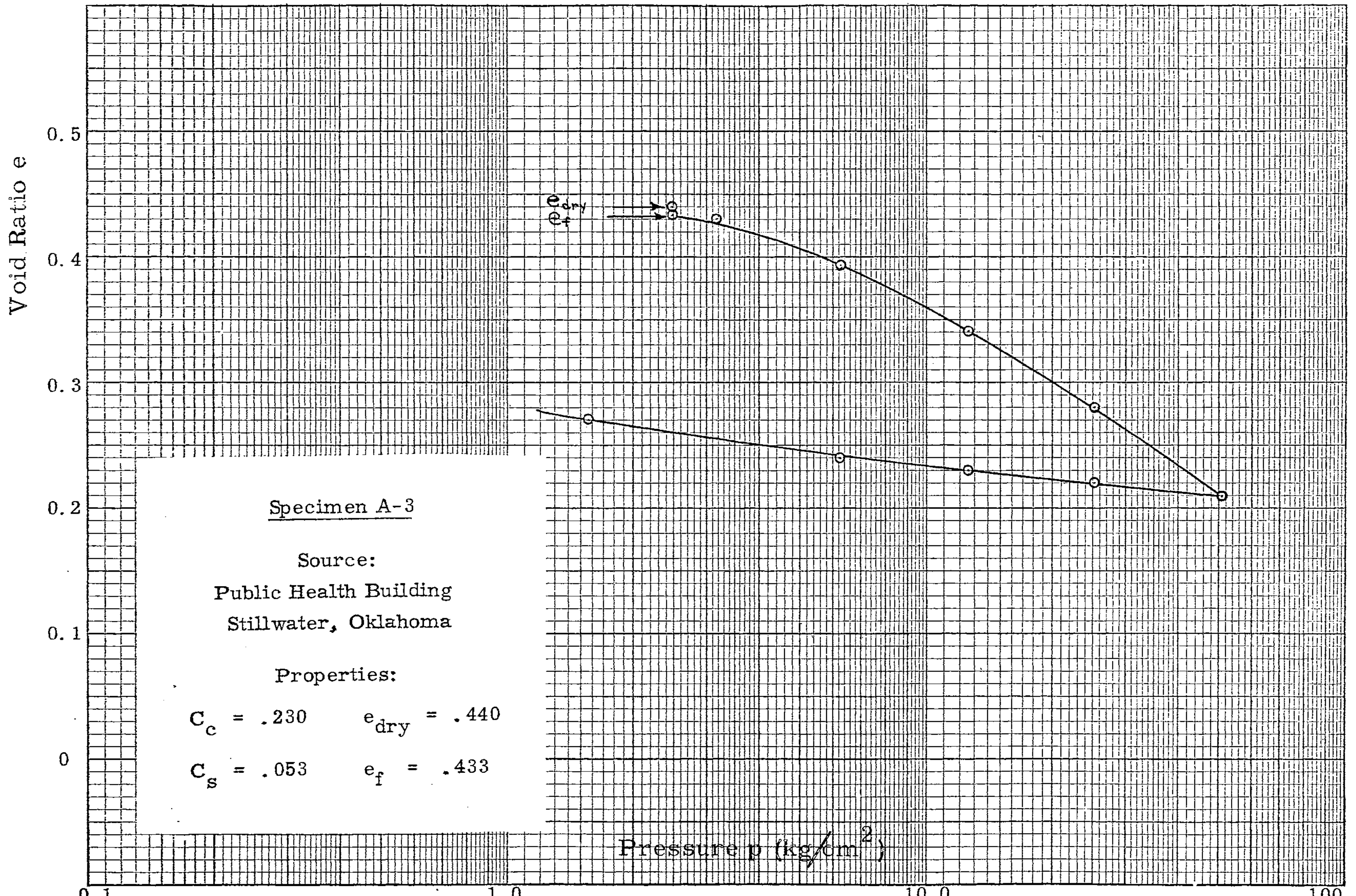


Figure 9

Consolidation Test (e - log p Curve).

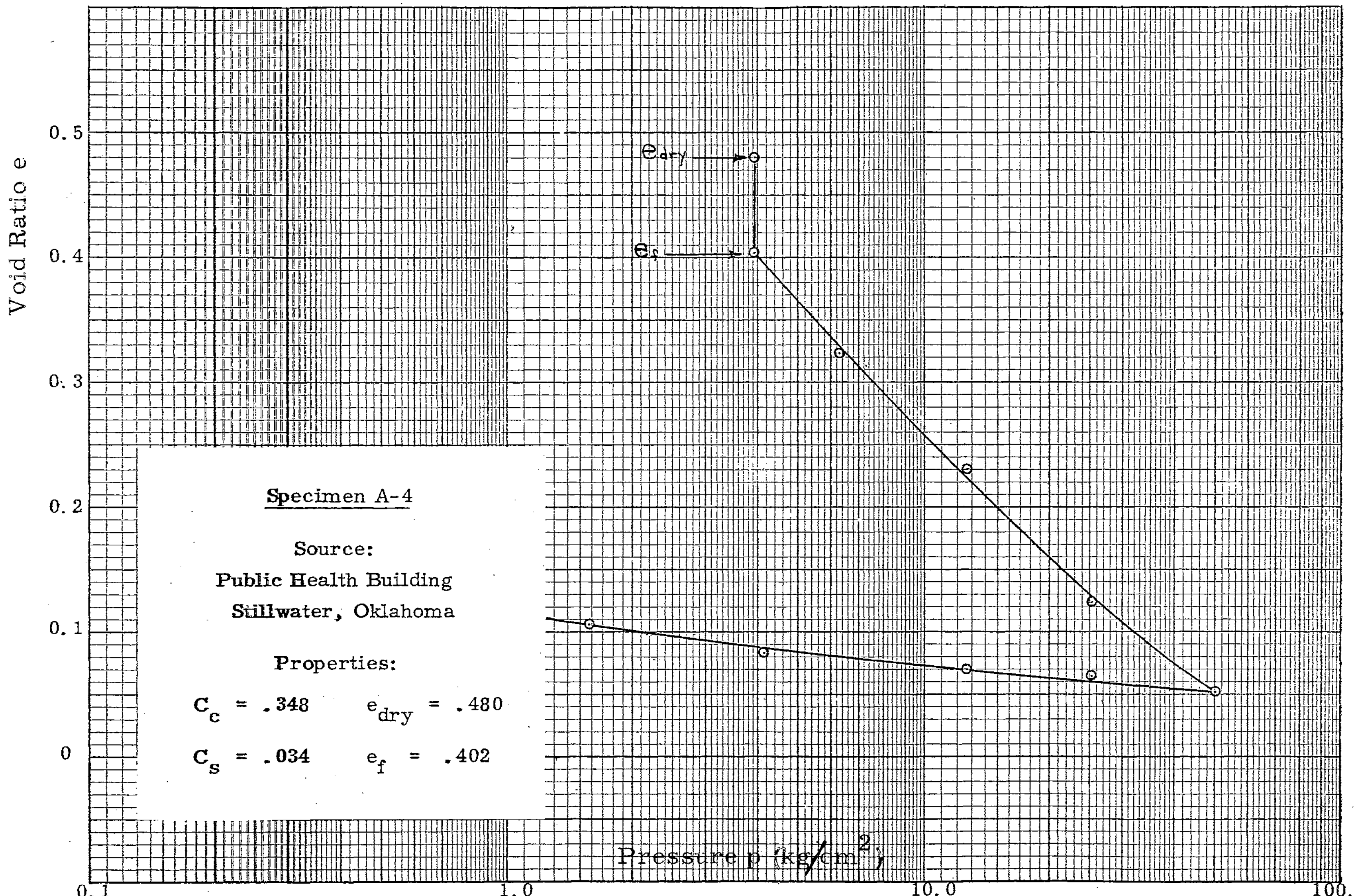


Figure 10

Consolidation Test (e - log p Curve).

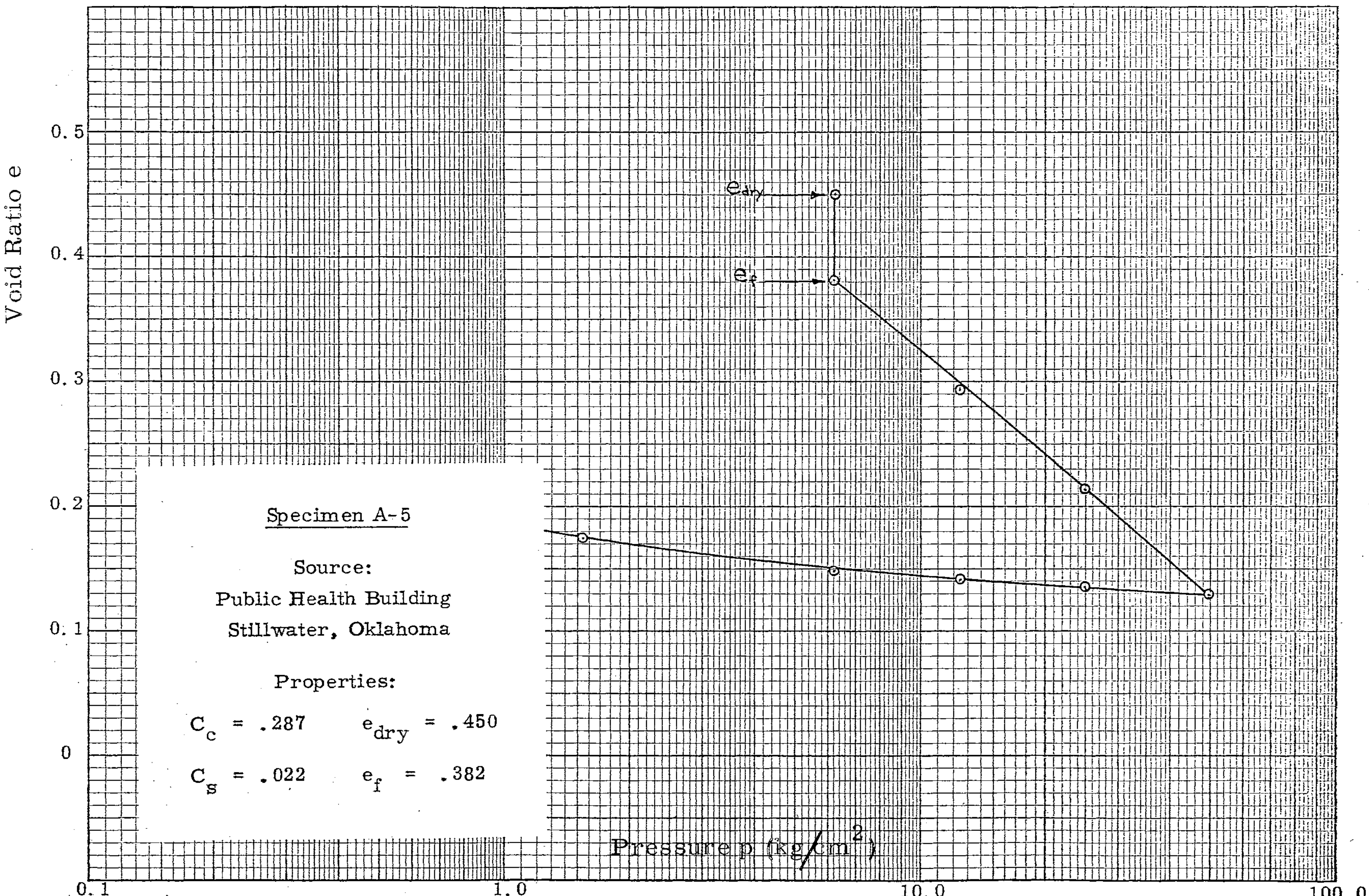


Figure 11

Consolidation Test (e - log p Curve).

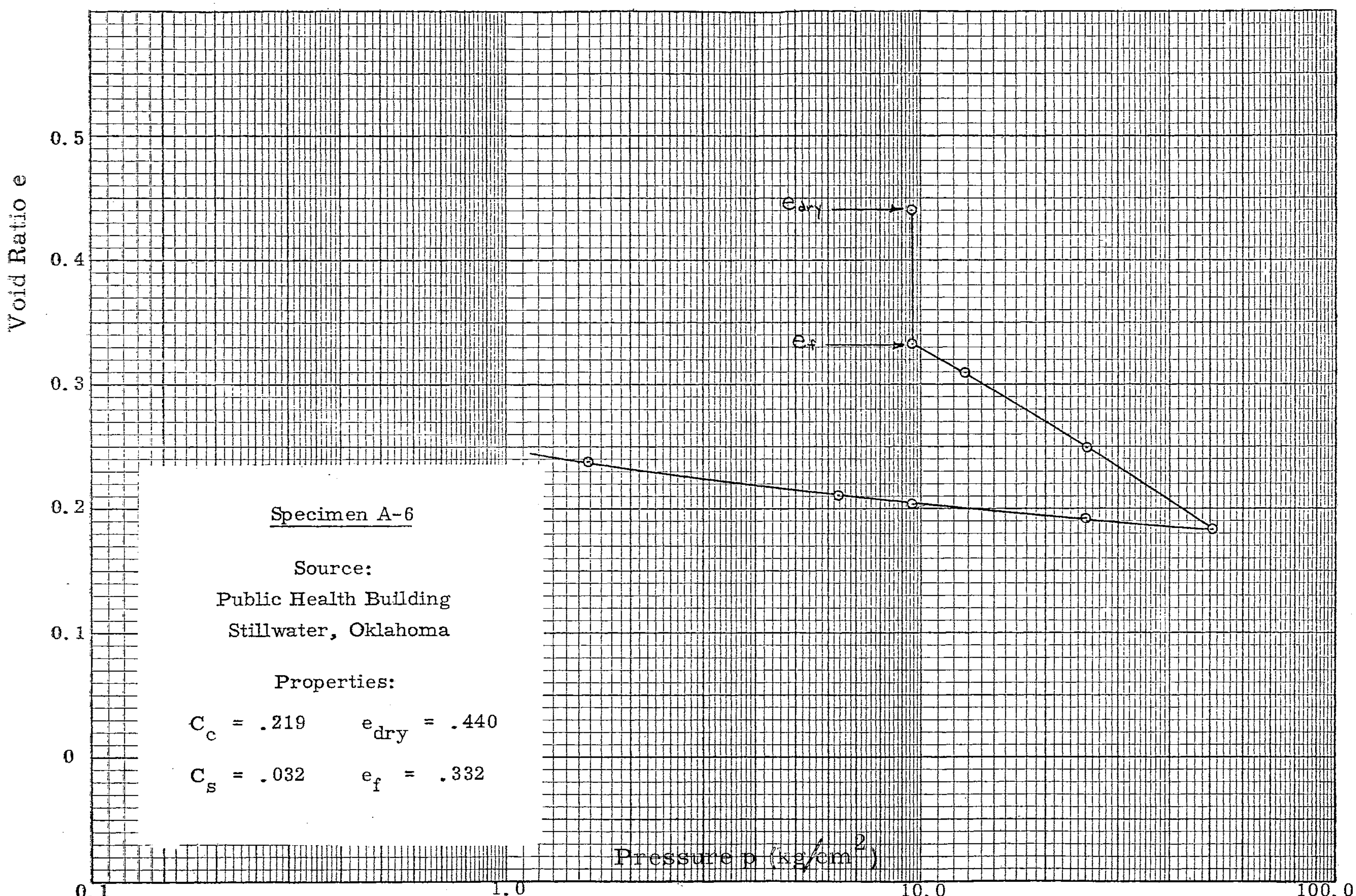


Figure 12

Consolidation Test (e - log p Curve).

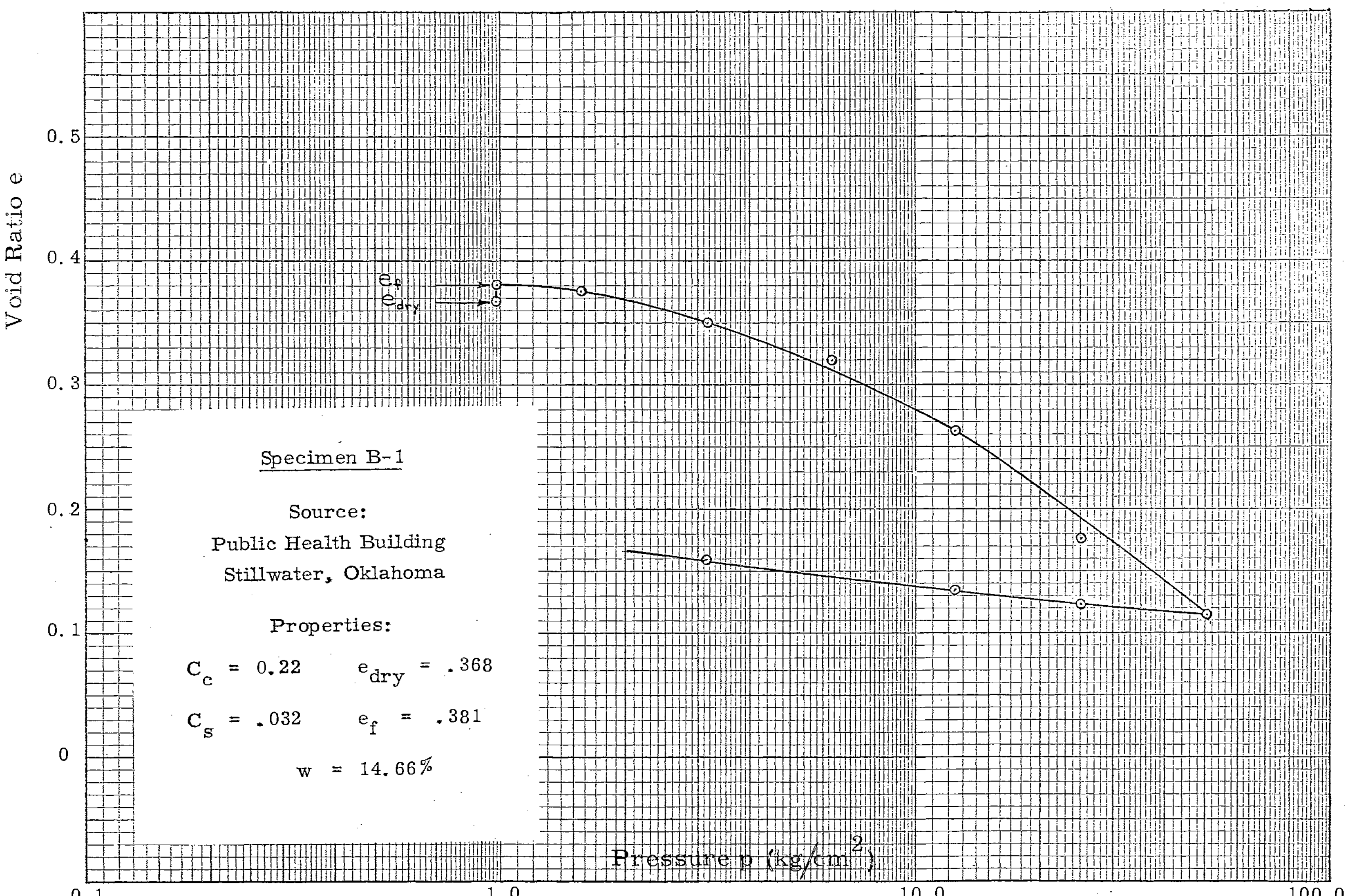


Figure 13

Consolidation Test (e - log p Curve).

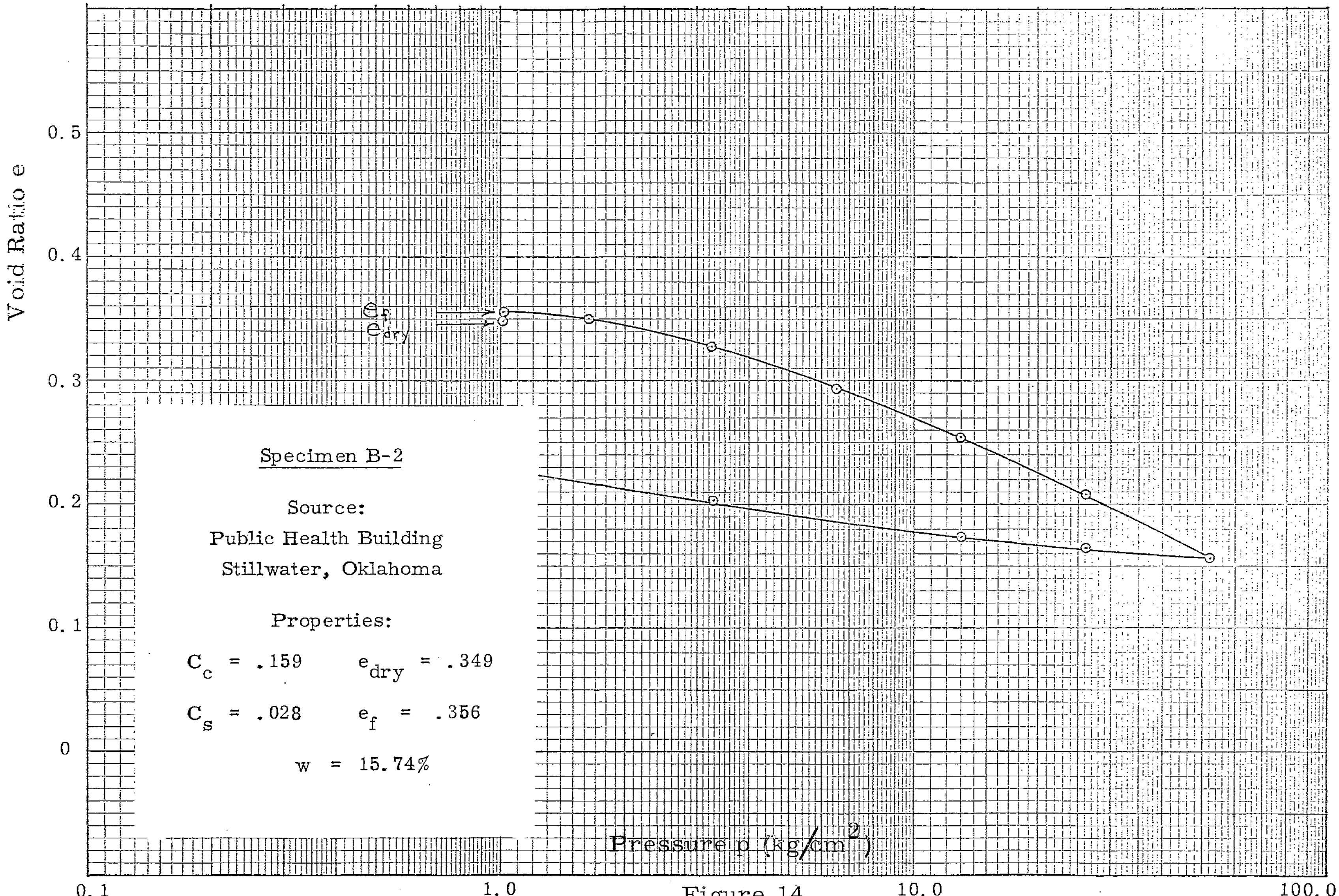


Figure 14

Consolidation Test (e - log p Curve).

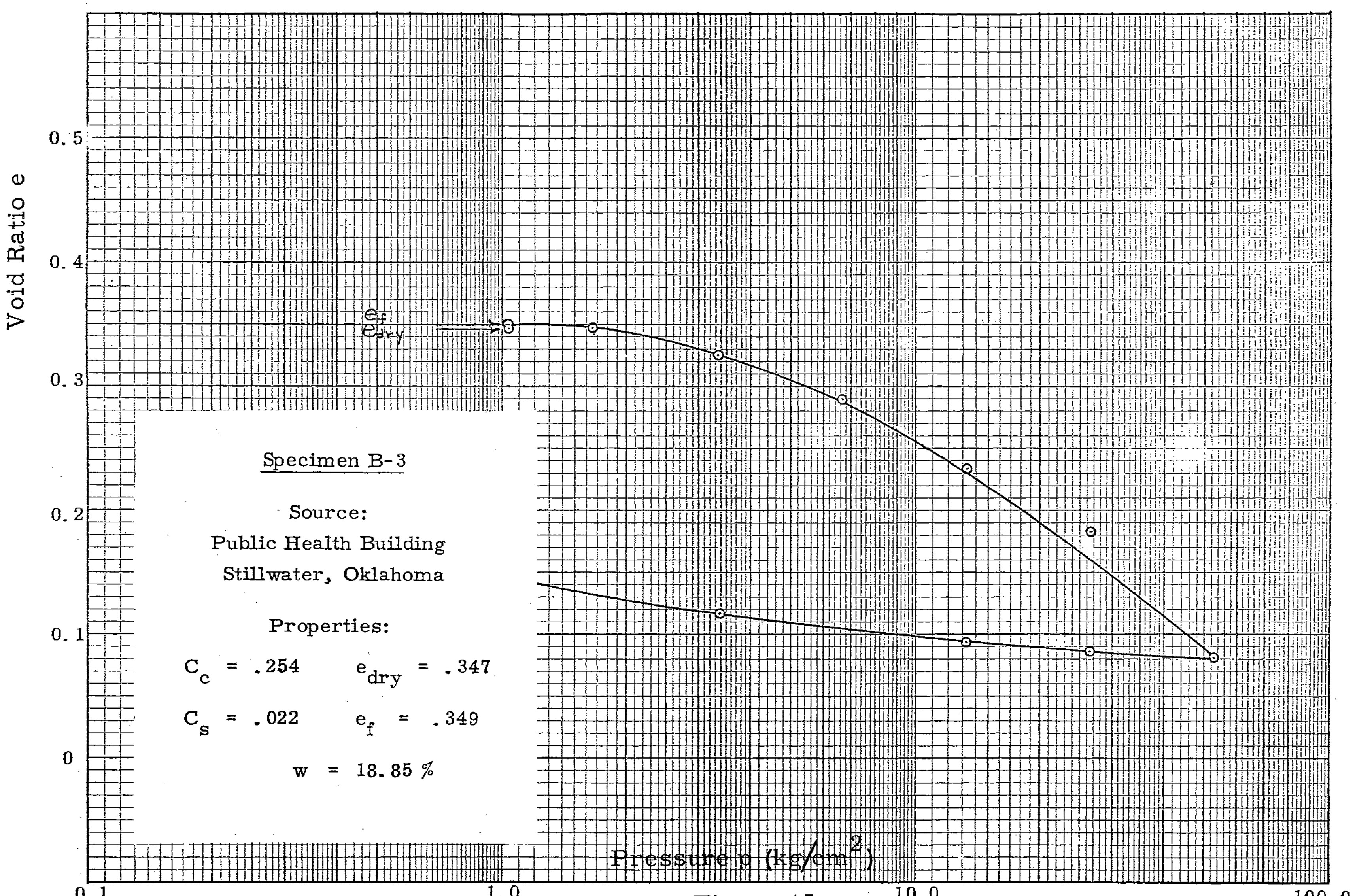


Figure 15

Consolidation Test (e - log p Curve).

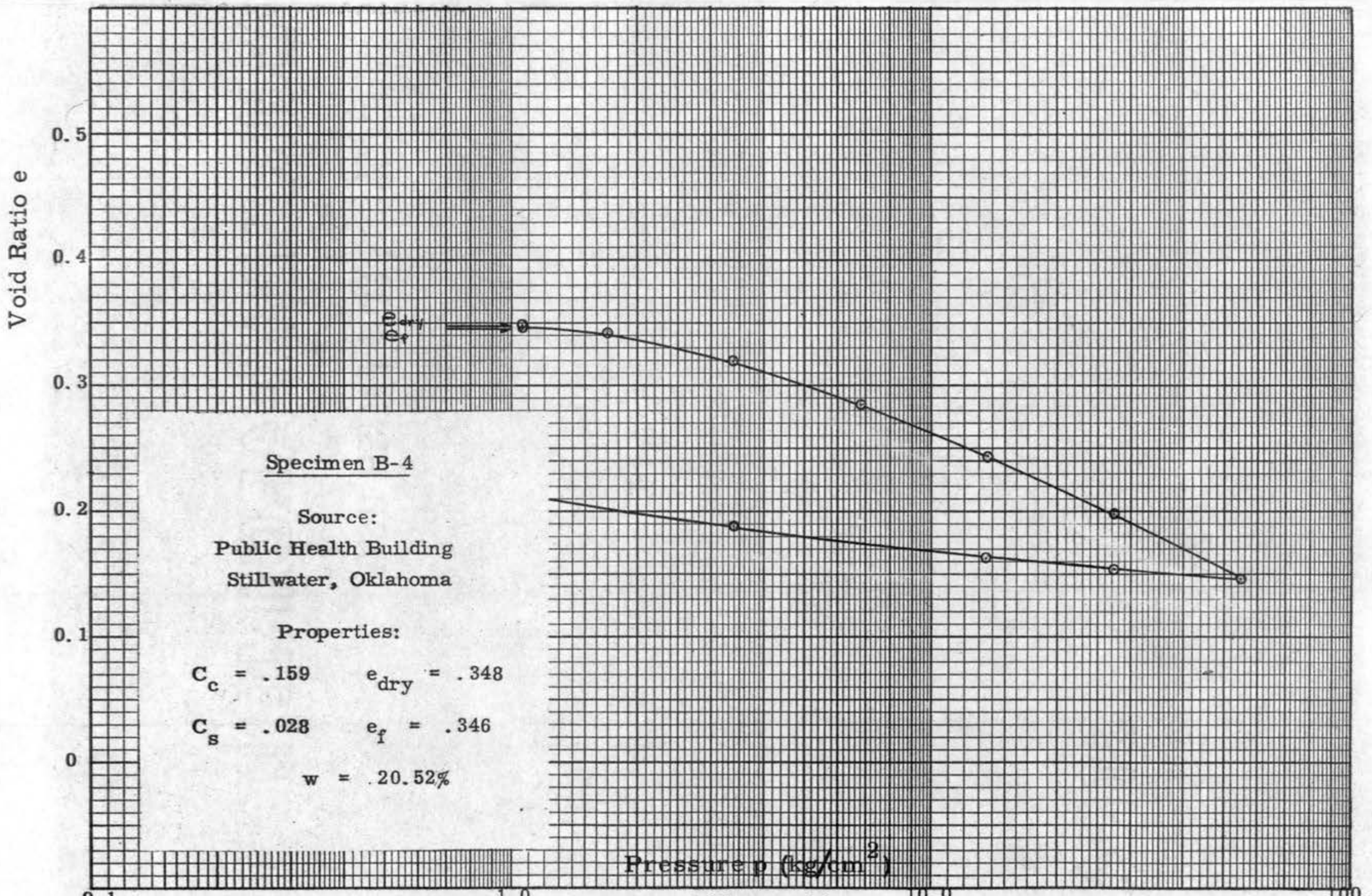


Figure 16

Consolidation Test (e - log p Curve).

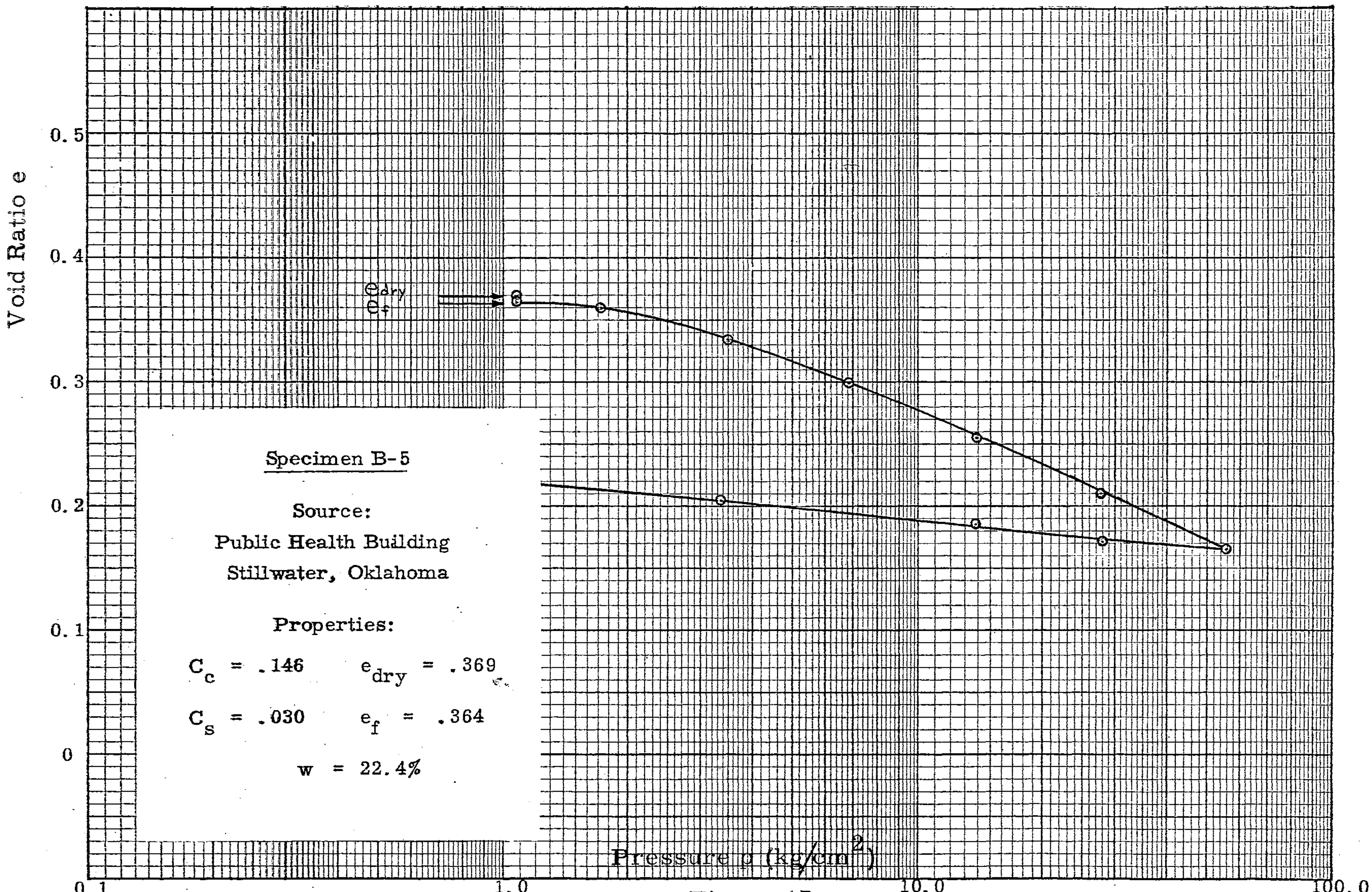


Figure 17

Consolidation Test (e - log p Curve).

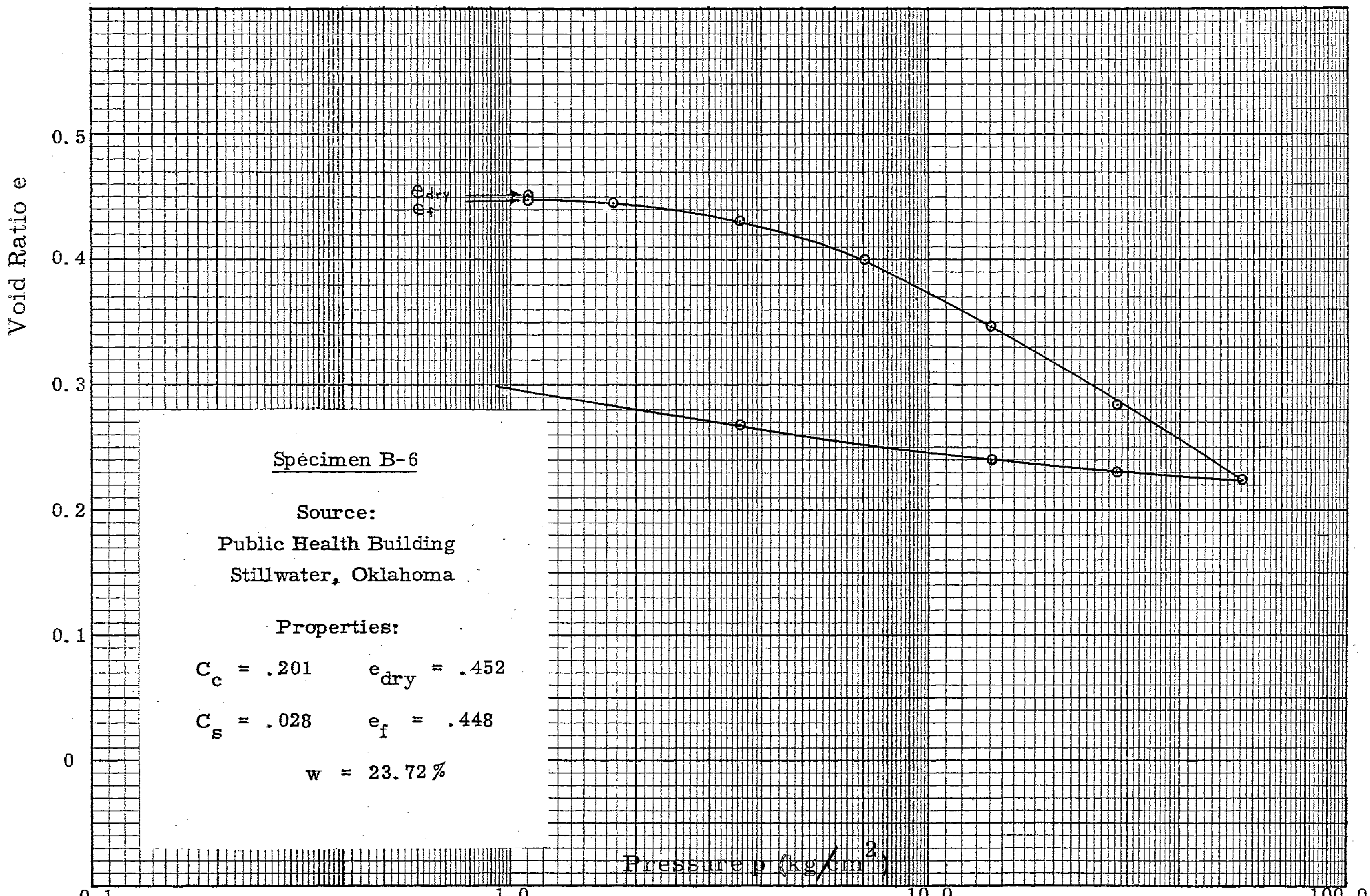


Figure 18
Consolidation Test (e - log p Curve).

CHAPTER IV

DISCUSSION OF RESULTS

GENERAL

Expansive clay investigations which have been conducted here at Oklahoma State University were used as the source of data against which the results of this investigation were compared. The results from the undisturbed test Series A were compared to results of investigations on undisturbed clay by Midani and Hauck. The results of the remolded clay Series B were compared to results on remolded clay by Theophanides.

UNDISTURBED CLAY

The objectives of the undisturbed test series were explained in detail at the beginning of the laboratory investigation section. From this explanation it can be surmised that the numerical value of P_s and the amount of swelling which will occur under a specific load, are the indicative quantities resulting from swelling tests which will best provide a means of comparing swelling results. The reasons these quantities seem best for comparison purposes are that they can be analyzed from a graphical perspective and mainly because they will either indicate or give directly the desired quantities.

In determining the value of P_s, the pressure theoretically required just to prevent swelling, data from the present tests and from previous tests are utilized. The manner in which these results are presented is basically the same as was used by R. E. Means[6].

Pressure on the clay is plotted against unit swelling deformation and lines are drawn through points indicating swelling of the specimens with the same P_s. If the lines when extended intersect the line of zero swelling at approximately the indicated P_s, then this would suggest that the method of determining P_s, which was to be checked by this investigation, is fairly reliable. If the extended lines do not intersect the line of zero swelling at the approximate values of P_s, this would conversely suggest that this method may not be very reliable. In Figure 19 for each confining pressure applied the corresponding unit swelling and P_s values have been plotted for all undisturbed swelling specimens tested at Oklahoma State University known to the author. After a careful examination of the possible relationships between P_s, pressure for zero swelling, and the location of clay used for testing, a few observations can be made. First, for the Blackwell Oklahoma Clay, lines connecting common values of P_s when extended do cross the zero swelling axis at about the indicated P_s. Second, the results of extending lines connecting approximate values of P_s for the clay specimens from the Stillwater Region also show to a lesser degree this relationship. Third, the clay from Blackwell has a greater swelling potential than any tested from the Stillwater Region.

The study of the amount of swelling which will occur under various loads was restricted to clay of the Stillwater Region. All data resulted either from this investigation or from swelling investigations conducted by Midani and Hauck. Data from each of these investigations has been plotted on Figure 20 with confining pressure versus unit deformation ($\Delta e/1+e$).

In order to consider the data plotted on Figure 20 from a comparative view point, it seems imperative that the test procedures used

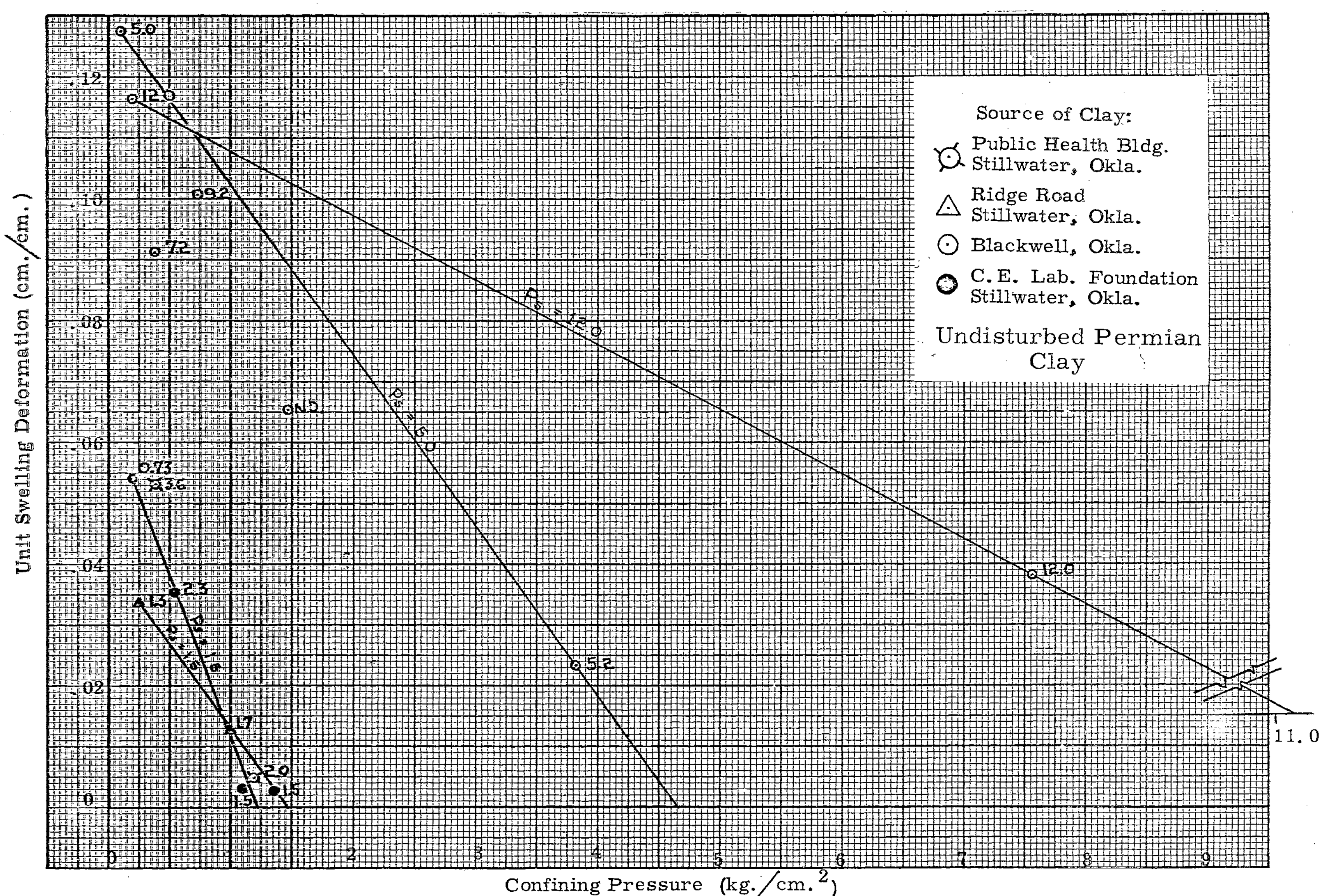


Figure 19

Relationship of P_s to Pressure for Zero Swelling

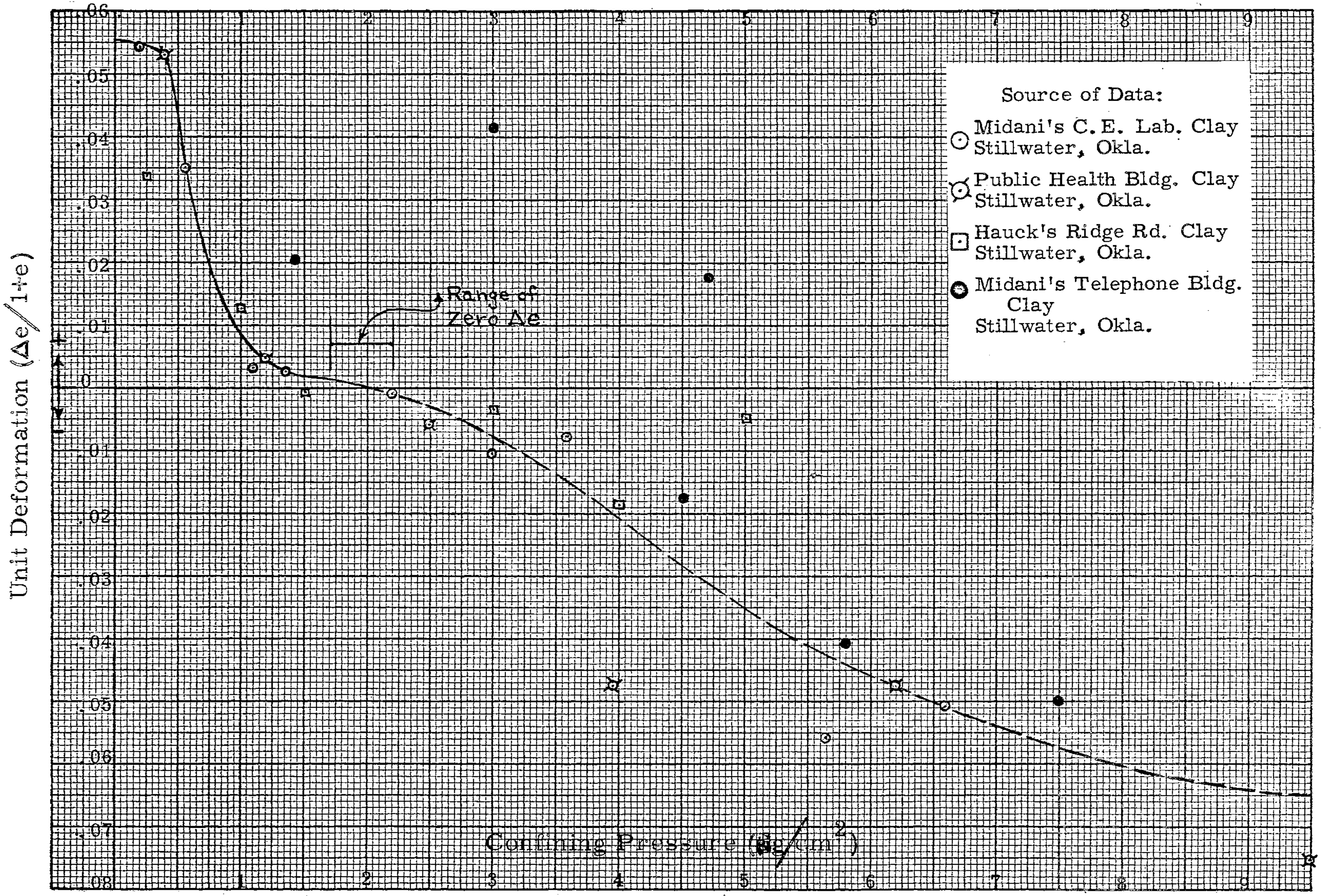
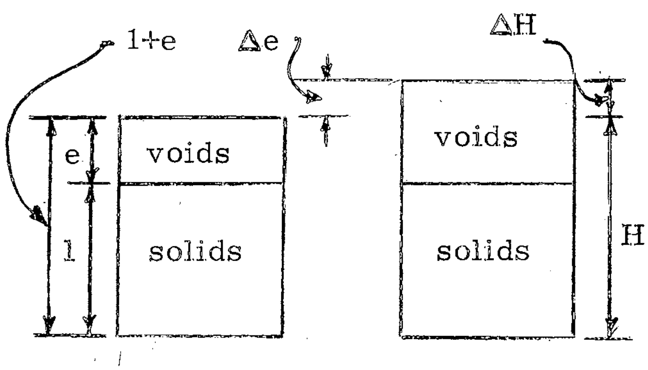


Figure 20

Deformation Pattern for Undisturbed Permian Clay to obtain each set of data be compatible. After reviewing the procedure used in each investigation it was found that they conformed very closely from the standpoint of the general approach and equipment used. It should, however, be noted that the method of laterally confining the clay specimens to be tested was different in each investigation. Midani confined his specimens laterally while they were still moist and applied a light load until they were air dried. Hauck used aluminum foil with paraffin wax to fill the shrinkage cracks around his specimens. In this investigation it has been pointed out that leadite within an iron ring was used to confine the specimens. As a result of the different confinement used by each investigator, some factor of difference is introduced in any comparison of the data. One would think for example that a clay specimen which was not confined adequately would not indicate as much swelling measured by vertical extension as would specimens confined very well. Midani in his procedure did not indicate any sample confinement problems, but he did not dry his clay specimens before placing a load on them. Hauck did experience difficulties and discussed them in his thesis. In any case, since the conditions which prevailed during testing are known the data can be more intelligently evaluated.

If the data plotted in Figure 20 are considered objectively, certain discrepancies and consistencies are noted. The discrepancies observed are found in the data Madani obtained using clay from the telephone building foundation here at Stillwater. This data indicated that the specimen confined under a pressure of 4.5 tons/ft.^{2} consolidated, while the specimen swelled which was confined under a pressure of 4.7 tons/ft.^{2}. The magnitude of each deformation was about the same. These data indicate an irregularity which is contrary to a widely accepted relationship which holds that swelling is inversely proportional to load. Another of Midani's test specimens indicated a P_s value of 21 which also indicates some erratic nature to that test as this clay's unit swelling potential shown in Figure 19 is less than that indicated by the Blackwell Clay and the maximum P_s determined for that clay was 12. Apparently, then, there is some error in Midani's data relating the Telephone Building test series. If the plotted points representing the data for this particular series is ignored, the remainder of the points form a rather definite pattern -- the mean of which is indicated by the dashed line in Figure 20. In particular it is observed that test results from a clay taken from the foundation of the C.E. Lab at Oklahoma State University, and the results of this investigation are fairly consistent. By consistent it is meant that tests conducted under similar loading conditions indicate deformations of the same sense which are of about the same magnitude. Hauck's data for Ridge Road Clay also indicate a general pressure-deformation trend of similar nature but of smaller magnitude. A smooth curve was constructed through the swelling points of the data from these three series. This curve should approximate a mean relationship between confining pressure and unit swelling for the clay of Stillwater. If this curve is connected to the dashed mean pattern line it is noted that the pressure intercept is 2 ton/ft.^{2}. This would mean theoretically that if this clay were loaded on the surface to a pressure of 2 tons/ft.^{2} and an unlimited supply of water made available, there would be no resulting change in volume due to swelling or consolidation. It will be recalled that specimen number 7 of Series A was flooded with water and the confining pressure adjusted to counteract any deformation. It was not possible, in this test, to establish a particular value of the confining pressure, the necessary pressure varying between 1.7 and 2.2 tons/ft.^{2}. Any where within this range the deformation indicated by the Ames dial was very small, but outside this range the specimen would swell or consolidate readily. This test result then would tend to support the results obtained by extending the mean swelling line.

If the unit deformation versus pressure plot is modified to a unit swelling deformation plot with the unit deformation now expressed in inches/ft. of clay a practical application of this test data may be surmised. First in order to change the $\Delta e/1+e$-p plot to the units of inches/ft. of clay the ordinates to the $\Delta e/1+e$-p plot need to be multiplied by 12. This comes from the following relationship.



e = initial void ratio

Δe = change in void ratio under some constant load

H = Thickness of clay engaged in expansion (1 ft.)

ΔH = change in thickness due to swelling (in inches)

$$\frac{\Delta H}{H} : \frac{\Delta e}{1+e} ; \quad H = H(\Delta e/1+e)$$

$$\Delta H \text{ (inches)} = 1 \text{ ft.} (\Delta e/1+e) \ 12 \text{ inches/ft.}$$

Once this was done the unit swelling deformation plot of Figure 21 resulted. In order to take into account the variation in swelling which is indicated in this plot, a band or range of probable unit swelling has been indicated for the Stillwater Region. An example of how this range of unit swelling potentials could be utilized is as follows. Suppose that the swelling potential for a load of 0.5 tons/ft.^{2} on clay around Stillwater is to be determined. By entering the graph of Figure 21 with the pressure of 0.5 tons/ft.^{2} it is found that the range of swelling will be between 0.23 and 0.6 inches/ft. of expansive clay. The problem of determining the thickness of the clay layer which will participate in expansion is a big problem. If, however, the assumption is made that only the desiccated upper layer of the clay formation will participate in the swelling, and if it is estimated that the soil drys and cracks to a depth of 14 ft. in extremely dry years, then it would be possible to predict a total volume change of between (14 $\cdot$ 0.23) 3.22 inches and (14 $\cdot$ 0.6) 8.4 inches. The time it will take for this total expansion to occur is unknown but one can be sure that it will take many years.

Since this estimated amount of total swelling will occur over a large area, most usually under a building which has kept evaporation from drying out the clay thus giving the clay water from capillary rise and rain, the total swelling would not be reflected in resulting movements of the structure. Only the differential swelling and the variation in frictional forces acting on different supports would cause deformation of the building. A non-structural slab floor of a building on the other hand indicates much better the magnitude of total swelling which may have occurred under a building. It has been observed for example that as much as 8 inches of differential swelling has occurred between a slab floor and the structural frame. Thus, although the total swelling potential of this clay may be quite great, the detrimental effect on structures is caused by the differential movement which may be of considerably smaller magnitude.

In considering the swelling of Permian Clays as a whole, it is known that variations will be encountered from site to site. Just what variations actually occur is not known, but as seen in this comparative

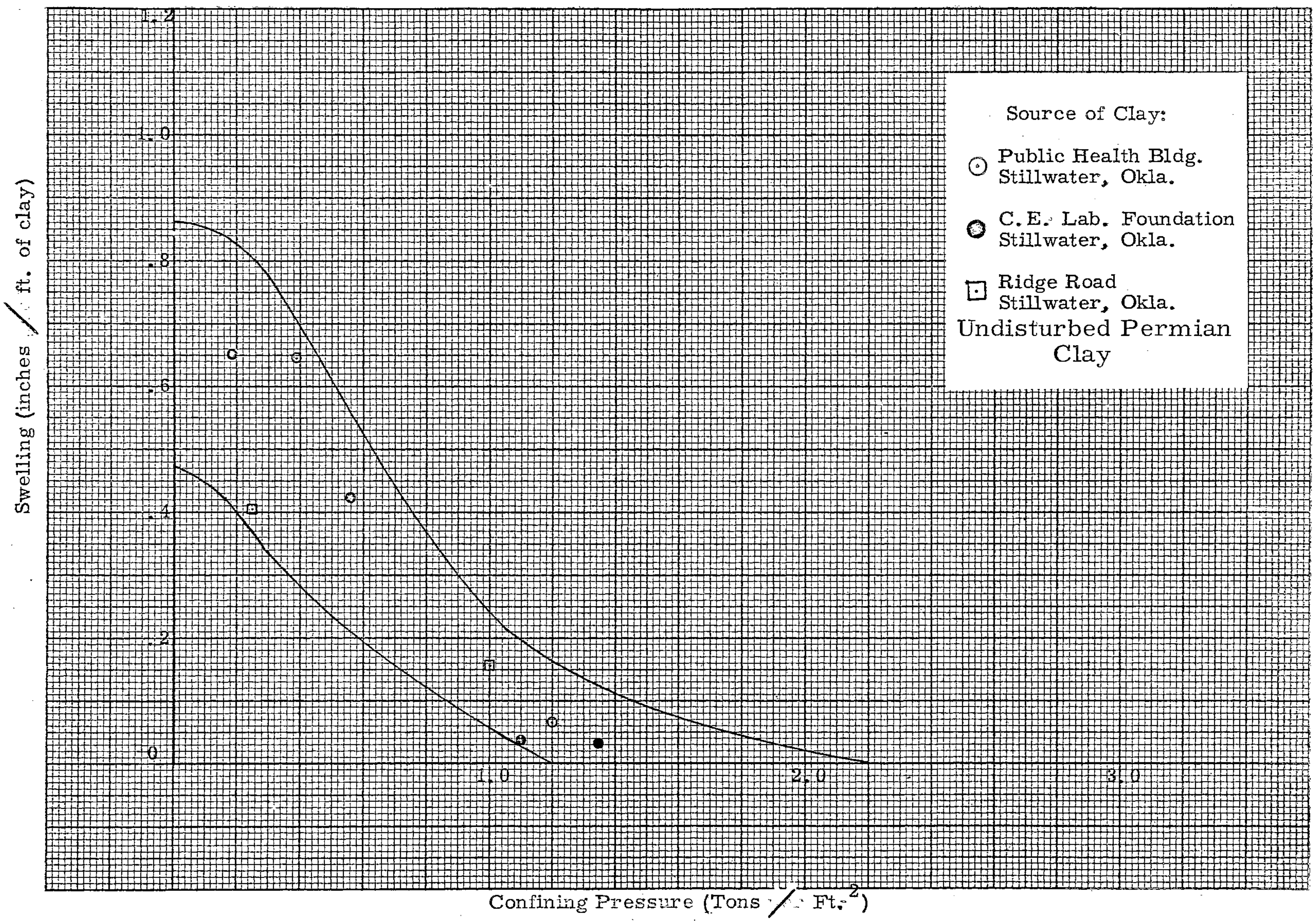


Figure 21

Range of Total Swelling

analysis, clay from sites two blocks apart gave fairly consistent test data, while a clay from another city produced data which was greatly different. It might be interesting also through another comparison to view clay from the Stillwater Region with clays from other areas. In reviewing publications concerning expansive clays it was found that a chart indicating the probable volume changes for expansive materials located in the western states has been published by W. G. Holtz[9]. The criteria used for classifying an expansive material for this chart are the Atterburg consistency test data and the colloid content (amount of particles smaller than 0.001 mm.). This chart was prepared for Western States and the author made it a point to limit its application thereto, but for the purpose of comparison only, it is presented here along with the criteria for Permian Clay, or at least one sampling of Permian Clay, necessary for comparison.

TABLE NO. II

Data for Making Estimates of Probable Volume Changes

For Expansive Materials

Data for Index Test **			Probable Expansion * total volume (dry to saturated)	Degree of Expansion
Colloid Cont. minus 0.001 mm	Plastic Index	Shrink--age Limit		
Western Clays				
> 28	> 35	< 11	> 30	Very High
20-31	24-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
< 15	< 18	> 15	< 10	Low
Permian Clays 26	22.7	10.72	6	

* Based on vertical loading of 1 psi.

** All three test should be considered in estimating expansive properties

It is interesting to note that the criteria established by Holtz for Western States do not seem to apply to Permian Clays of the Stillwater Area. According to the colloid content, plastic index and shrinkage limit the Stillwater Clay would be expected to have an expansion coefficient at 20-30%, but this is not the case. The probable expansion may be estimated by obtaining from Figure 20 the unit volume change corresponding to a pressure of 1 psi. If this is done, a value of about 6 % is obtained.

REMOLDED CLAY

The investigations on remolded clay in this thesis and the work done by Theophanides with remolded clay both have as an objective the determination of the relation of swelling and molding water content. In Theophanides' work clay specimens remolded under two different compactive efforts and varying water contents were subjected to various confining pressures and allowed water for swelling. In this supplement to that investigation, clay specimens were remolded under the same compactive effort, but varying water content and then allowed water for swelling under a constant confining pressure. The comparative results of each of these investigations along with some statements relating to remolded clay are presented in this discussion with the hope of gaining some insight into the swelling potential of compacted clay.

There is a close similarity between the clay used in this investigation and that used by Theophanides. The clay that Theophanides used was taken from a building foundation located about a city block away from the site from which clay for this investigation was obtained. The results of the usual identification tests for clay, along with a Standard Proctor compaction test showed good correlation of the clays. The clays should, thus, exhibit similar swelling properties. However, the testing procedures used for investigation were different. The main differences are in the moisture content of the specimens at the time they were flooded with water, and the confining pressures. Theophanides allowed his clay specimens only a 12 hour period between the time they were molded and the time they were flooded with water. On the other hand in this investigation the clay specimens were allowed to air dry until their moisture contents were at or below the shrinkage limit. Thus, we would expect that more water would be drawn into the dryer clay causing greater unit swelling for the same confining pressure. The confining pressures used by Theophanides were all less than 0.3 $T/ft.^2$ while the constant confining pressure of this investigation was 1.0 $T/ft.^2$. This difference in pressure would prevent direct comparison of the results. Mention was made of the fact that two different compactive efforts were used by Theophanides and that only one, the Standard Proctor, was used in this investigation. Thus only the data of Theophanides' obtained by using the Standard Proctor compactive effort can be used for a general comparison of the two investigations. As a result of the difference in procedures used in the two investigations, a rigorous comparison would seem to be unjustified. However, a general comparison is of considerable interest.

If a plot of log $\Delta e/1+e$ versus confining pressure for the data of the two investigations is made (Figure 22), it may be seen that under a confining pressure of 1.0 $T/ft.^2$ greater swelling occurred than under smaller confining pressures. This apparent paradox is probably the result of the differences in the initial moisture content when flooded. On this plot, for each point indicating swelling, the molding water content is shown. If we make the assumption that swelling should decrease as

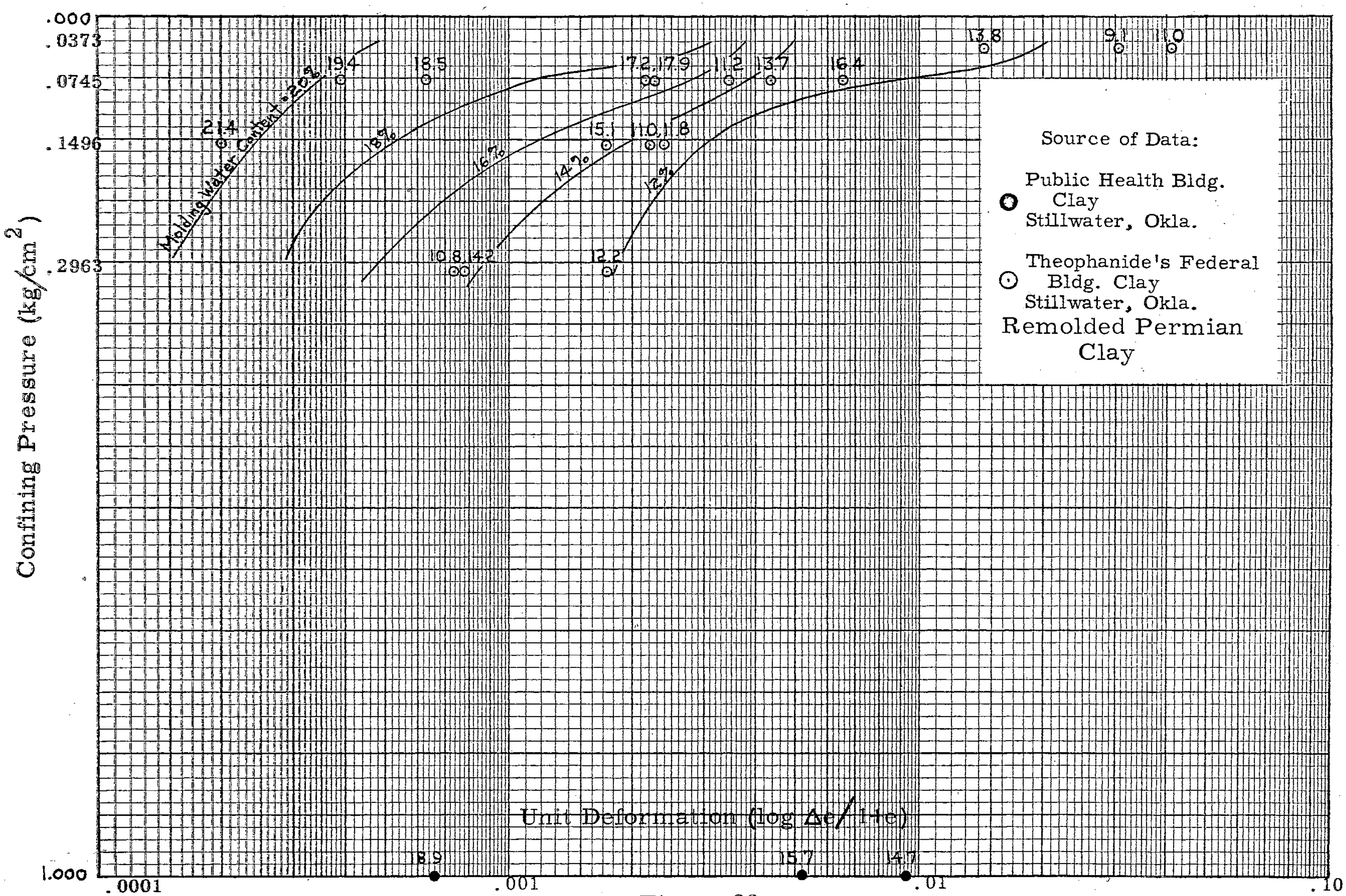


Figure 22

Relationship of Swelling and Molding Water Content molding water increases, under a constant confining pressure, a family of curves corresponding to different molding water contents can be drawn from the data. These curves indicate that with increased pressure and molding water content the swelling decreases which seems a correct relationship.

The data from this swelling investigation lends itself readily to a plot of molding water content versus swelling, as three specimens of the six swelled and three consolidated when confined under the constant pressure of 1.0 T/ft.^{2}. The data from the previous investigation on the other hand does not lend itself as readily to such a plot. However, if we make use of the contoured molding water content families in Figure 22 similar data for confinement pressures of 0.1 and 0.25 T/ft.^{2} could be obtained. For example, the data for 0.1 T/ft.^{2} is obtained as follows. By entering Figure 22 with the confining pressure of 0.1 T/ft.^{2} a unit swelling value may be read corresponding to each intersection of the 0.1 line with a molding water content contour line. The value of the unit swelling and the percent molding water content contour resulting in that value are recorded. The data for this investigation and the data for Theophanides' investigation obtained as indicated were plotted on a molding water content versus unit swelling plot (Figure 23). A smooth curve was constructed through data for the same confining pressures and in the case of the data from Theophanides' investigation an extension was made to the zero swelling axis. Two interesting conclusions may be drawn from the plot: 1) Drying of the clay prior to admitting water for swelling has an enormous effect on the magnitude of the swelling, and 2) The equilibrium moisture content varies inversely as the confining pressure.

On the same plot of molding water content and unit swelling a

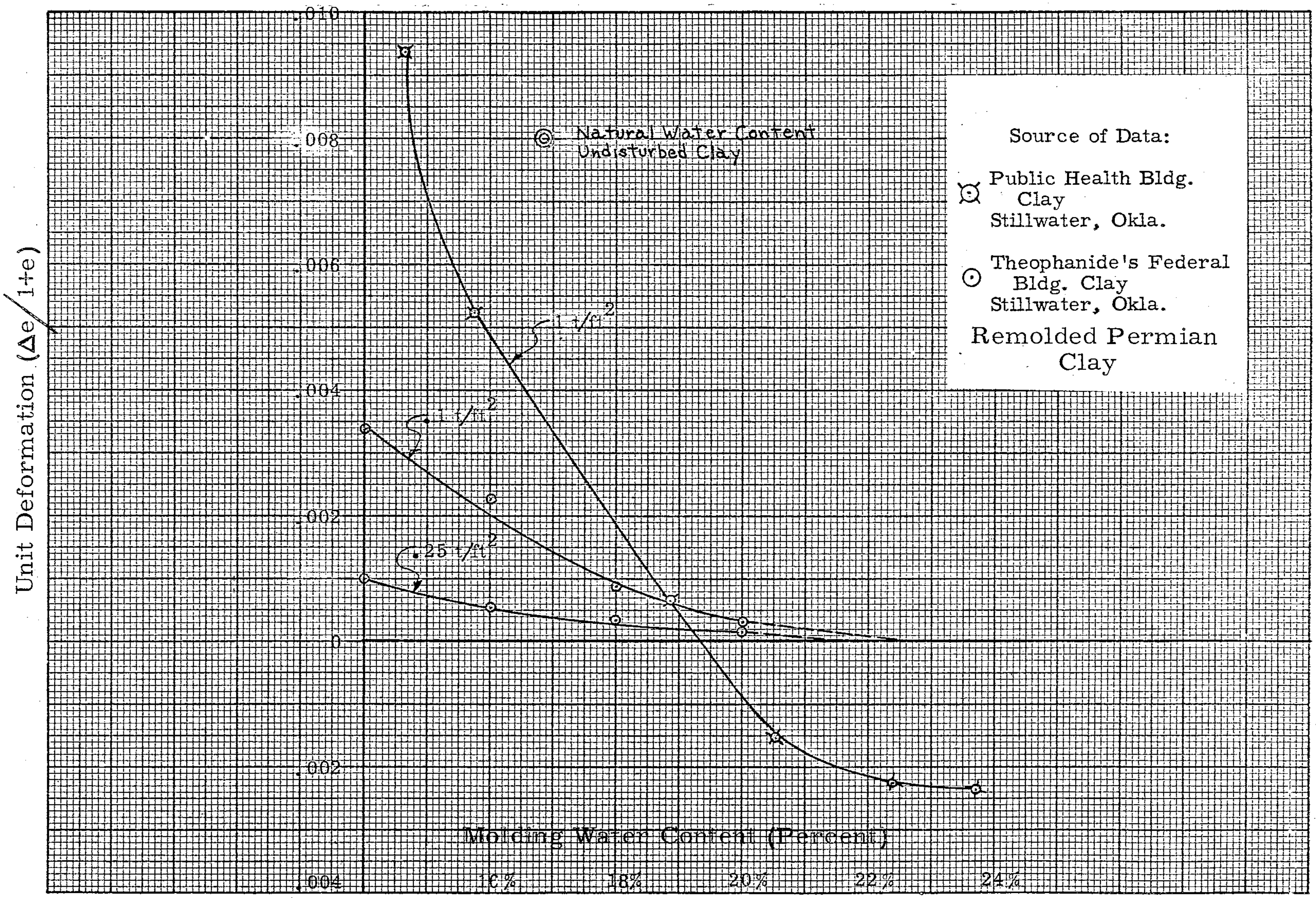


Figure 23

Relationship of Swelling and Molding Water Content for Constant Confining Pressures point comparison between the remolded clay and the undisturbed is shown. For a confining pressure of 1.0 T/ft.^{2} the unit deformation was obtained from Figure 20 for the undisturbed clay tested in this investigation. Then using this value and the natural water content of the undisturbed clay a point was plotted on Figure 23. This point is labeled undisturbed clay on the plot and indicates that the remolded clay compacted at optimum moisture content with a Standard Proctor compactive effort would swell under a confining pressure of 1.0 T/ft.^{2} about 87% as much as would undisturbed clay. From the results of test specimen 7, the no-swell test, it is also noted that for a sample compacted with Standard Proctor effort at a molding water content of 20.52% the average confining pressure for no swelling was 1.6 T/ft.^{2} or 80% as great as the undisturbed clay.

In literature recently published on remolded clay investigations, certain hypotheses were presented which could be compared to the data of this and Theophanides' investigations. The particular hypotheses which were compared to test results on these investigations were made by T. W. Lambe in a paper entitled "Engineering Behavior of Compacted Clays" presented in the ASCE <u>Soil Mechanics Journal</u> for May 1958. They are as follows:

1. The amount of compression which occurs under an increment of load when the pressure is small will cause a greater settlement of a wet-side compacted sample than on a dry-side compacted sample, if they are both initially saturated and at the same void ratio.

2. Under higher pressures the slope of the void ratio-log, pressure curve is steeper for the dry-side than for the wet-side samples.

The first hypothesis was checked against the data for specimens of both this and Theophanides' investigations. From Theophanides' data specimens 3-2 and 3-4 were compared while from this investigation specimens A-1 and A-5 were compared. The reason these particular specimens from each investigation were chosen is that each pair had one specimen compacted on the wet-side of optimum and the other on the dry-side, while the initial void ratio of both was about the same. The comparisons are shown below.

Δe Accompanying Incremental Pressure Increase

Theophanides' Data / This investigations' Data

Increment of pressure $T/ft.^2$	0.15-0.30	0.15-0.50	Increment of pressure $T/ft.^2$	1.0-2.0	1.0-3.0
$w_{dry} = 11.8\%$ $e = .6211$	+.003	-.018	$w_{dry} = 14.66\%$ $e = .367$	-.011	-.027
$w_{wet} = 18.8\%$ $e = .6217$	-.0097	-.0167	$w_{wet} = 22.4\%$ $e = .366$	-.01	-.024

As a result of this comparison it is observed that the data of Theophanides for the lightest increment of pressure agrees with the statement made, and that for increments of pressure greater than 0.5 $T/Ft.^2$ the statement seems to reach its implied limit.

Hypothesis number two states that the value of C_c, the slope of the virgin compression branch of a e-log. p curve, for a wet-side compacted sample should be less than the C_c of a dry side sample. If all the data on remolded Permian Clay that is available for this comparison is used, some 35 e-log. p plots, no conclusive verification of the second hypothesis is apparent. However, when the average value of C_c for all test was calculated and compared to the average C_c of the wet-side sample and dry-side samples this hypothesis is observed not to hold true. The average C_c of all tests was .167. For the wet-side test the average C_c was .17 while for the dry-side tests the average was .164.

SUMMARY AND CONCLUSIONS

CONCLUSIONS

This investigation had in particular three objectives. Each of these objectives, along with pertinent information and conclusions, is presented below. Since the characteristics of natural soil deposits may vary widely from one location to another, the conclusions presented should tentatively be viewed as being applicable only to the Permian deposits in the vicinity of Stillwater, Oklahoma.

(1). To determine the validity of an assumption which has been made in order to predict the swelling pressure an undisturbed desiccated clay will exert.

Conclusion: The results of this investigation indicate that the assumption or method of determining the no-swell confining pressure, gives results which are fairly reliable. This relationship can be observed by referring to Figure 19.

(2). To determine the amount of swelling which will occur under different loads or confining pressures.

Conclusion: The data from the Stillwater Region indicates a general pattern of deformation shown in Figure 20. From this general pattern a range of swelling deformations was surmised and is shown in Figure 21. Figure 21 thus allows the determination of the probable maximum to minimum amounts of total swelling which will occur under given confining pressures. A confining pressure of 2 $T/Ft.^2$ is indicated as the pressure which would prevent any volume change from occurring even if the samples were flooded with water. A simplified example of the application of Figure 21 is also given in the discussion.

(3). To determine the relationship between molding water content and swelling potential for remolded Permian Clay.

Conclusion:

1. The results of the series of tests indicate that swelling varies inversely as the molding water content up to the equilibrium or zero-swell water content.
2. The clay compacted by a Standard Proctor effort, confined under 1 T/Ft.^{2} has an equilibrium water content of 19.5% with 15% as optimum.
3. By comparison with previous results the equilibrium water content was observed to vary inversely with confining pressure for a Standard Proctor compactive effort.
4. The initial moisture content of the remolded clay influences greatly the swelling potential of fill areas.
5. A check of data indicates that under an increment of load when the pressure is small, a wet-side compacted sample compresses more than a dry-side compacted sample, if they are both initially saturated at the same void ratio.

SUGGESTIONS FOR FUTURE INVESTIGATIONS

The swelling of clay will always be an object of investigations as long as it creates problems for structures. It will especially be of interest to note developments of the new Physico-Chemical concepts of clay behavior. Several areas toward which research in the future could be directed are as follows:

1. Control of swelling by Salt Stabilization.
2. Lateral swelling pressure of clay.
3. Triaxial swelling investigation.

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