SWELLING AND INTRUSION CHARACTERISTICS OF UNDISTURBED PERMIAN CLAY

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SWELLING AND INTRUSION CHARACTERISTICS

OF UNDISTURBED PERMIAN CLAY

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PREFACE

The engineer is not a scientist. He must not merely determine and retain facts, see, visualize, formulate, and compute, but must use judgement and circumspection; he must estimate, weigh, decide. Mathematics is only one of his tools, no more important than experience, experimentation, and imagination. Particularly is this true for the foundations engineer who deals with materials and forces which are never quite uniform, and which are as universally important as they are unique. Much is unknown about them, and experimentation remains the main tool in gaining more knowledge and understanding of their behavior. It is employed in this thesis to gain some information on a particular soil in a particular area, and possibly to aid in the effort of overcoming the difficulties that this soil presents.

The author wishes to express his gratitude to several people who gave invaluable aid in this effort. Professor R. E. Means gave the research project its basic direction and scope, and his counsel sustained it through many difficulties. Almost daily conversations with Professor J. V. Parcher gave the author much of whatever understanding of the problem he has and helped in overcoming a variety of practical problems. Also appreciated is the cooperation of Mr. Hsu-Hwa Yie, who laid some important groundwork in the problem of intrusion, using compacted clays. Furthermore thanks are due Dr. C. A. Dunn, executive director of the Engineering Research and Experiment Station, who made available the funds and equipment needed for this research project.

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CHAPTER I

INTRODUCTION

General

While it is true, and important to remember, that the soil on which a building or other structure is to be erected must be considered as a construction material no less than steel, concrete, or other materials in the structure proper, these considerations are often quite different from those applied to the design and analysis of the superstructure. Factors that are not usually considered design criteria for steel or concrete, but should be so considered for soils, especially clays, are volume decrease during extended periods of time due to increased load (i.e. consolidation) and due to drying (i. e. shrinkage), the reverse process of volume increase (i.e. swelling), the change in strength due to climatic and weather conditions, and other properties inherent in the heterogeneous nature of a material composed of solid matter, water, and air in widely varying composition and structure. While a great many, if not all, of these factors have a bearing on this investigation, this research is primarily concerned with shrinkage, which shall here be defined as decrease in volume due to evaporation of water from the soil, and swelling, which shall be defined as the process of volume increase due to water entering the soil by gravity, capillary force, or adsorption.

Purpose of Investigation

Another facet that distinguishes manufactured materials, such as steel or concrete, from natural ones, such as timber or soil, is the fact that the former can, within limits, be produced to suit the user's requirements, while the latter must be used as nature provides them. Since the characteristics of soils, particularly cohesive soils, depend not only on their composition, but to a large degree on the manner of their deposition or formation, they vary widely according to geographical location. It is therefore not surprising that national and international coordination and cooperation in the investigation of the soil of a particular region falls far short of the effort put forth in the development of more universally used engineering materials.

The soil found in Oklahoma and northern Texas is dominated by a red Permian clay that may almost be considered unique, and the properties of which present a problem shared by only a few localities in the world. This clay, for reasons discussed later, is quite stiff and of high shear strength, so that consideration of bearing capacity in the design of foundations becomes routine and of subordinate importance. Due to its history and climatic conditions, however, the clay is capable of swelling against considerable loads when moisture is available during extended periods of time. This property presents a serious problem to architects and engineers. Many failures creating extensive and costly damage, a number of them in buildings on the campus of this university, are attributed to this cause. Widespread disagreement and doubt still exist as to the precise causes of this phenomenon, the forces and deflections involved, and the best means of combatting them. The schools of Architecture and Civil Engineering have for years been engaged in the search for causes of and solutions to this problem. This investigation may be considered a part of this sustained effort to put into the hands of Oklahoma engineers and architects better tools for the design of foundations.

Scope of Investigation.

In practically all cases buildings in Oklahoma are erected on clay layers of only partial saturation. This fairly dry state of the soil is caused by repeated and prolonged evaporation of water from the surface in a semi-arid climate. Since after construction the building will largely prevent evaporation from continuing, the clay will absorb additional moisture, even during dry periods, and retain it thereafter. The clay will, therefore, tend to swell, and unless proper precautions have been taken, the resulting pressure against the foundation elements will cause heaving damage.

One method, aimed at prevention of such damage, that has been proposed by builders during recent years, entails the placing of a layer of gravel several inches thick below the sole of the foundation and immediately above the clay. The theory, on which this practice is based, is that the clay, when swelling, will penetrate or intrude into the voids of the essentially incompressible gravel rather than lift the foundation. This method is rather untested, it has been adversely criticized, and is the subject of wide disagreement as to its applicability.

The purpose of this investigation, then, is to determine the practicability of this method by studying two distinct aspects of the behavior of this soil:

a. The amount of swelling of laterally confined undisturbed Permian red clay against certain imposed pressures.

b. The amount of intrusion of the same clay into layers of various types of gravel with respect to time and to increasing pressure.

The writer hopes to determine by theoretical consideration whether the reduction or prevention of heaving by the use of gravel beds is an impossibility or a method that may be used successfully under certain conditions. He further hopes to provide, by means of laboratory investigation, a starting point for further research in this direction by theoretical deduction, by laboratory testing, and by experience in the field.

Previous Investigations

Investigation of consolidation and swelling characteristics of Permian red clays has been in progress at Oklahoma State University for a number of years. Research has been conducted on both undisturbed and remolded samples of this clay, and has been applied to the design of building foundations in this region.

In 1950, R. E. Means, W. H. Hall, and J. V. Parcher published a bulletin describing design criteria for foundations on Permian clay (5)*. This subject was later extended in another bulletin by R.E.Means to include exploration methods (4).

Specific investigations closely related to the subject under con-

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* Numbers in parentheses refer to bibliography at end of text.

sideration are covered in several Master's Theses, compiled and submitted at Oklahoma State University. Of interest are William H. Hall's investigations of Permian desiccated clays (2), swelling tests on remolded clay by P. J. Theophanides (10), and swelling tests on undisturbed samples of this clay by A. M. Midani (6).

Also of interest is a report by Hsu-Hwa Yie, in which he describes intrusion tests on remolded Permian clay (12). The research for this report was conducted by Mr. Yie simultaneously with the author's investigations of undisturbed samples reported on in this thesis.

Recent Related Research

Since, as previously pointed out, expansive clays are prevalent in a number of locations around the globe, a review of some research conducted elsewhere in this direction will be of interest here.

a. Apparatus for Measuring Swelling Pressure.

At the Israel Institute of Technology, I. Alpan has developed an apparatus for measuring swelling pressures of compacted soil specimens in a rather novel manner (1).

A sample of soil, compacted into a swell pressure cell mounted on a triaxial machine adapted for the purpose, was allowed to take up moisture through a porous suction plate. The amount of water taken up was controlled by applying a regulated partial vacuum to the water supply. The sample was then allowed to swell against a proving ring at various degrees of saturation.

Mr. Alpan, by this arrangement, meant to overcome two features of conventional tests conducted to this end that he considered objectionable: (a) full, uncontrolled saturation as compared to moisture variations below full saturation in nature, and (b) the necessity to employ various compaction energies to obtain a range of initial conditions, thereby altering the structural arrangement of soil particles.

The tests indicated that further experimentation of this nature is warranted. The writer believes that swelling tests of this kind, allowing variations of moisture content below full saturation to be controlled, would indeed be a valuable supplement to the research program of expansive clays in the Southwest.

b. Determination of Effective Shrinkage.

Experiments to measure the shrinkage rather than the swelling forces of clayey soils were carried out by R. M. Palit and S. S. Joshi in India (7).

Their test apparatus consisted of a rubber tube filled with water which was surrounded by a compacted soil sample inside a mould. The rubber tube, through a stopper in one end, was connected to a syringe, which in turn was fitted with a piston and filled with mercury. The piston rested against a proving ring.

As the specimen dries in an apparatus of this type, it shrinks toward the center, and the resulting pressure is transmitted through water, mercury, and piston to the proving ring, where it is recorded.

Only three such tests were recorded, but two results could be stated in general terms:

(1) Only a very small part of the shrinkage pressure of a soil is exerted on any adjacent structure, most of it serving to compact the soil.

Shrinkage, then, is much less a critical factor in foundation design than swelling.

(2) When the soil cracks while drying, the pressure exerted by it on the structure will rapidly be reduced to zero.

CHAPTER II

ORIGIN AND PROPERTIES OF MATERIAL

The writer believes that a clear understanding of the origin and main physical properties of the Permian red clay is mandatory, but he also realizes that much has been published on this material and that a detailed description here would result in unwarranted duplication. Therefore this discussion will be confined to a brief and general description of those properties that have a direct bearing on this rerearch project. References will be given to aid those readers interested in obtaining more specific and comprehensive information.

Geologic Origin

A considerable portion of Oklahoma and the Southwest is covered by beds of Permian red clay. These originated in a salt water sea, which covered this general region during the Permian period, and into which deposits of soil were carried from the mountains by torrential rains of short duration. Later, towards the end of the Permian period, the sea dried up three times, leaving each time deposits of gypsum and salt with layers of clay and sandstone.

These deposits were later covered with an overburden of several hundred feet from the Mesozoic and Cenozoic eras. This overburden has vanished by erosion since that time. The clay layers were, of course, heavily consolidated under this great overburden, but the precise value of this preconsolidation pressure has not yet been reliably determined.

A comprehensive description of Permian soils in the Oklahoma area has been presented by R. E. Means (4).

Structure of Soil

The Permian red beds in Oklahoma consist largely of red and yellow clays (4), the subject of this investigation. These layers vary greatly in thickness, reaching depths of as much as 100 feet in some locations. Obviously no very reliable foundation design is possible without some subsurface investigation at the building site.

The nature of these clays is explained by the nature of their formation and their further history. After the soil had been carried to the sea by rapid rain waters, during which process the flow was too fast to permit segregation of the soil particles by grain size, the salt water caused the clay particles to flocculate and settle out together with any sand or silt particles interspersed with them. A clayey soil of flocculent structure was thus formed, which generally constitutes the Permian red beds. Due to variance in the rapidity of the flow of water, however, clay, sand, and silt were deposited together in varying proportions, so that the composition of the soil varies greatly according to location . Some clays contain but little sand or silt, and these are usually badly fissured. The clay tested for this thesis is of the latter type. It is emphasized that no claim is made that the results of these tests apply to any other clay. Careful determination of the clay at any particular building site is desirable and is imperative for important structures.

It is remarked here that due to lateral confinement, both in the swelling and intrusion tests described herein and under field conditions, the fact that the clay is fissured is of little or no significance, except as it affects the rate of reaction.

Because of the great overconsolidation under the heavy overburden pressure imposed on the deposits after the Permian period and due to many cycles of drying and wetting to which the clay has been subjected in a semi-arid climate, the clay is very stiff and capable of supporting substantial loads.

Consolidation

It follows from the preceding that these desiccated Permian clays may be said to have two preconsolidation pressures, one due to shrinkage forces and the other due to previous overburden. Over the years many consolidation tests were run in the Soil Mechanics Laboratory of Oklahoma State University with the purpose of determining these preconsolidation pressures.

In "Soil Investigation for Building Foundations" (4), R.E.Means reports the results of one such test on a sample of undisturbed Permian red clay, which in essence were borne out by a considerable number of other tests. It was found that pressures of 3.5 to 4.5 tons per square foot were required to recompress samples to the void ratio to which they had been reduced by shrinkage. This value then may be considered the preconsolidation pressure due to shrinkage. It was found to coincide very nearly in many cases with the preconsolidation pressure as determined by the Casagrande method from the void ratio - log pressure curve.

On the other hand, the preconsolidation pressure due to previous overburden was judged to lie between 20 and 30 tons per square foot, corresponding to approximately 400 to 600 feet of overburden, which roughly agrees with the estimate of geologists. However, the test apparatus available for consolidation is limited to a load of little more than 1500 kg, so that the use of consolidation rings sufficiently large to offset the effect of friction along the walls limits the applied pressure to the order of 20 tons per square foot. At pressures of this magnitude the e - log p curve is not sufficiently straight to allow the employment of A. Casagrande's method for determination of the preconsolidation pressure.

Other values derived from the test referenced above and cited here for completeness are:

Coefficient of Permeability k : 0.286×10^{-9} cm/sec,

Compression Index C_c : 0.156,

Swelling Index C_s : 0.064,

and a rebound of 62 per cent of the total deformation after the pressure was released from 38 to 0.15 tons per square foot.

Shear Strength

Due to the fact that the clay dealt with here possesses great shear strength because of the aforementioned heavy preconsolidation loads, little attention has been devoted to shear tests of this clay in the past. W. H. Hall (2) reports that he attempted triaxial tests, but that he encountered difficulties of such magnitude in the preparation of samples, and that as a result his data were so erratic, that reporting them seemed futile.

In problems involving intrusion, however, shear strength does assume importance, since in this case the clay is intended to fail locally in shear. A few triaxial compression tests were therefore conducted, the results being reported later in this thesis. Since only one clay was tested, however, and since furthermore the samples for compression and intrusion tests were taken from different locations (Stillwater and Oklahoma City, respectively), any attempt here to relate shear strength and intrusion values would be insignificant and worthless. It is hoped that the values recorded will be of some assistance in future research directed toward establishing some such relationship.

CHAPTER III

THEORETICAL CONSIDERATIONS

Shrinkage and Swelling of Clay

Since the solid particles comprising clay are very minute (smaller than 0.002 mm according to the MIT classification system), the voids, which must be thought of as interconnected passageways or pores, are correspondingly small in diameter, although their size varies. When a mass of clay, fully saturated and possessing free water on the surface, starts drying out, by evaporation of water from the surface, the surface of the water starts receding into these passageways which now act as capillary tubes. As the water further recedes into smaller pore spaces the tension in the water increases as the radii of the pores decrease. This, of course, is due to the fact that smaller pores permit smaller menisci, and that the capillary tension in the water is inversely proportional to the radius of the menisci, the latter being equal to the radius of the largest pore in the system. The tensile stress in the pore water produces the same effect as externally applied compressive pressure.

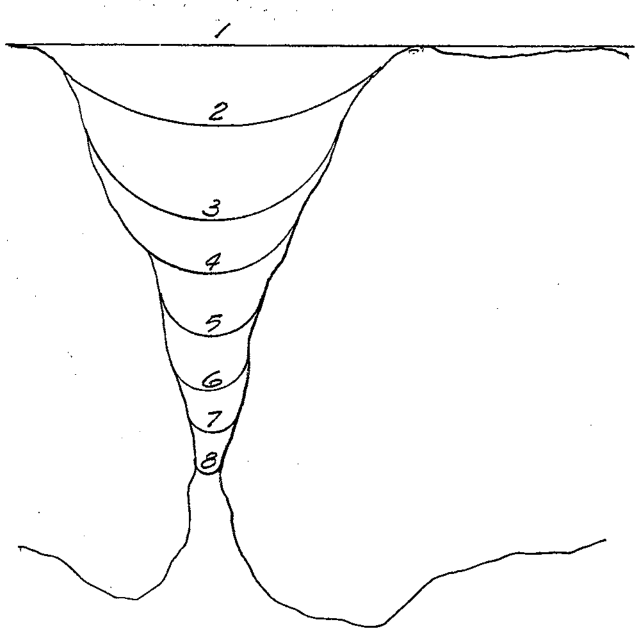


Figure 1.

Capillary Pore in Clay

Figure 1 shows successive positions of a meniscus receding into a pore.

Apparently, for a given arrangement of soil particles, increase in pore water tension ceases when the menisci have reached the smallest pore space in the soil mass. The clay particles, however, being acted on by tensile forces, will tend to move closer together, thereby decreasing the radii of the pore spaces and, as a result, increasing the tension in the water further. This process will repeat itself a number of times, but can not continue indefinitely. As the particles move closer together, they tend to form a structurally stronger system, until this system can withstand the corresponding maximum tension in the pore water. At this point the decrease in volume, which, of course, accompanies the process described above, will cease. The water content at this point was defined by Atterberg as the shrinkage limit of the soil. Due to the small size of the particles, the menisci need only recede a very short distance from the surface to achieve this condition, so that the soil may be considered saturated at the shrinkage limit. Further decrease in moisture content does not, according to this theory, result

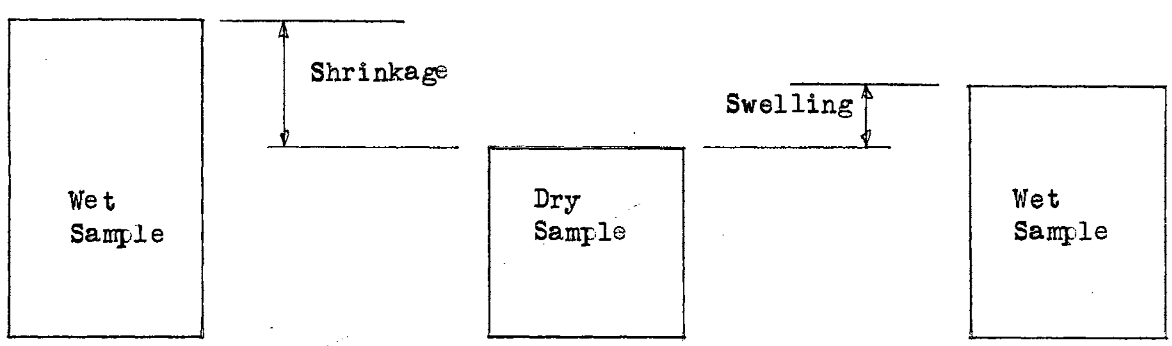


Figure 2.

Successive Shrinkage and Swelling of Clay Sample.

in further shrinkage.

When free water is now made available to the clay mass, it can be observed that the latter tends to approach its original volume by swelling, but does not necessarily reach it (fig. 2). The question arises as to what causes this increase in volume and what determines its magnitude.

It was previously theorized that the individual soil particles rearrange themselves in some manner during shrinkage in order to withstand the added tension in the water. If this rearrangement is, wholly or partially, effected by grains slipping past one another, then it is quite unlikely that this relative movement will be reversed by filling the pore spaces with water; the stress in the water is merely neutralized, not reversed, by changing from tension to zero stress rather than compression (the hydrostatic pressure is negligible). This slipping action then, if present, results in permanent or inelastic deformation.

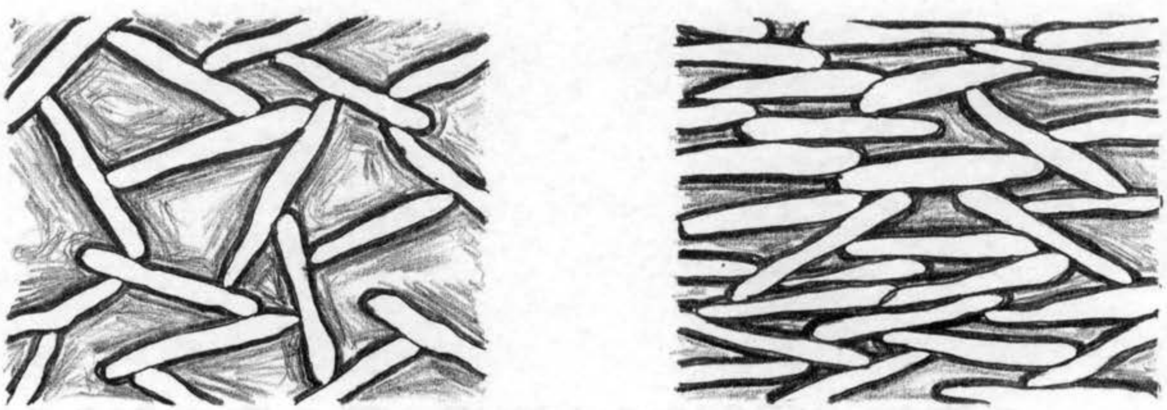
On the other hand, the clay particles may rearrange their structure by internal deformation rather than relative movement, i.e. the particles, of flaky shape, may bend or twist. Since clay is largely composed of minerals, such as kaolinite and montmorillonite, which possess almost perfectly elastic behavior somewhat like mica flakes, this type of deformation will be recovered when the pressure that induced it is removed. Therefore, when pore water pressure reverts to zero, the clay particles assume their original positions, lending elastic behavior to the soil mass as a whole. Swelling takes place.

Finally, electromagnetic forces may contribute to elastic rebound when water is made available. Electric charges, usually negative, are concentrated along the surfaces of most of the scale-like particles comprising clay. Water molecules, being bipolar, are strongly attracted to these particles and orient themselves on their surfaces. This electromagnetic orientation lends the water much greater viscosity than normal. Immediately adjacent to the surface of the particle, where the water molecules are perfectly oriented, the water approaches the solid state, and this layer is often referred to as adsorbed water (3). At some greater distance from the particle, the orientation becomes less perfect and hence the water less viscous until it finally reaches normal viscosity. The layer between the adsorbed water and the normally viscous or free water may be called the double layer. The latter is usually much thicker than the layer of adsorbed water and varies in thickness for different minerals. T. William Lambe gives the following dimensions for two of the more common minerals (3):

	Montmorillonite	Kaolinite
Length of particle	1000 Å	10000 Å
Thickness of particle	10 Å	1000 Å
Adsorbed water	10 Å	10 Å
Double layer	200 Å	400 Å

It is evident that the adsorbed and double layer water accounts for a substantial portion of the total volume of the clay sample, and is capable of causing considerable shrinkage when it is removed by pressure or evaporation. Conversely, when water becomes available to a dry clay, its great electromagnetic attraction to the mineral particles will surely cause swelling, even against fairly large imposed pressure.

Of course, all the free water around a particle must be removed by either consolidation or desiccation before any of the more viscous water can be extruded, but some question exists in the author's mind whether all the free water in an entire sample would necessarily have to be evaporated before any of the tightly held water can be removed. If this is necessary, then shrinkage and swelling by this process become possible only after desiccation has reached a point far beyond the shrinkage limit; if not, then shrinkage by removal of adsorbed water will commence as soon as a sample of soil has been dried out to the shrinkage limit. At any rate, it becomes evident that if the electromagnetic forces come into play, not all shrinkage ceases at the so-called shrinkage limit, but volume decrease could possibly continue until the entire clay structure is supported by mineral - to - mineral contact. At this point, which might be referred to as the ab-



(a) Undisturbed (b) Remolded

Figure 3.

Undisturbed and Remolded Clay in Mineral-to-Mineral Contact.

solute shrinkage limit, the moisture content will be very low, but undoubtedly higher for undistubed than for remolded clay (see fig. 3).

Summarizing, it appears reasonable that shrinkage of clay will occur due to (a) slippage of particles past one another, (b) deformation of elastic mineral flakes, and (c) squeezing out and evaporation of double layer and possibly adsorbed water. The first of these processes is irreversible and undoubtedly ceases after a number of drying and swelling cycles, since the particles will eventually form a structure of maximum strength. This view is supported by the fact that no permanent downward movement of points on the surface of clay beds has been observed. The other two actions are reversible and can correspondingly be considered to be perfectly elastic.

Thus swelling action depends on the mineral composition of a particular soil, the structure of the soil, and the amount of water available.

Since this is not a treatise on the behavior of particles, but rather an investigation of the action of a particular type of clay, it will be attempted to find an answer to the following questions by laboratory testing methods:

What effect do repeated cycles of drying and wetting have on a non-desiccated clay of this type?

Against what imposed pressures is a desiccated clay capable of swelling and by what amount?

What applied pressure is necessary to recompress the clay to its shrinkage volume after flooding?

With the answers to these questions known, a more accurate estimate can be made as to the swelling action of Permian clay against the foundation of a building, and means of preventing differential heaving can be developed. This is the ultimate goal of this investigation.

Foundations on Permian Clay

During recent years a number of methods and criteria for the design of foundations on Permian red beds aimed at overcoming or combatting differential heaving have been developed and employed with varying, but often good, success. Some of these are listed in the following.

1. Independent footings of the spread or bell-bottom type are placed well below the surface near the water table or, at any rate, at a depth where periodic drying of the clay seems impossible to occur. This is an excellent method of preventing heaving, but with the water table often at a depth of 50 feet or more, the cost of drilling or digging is quite high. Thus the expense for a foundation of this type, especially in the case of light buildings, often becomes prohibitively large in proportion to the total building cost.

2. Footings are so designed in size and location that their sole pressure counteracts any tentency to differential heaving. When one considers that the sole pressure is limited by the safe bearing pressure and that pressure decreases rapidly with depth, while the swelling pressure fails to do so significantly, it becomes evident that this method contains many shortcomings and difficulties, and

that its application is definitely limited. In this as well as the preceding method, proper precautions must be taken to prevent the piers from being lifted by shear action of the clay along its surface. This may be achieved by either providing a void space between pier and clay or separating them by some material of very low shear strength.

3. A third method takes advantage of the fact (known from experience) that clay generally does not dry out under a building once it is saturated, since floor slabs form an effective barrier against evaporation. A dike is erected around the construction site of a proposed building and the enclosed area flooded with water, which is maintained until the underlying clay has been fully saturated. At that time the foundation may be constructed without fear of heaving. The clay being highly impermeable, however, many years are required before all swelling will cease. Certainly a flooding period of a year or more is needed for even a fairly successful application of this method. The inherent economic drawbacks are obvious.

4. Finally, an idea has been advanced which would also make use of the experience that clay swells only once after a building has been erected on it, while it would avoid the loss incurred by letting a site go unused for years while flooded. The footings, and possibly other foundation elements, are cast not immediately on the clay, but rather on a bed of gravel several inches thick. It is desired that when the clay subsequently swells, it will intrude into the voids of the essentially incompressible gravel rather than lift the structure. The clay, of course, has two alternatives to this action. It can either, when sufficiently stiff, refuse to penetrate into the voids by

a sufficient amount, or it can, when sufficiently weak, intrude into the gravel when the building is being erected without waiting for any swelling action to take place. This method of intrusion, though quite untried, has the advantage of small cost if and where its application is possible, and therefore deserves investigation. It is the subject of this initial research project, and its theoretical aspects will be discussed further in the next section.

First, however, some remarks on basement floor systems will be desirable, without which this chapter would not be complete. System 3 is the only one that allows the use of conventional concrete slabs cast on ground without special precautions, since the pre-saturated ground will exert no swelling pressure against them. In systems 1 and 2 slabs of this type would certainly crack, and have in fact done so innumerable times, when the underlying clay begins to swell. A floating slab independent of beams and columns would be a better solution, but would also present a number of architectural disadvantages and difficulties. A third possibility entails the use of a structural slab or joist system separated from the ground by an air space several inches high. A report and design guide on the most feasible of such systems (in the opinion of the writer) is at present being compiled by Professor Dean Irby and the writer. System 4, the subject of this thesis, also demands void spaces under slabs, since the pressure necessary to force intrusion of clay into gravel will generally be too great to be resisted by a slab, as will be shown later.

Intrusion

As stated previously, the protection afforded by an underlying gravel sheet against heaving of a footing or foundation element of any kind necessitates the entering of clay into the void spaces of the gravel when swelling occurs. This action is here defined as intrusion. There exists the possibility of making use of intrusion in two distinctly different ways, although without experimental data no information on the effectiveness of either is available. Firstly, any swelling effect may be counteracted by intrusion if swelling takes place at the same rate with respect to time as does intrusion. Secondly, differential heaving may be combatted if, due to the stiffness of the superstructure, lifting of one particular footing would result in increased pressure under that footing of such magnitude that an equal amount of intrusion would take place. These two possibilities will be discussed and investigated in turn.

The expected rate of swelling of the clay under a proposed building can be roughly estimated when swelling tests have been conducted, subsurface investigations carried out, the amount of available water estimated, and appropriate calculations made. The problem remains to find the conditions under which a rate of intrusion equal to the rate of swelling can be achieved. Without need for experimentation several effects of the various variables involved can confidently be estimated in general, qualitative terms.

Increased pressure will result in increased intrusion, but not necessarily in an increased rate of intrusion. The pressure can be varied only by redesign of the foundation.

Since clay becomes weaker with the addition of water, the supply of moisture will more effectively contribute to intrusion when entering the clay from the gravel sheet rather than from some depth by capillary action.

The type of gravel used may well be the single most important variable in the design. Experience suggests that for easy intrusion the gravel should have a high void ratio, large pore openings, few flat surfaces, smooth surfaces, and resistance to crushing. For maximum intrusion, then, the gravel should be poorly graded, very coarse, wellrounded, smooth, and strong.

So many factors, varying from case to case and largely unknown, enter into the behavior of clay and gravel that any detailed or quantitative theorizing on this subject seems a thankless and futile task, except as it may be used to interprete experimental data. Some such data will be reported in the next chapter.

In summary, the rate of intrusion should be set equal to the rate of swelling and may be adjusted by proper selection of contact pressure, type of gravel, and, possibly, supply of moisture.

The other method, based on increase in pressure, contains no possibility of cancelling out heaving unless it is combined with the procedure described above. It has as its aim the compensation for differential swelling only, which may occur due to different pressures under the various footings, due to varying thickness of the clay bed under the building, or due to different rates of moisture supply and evaporation. The principle is applicable under two distinct conditions. On the one hand, differential heaving of the various footings may occur.

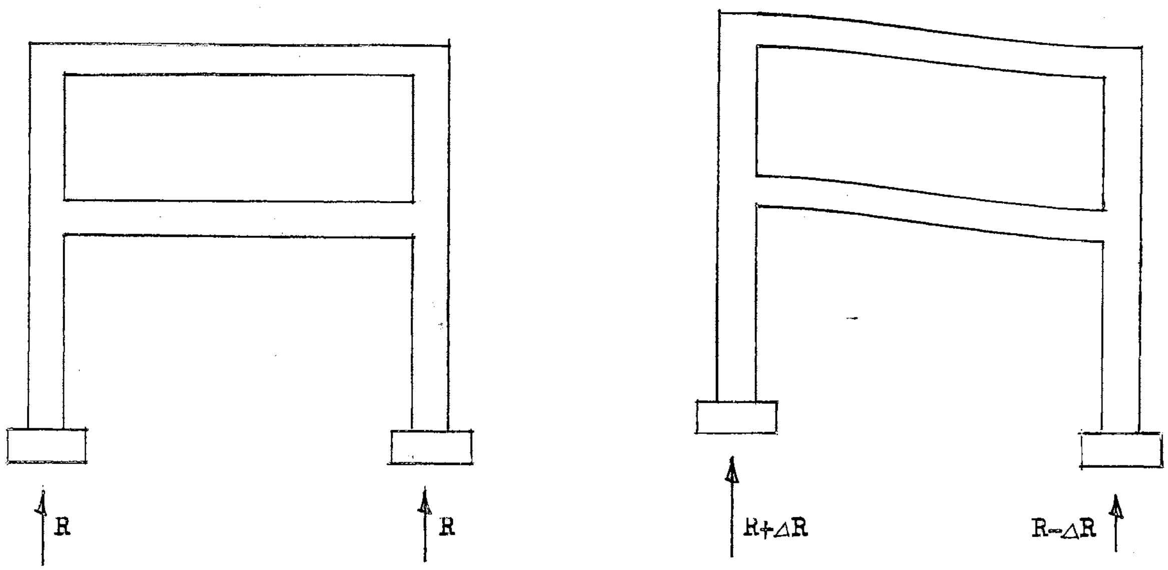


Figure 4.

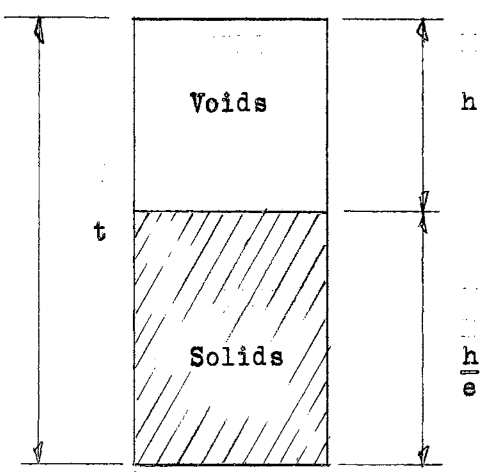
Increased Footing Load due to Heaving Deformation.

Particularly interior footings may be subjected to a more rapid rate of heaving than the exterior ones. On the other hand, exterior footings often experience greater heaving on the inside than on the outside. The reason, of course, is that in both cases surface evaporation from the area immediately surrounding the building influences heaving of exterior foundation elements by tending to keep the moisture content of the clay under them lower than that of the interior clay where no evaporation can take place.

In both cases the flexural rigidity of the superstructure, mainly of the girders in the first case and of the column in the second, will cause increased stress to be exerted on the element where the most heaving is taking place. If gravel has been placed under this element, increased intrusion will no doubt take place due to the increased pressure, and ideally the amount of heaving will be completely compensated for by an equal amount of intrusion.

To make this possible, the increased pressure exerted by the superstructure due to a certain displacement of a particular footing must be known, as well as the increase in pressure required to cause additional intrusion by the same amount as the displacement. The former information will be available from the structural analysis of the building by any of the methods in common use, but the latter value will have to be determined experimentally from intrusion tests of the pressure-deformation type as discussed in the following chapter.

In any of the above methods, care must be taken to select a gravel bed thick enough to accomodate the increased volume of clay due to total or differential swelling, as the case may be. The thickness of gravel t, the void ratio of the gravel e, and the expected amount of heaving h will have the following relationship (fig. 5):



$$t = h(1-\frac{1}{e})$$

Figure 5.

Gravel Layer and Heaving Relationship.

CHAPTER IV

LABORATORY RESEARCH

General

Laboratory investigations reported here were conducted in the Soil Mechanics Laboratory of the Oklahoma State University. Their primary purpose was the determination of pressures and volume changes involved in swelling of desiccated Permian clay and in intrusion of this same clay into various types of gravel. This purpose stemmed from the quite practical desire of obtaining quantitative as well as qualitative information on the practicability and pertinent design criteria of the method of combatting heaving by intrusion as discussed in the preceding chapters. Some indications of a more theoretical nature may also be gleaned from the test results, particularly from those concerned with swelling and shrinkage of clay. The intrusion tests, on the other hand, were of a very exploratory nature, being the first experiments of this type to be conducted to the knowledge of the author, and admittedly lacked in refinement. Many more tests of a similar nature will have to be conducted before it will be possible to develop a reliable theory of this process. It is hoped that the tests described here will aid in future experimentation of this kind.

Samples

All samples of clay tested were obtained from locations in Oklahoma City and Stillwater, Oklahoma. They represent the reddish brown clay typical of this region, a clay that is heavily fissured and contains small rock fragments of up to about 3 mm average diameter, distributed throughout its mass. These two facts, combined with the great stiffness of the clay, make the preparation of samples exceedingly difficult. The samples tested represent the author's best efforts in this direction, but it is pointed out that some disturbance along the boundary surfaces is characteristic of all of them. To what degree these disturbances are reflected in the test results is unknown, but due to the dimensions of the samples used, the errors introduced are not believed to decrease the overall accuracy seriously.

Rough samples were taken manually from excavations by cutting a groove all around a block of clay. The stiffness of the clay makes

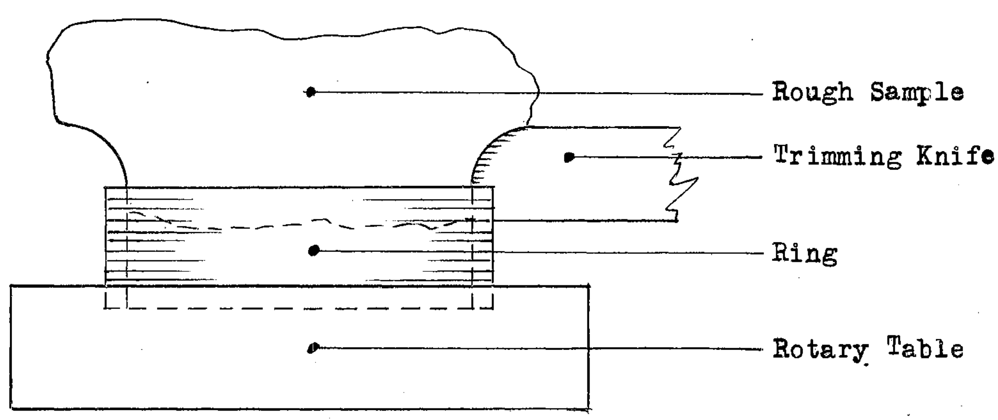


Figure 6.

Sample Preparation by Turntable.

this method necessary if the best possible samples are to be obtained.

Two methods were employed in the preparation of specimens for consolidation tests.

In the turntable method, the consolidation ring is placed on a rotary lathe, and the clay sample, roughly cut to size, is set on the ring. With the aid of a steel blade, featuring a curved cutting edge and an offset for guidance along the ring, the sample is trimmed to fit the ring. After complete penetration, the sample is trimmed off flush with the ring by means of a straight edge (fig. 6).

Using the sharpened ring method, a consolidation ring with a sharpened cutting edge is placed on the rough clay sample. By alternately carefully trimming the specimen with an ordinary paring knife to approximately the size of the diameter of the cutting edge and exerting slight pressure on the sample, the latter is gradually driven into the ring. It is then trimmed off flush as before.

The preparation of compression samples for triaxial tests was performed in much this same manner, with a tube of smaller diameter than the ring and much greater height. The completed specimens were extracted from the sampler with a piston, and placed into a rubber membrane for testing.

Shrinkage and Swelling Tests

Shrinkage and swelling tests were conducted by placing samples of about 2 cm height and 10 cm diameter into a consolidation machine. In this machine two decimal beams are employed in series, so that a lever arm ratio of 100 : 1 is achieved. A movable fulcrum allows adjustment for deformation. The sample in its ring is placed between two porous stones of slightly smaller diameter than the specimen. This floating-ring type container allows compression of the sample from both sides, thereby minimizing the effects of friction along the walls. The apparatus is placed into a bowl in order to supply an unlimited amount of water to the sample for saturation, and load is applied through a head to the top stone. A dial is placed against the head to allow measurement of deflection.

Three series of tests of this type, designated A, D, and G, were concluded, another had to be abandoned because of excessive loss of soil from the consolidation ring. Series A consisted of six consolidation tests after each sample was allowed to dry out and then swell under pressures varying from 0 to 5 ton/sq. ft. Series D consisted of four samples subjected to a number of drying and swelling cycles under pressures varying from 1.2 to 4.8 tons/sq.ft. Series G was another consolidation test, but using the four samples of series D after allowing them to swell under their original loads. A description of these tests and the presentation of their results follows.

Series A. The samples were taken from a basement excavation on Ridge Road, Stillwater, Oklahoma, at a depth of 5 feet below grade. One sample (A-1) was prepared with a sharpened ring which was subsequently used for testing, and the remaining five samples (A-2, A-3, A-4, A-5, A-6) by the use of the turntable. The samples were placed into the consolidation machine at their natural water content and dried out under the slight load of 0.025 tons/sq.ft. After about three weeks all shrinkage had ceased and pressure was applied as shown:

Sample A-1	-	0	tons/sq.ft.
A-2	-	1	"
A-3	-	2	"
A-4	-	3	"
A-5	-	4	"
A-6	-	5	"

The samples defamed further under these loads by some small amounts. After one day, water was added to allow swelling. However, all but sample A-1 decreased further in height. It was noted that the entering water had succeeded in slaking a certain amount of soil from the shrinkage cracks that had formed between the samples and their rings, in all cases but A-1. The soil, deposited at the bottom of the bowl, was later carefully collected, dried, and weighed. Making the assumption that the weight of this residue was related to the dry weight of the entire sample as the deformation due to its loss to the height of the sample, a correction was computed and applied to the readings. The result showed that A-2 also swelled, and that the consolidation of the remaining samples amounted to much less than indicated.

Now, at intervals of three days, the pressure on each sample was approximately doubled up to a maximum of about 20 tons/sq.ft., the capacity of the machines. It was then reduced successively by three fourths down to the swelling pressure indicated above and finally to 0.025 tons/sq.ft. Readings for each load taken one day (1440 minutes) after each load was applied, at which time primary consolidation had ceased, were used to compute void ratio, except that at the final, smallest pressure a reading was also taken after about two weeks of swelling against this pressure. These values were used to plot an e - log p curve for each sample (figs. 7 to 12). C_c and C_s by definition represent the slopes of the virgin and swelling branches of these curves, plotted to a semilogarithmic scale.

It is seen from the curves that in the case of samples A-1 and A-2, the only ones that swelled at all, the pressure required to recompress the specimens to their shrinkage volumes was approximately 1.5 kg/cm^2, a value that is much lower than corresponding data found by A. M. Midani (6) and others for similar clay. The other samples, flooded under pressures exceeding 1.5 kg/cm^2 failed to swell at all, a fact that seems to bear out this rather surprising result. Furthermore, the preconsolidation pressure of A-1 as determined by the Casagrande method is only about 0.36 kg/cm^2 or 720 lbs./sq.ft., a far different value, and one that is strikingly similar to the pressure induced by the overburden of about 5 feet at the site, which would amount to roughly 700 lbs./sq.ft. Finally, the site from which the sample was taken is located near the bed of an intermittent stream, so that the combination of all these factors strongly suggests the possibility, almost certainty, that this clay has not been desiccated innumerable times and may be a fairly recently deposited, normally consolidated clay. The fact that all the e - log p curves of this series are very nearly straight in their terminal ranges supports this conclusion.

Series D. The clay for this test was taken from a depth of 8 feet below grade at Shartel & 38th Streets, Oklahoma City, Oklahoma.

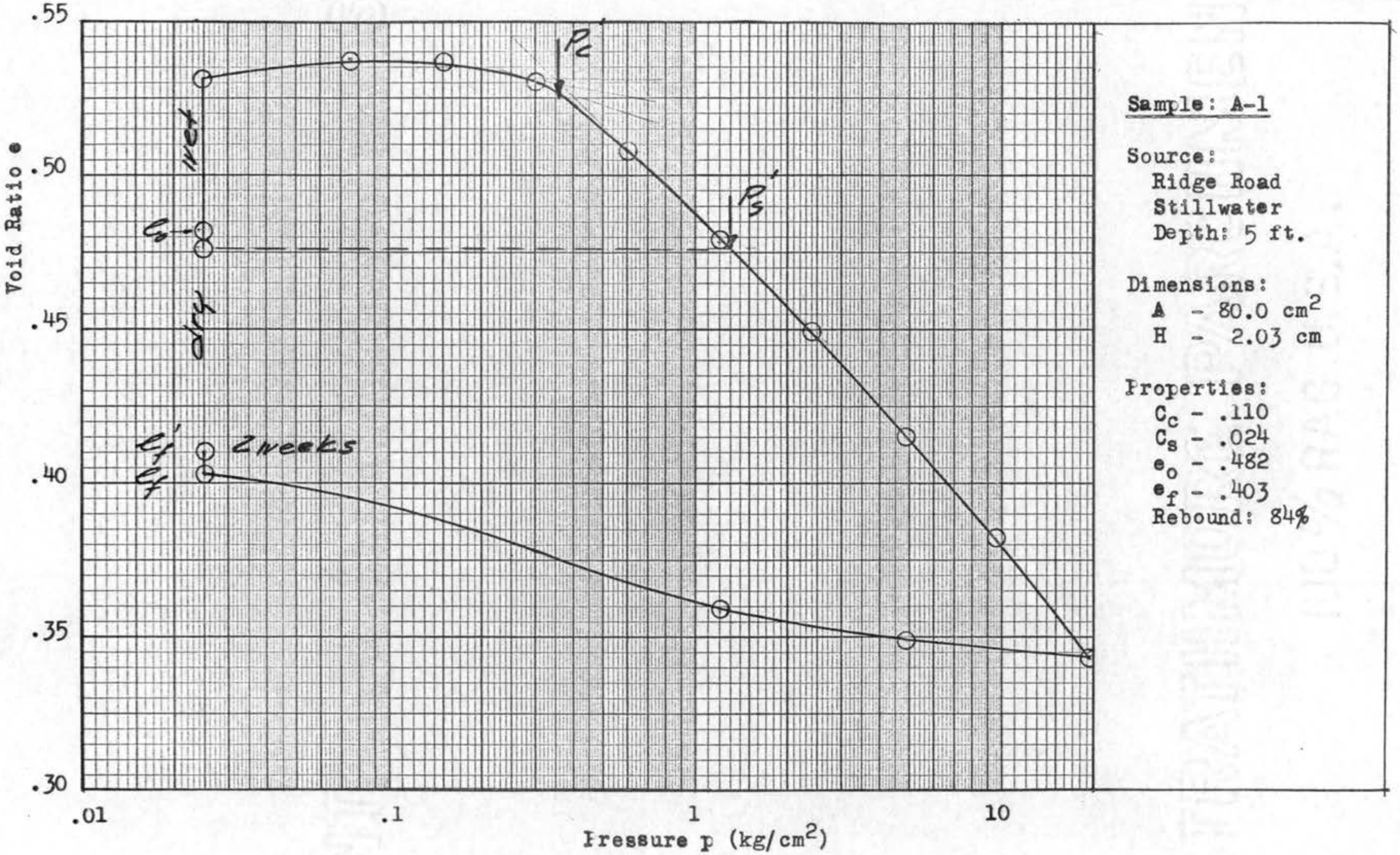


Figure 7. - Consolidation Test (e-log p Curve).

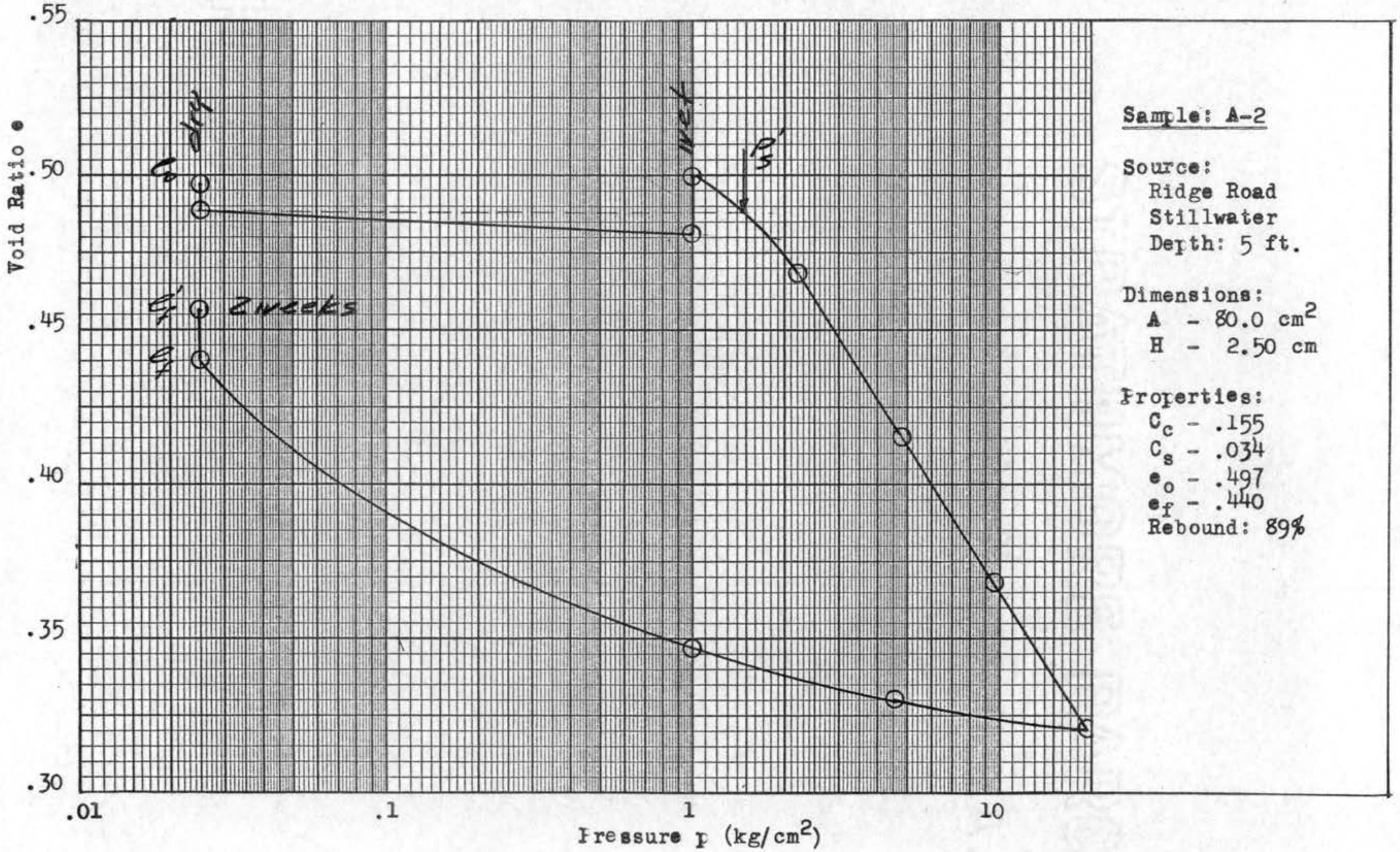


Figure 8. - Consolidation Test (e-log p Curve).

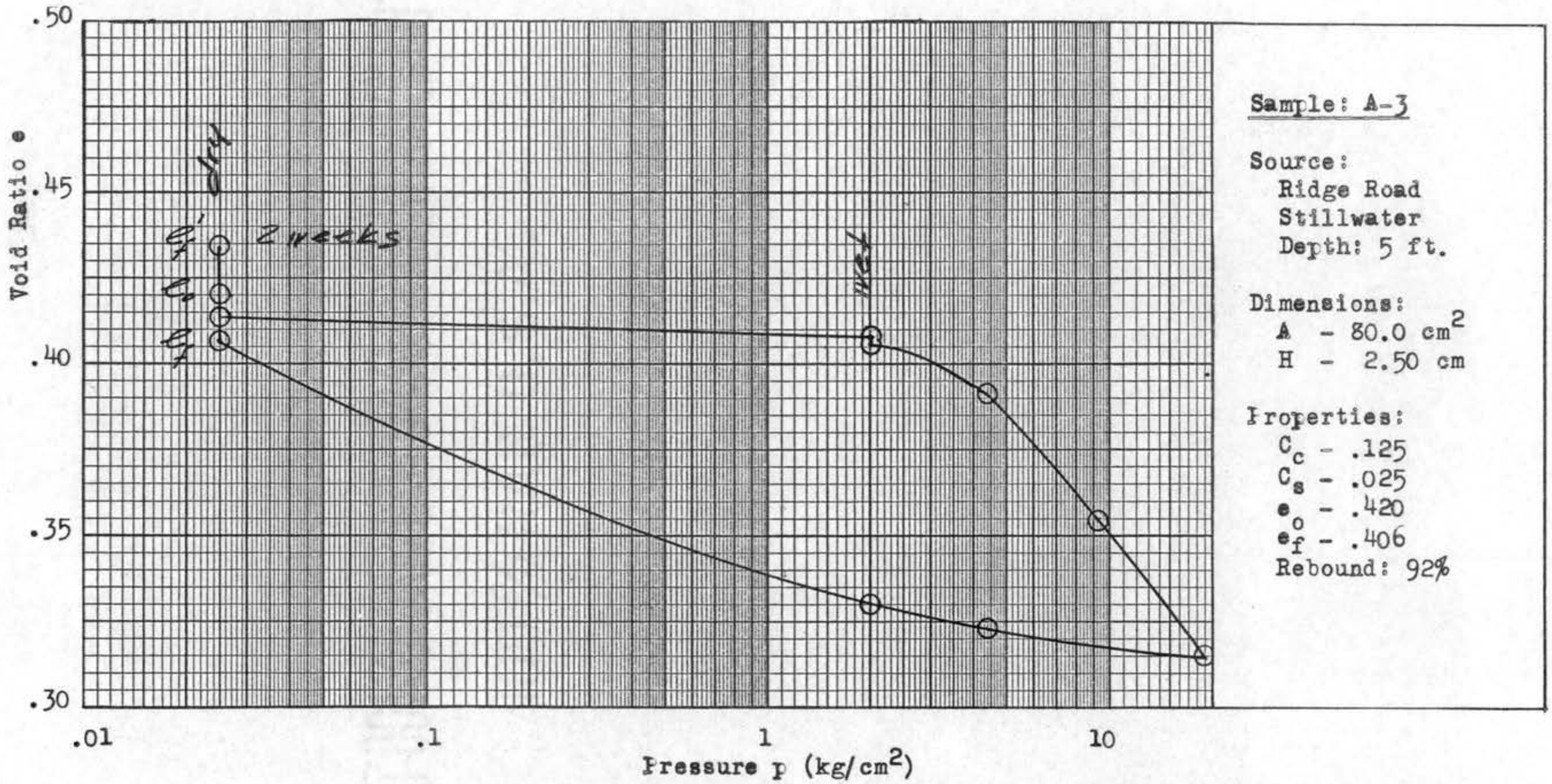


Figure 9. - Consolidation Test (e-log p Curve).

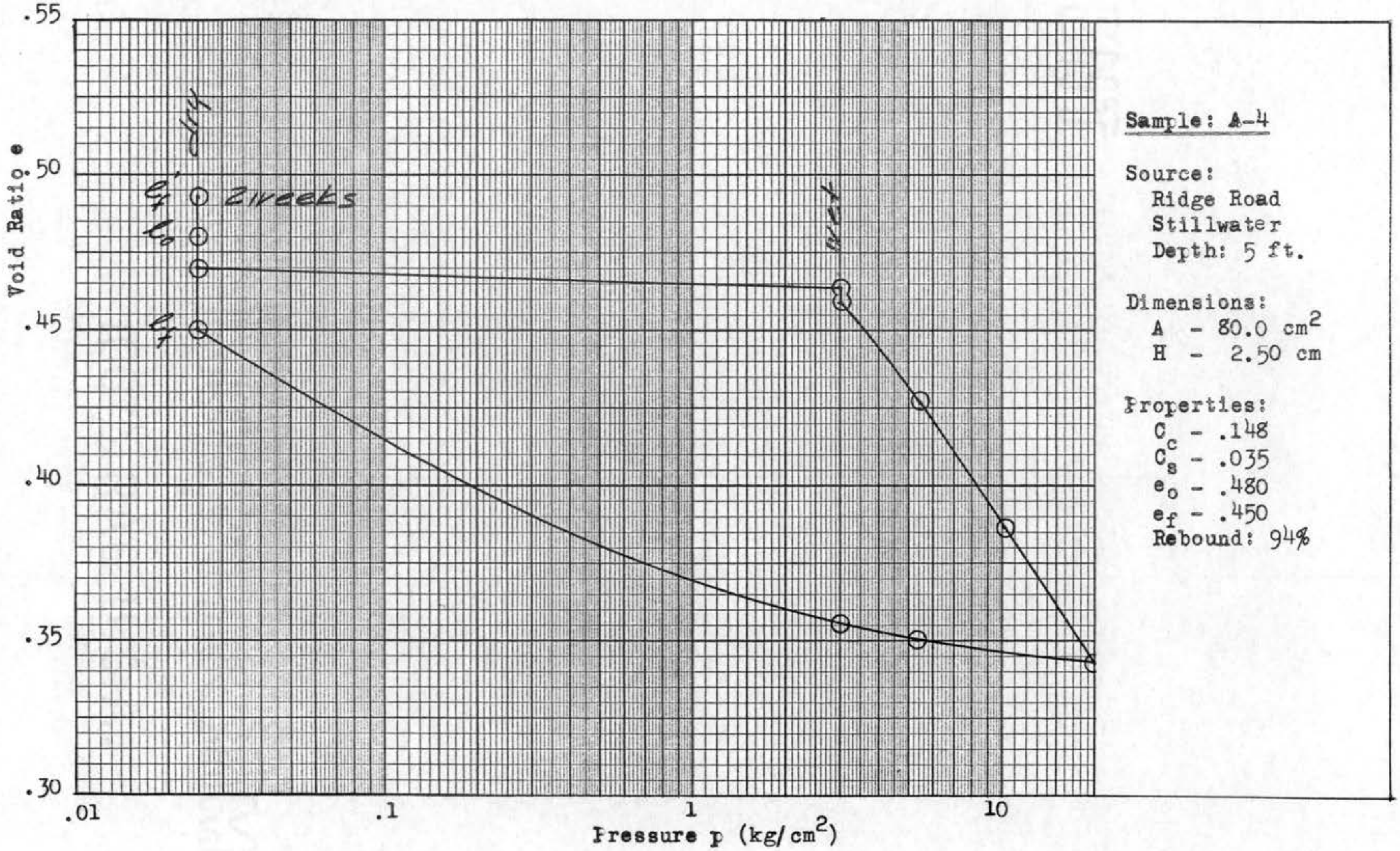


Figure 10. - Consolidation Test (e-log p Curve).

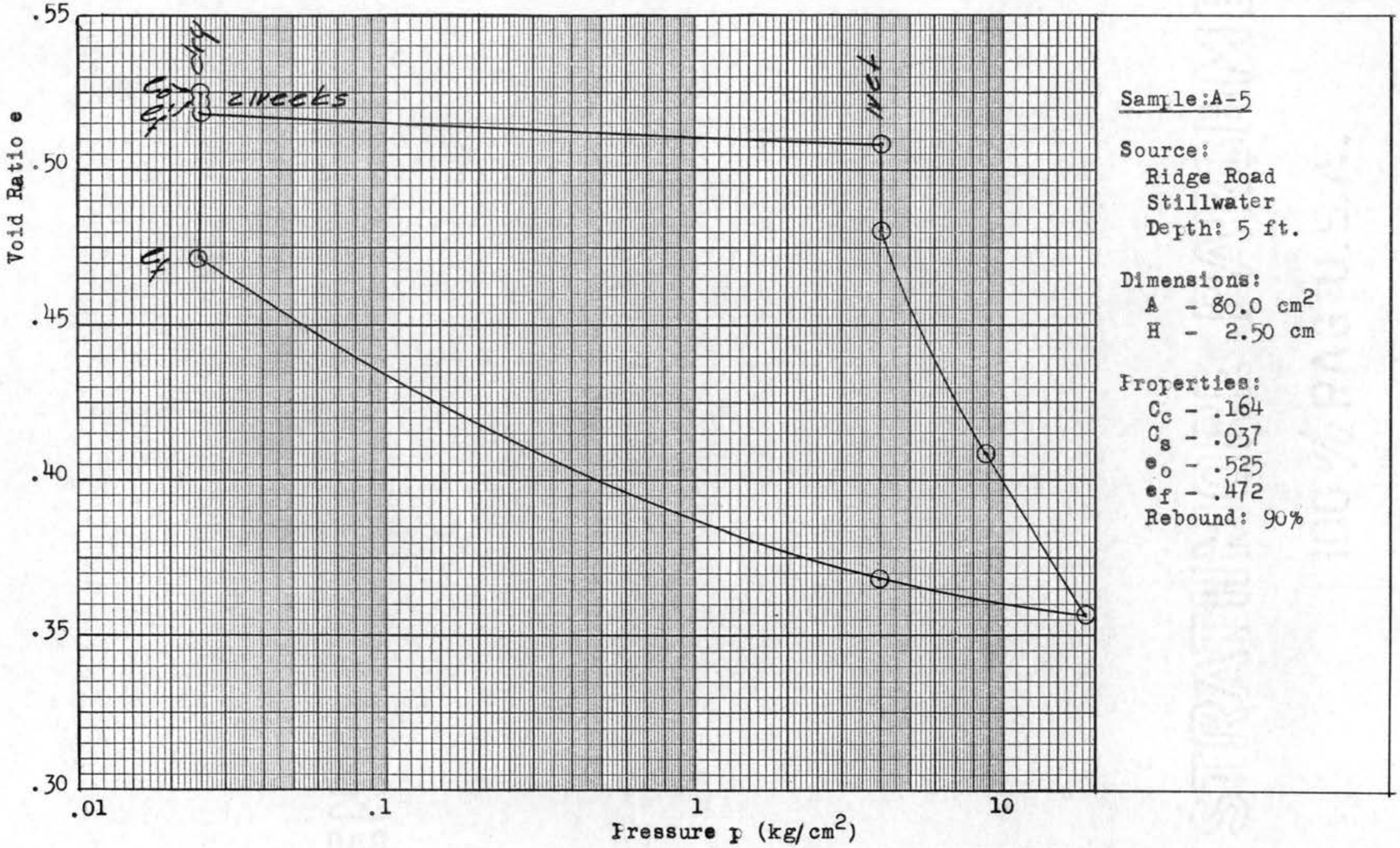


Figure 11. - Consolidation Test (e-log p Curve).

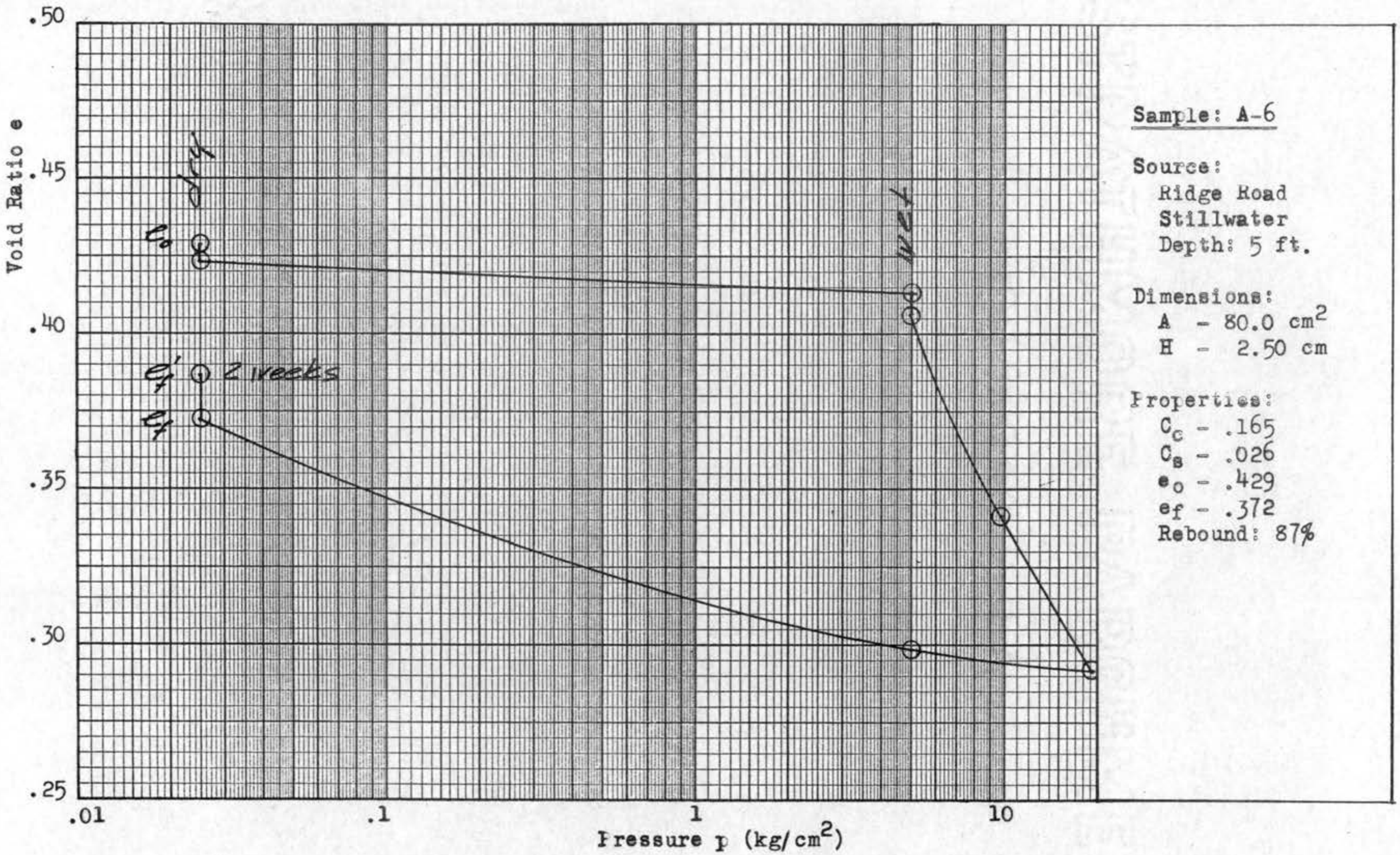


Figure 12. - Consolidation Test (e-log p Curve).

Four samples were prepared by the sharpened ring method and allowed to dry out under a drying oven, at a temperature of about 80°C. When thoroughly dry, they had shrunk sufficiently to produce cracks of approximately 1 mm width between soil and rings. Such a space, when left open, will lead to slaking of soil when water is added, as was shown in series A, and will, moreover, lead to unrealistic results, as such cracks in nature are usually filled by silt, thus providing lateral confinement. Rings of the exact diameter of the dry samples not being available, rings of the same size as the cutting ring were chosen, and the shrinkage voids filled with wax. To protect the soil from the soft wax and its certainly undesirable influence on permeability and possibly other properties of the soil, it was first covered with aluminum foil. A layer of wax was then applied around the perimeter of the sample, whereupon the latter was inserted into a slightly warm ring. The temperature of the consolidation ring must be high enough to allow easy penetration, yet not so high as to permit the wax to melt and flow out of the crack. After the wax had hardened, the circular portions of foil covering the top and the bottom of the sample were removed, and the sample placed into the consolidation machine. Pressures were applied as follows:

Sample D-1	-	1.2 tons/sq.ft.	
D-2	-	2.4	"
D-3	-	3.6	"
D-4	-	4.8	"

When water was added, all samples failed to swell, even though no soil leaked out. As would be expected, sample D-1 consolidated by the least amount of all, and D-4 by the greatest. Sample D-3, however, showed a value falling between those of samples D-1 and D-2, which might possibly be explained by an unsuccessful attempt to fill the shrinkage crack of D-2. Since D-2, however, seems to follow the overall pattern more closely than D-3, it appears more likely that some factor contributing to the heterogeneity of the material, such rock fragments, is responsible for the discrepency.

The samples were then dried out in the machine under load and showed further shrinkage, which ceased after about two weeks. When flooded, swelling was observed for all samples, but by an amount substantially less than the shrinkage. The samples were next subjected to several alternate drying and flooding cycles, each period lasting until no further shrinkage or swelling could be observed. For all samples, the amount of swelling continued to fall short of the shrinkage in each cycle, but the differential grew successively smaller, until swelling and shrinking approached equality after the fourth cycle.

Curves for swelling and shrinkage were plotted as unit deformation versus cycles for each sample (figs. 13 to 16). For purposes of these graphs, positive swelling values indicate volume increase, and positive shrinkage values indicate volume decrease. This sign convention is consistent with the meanings commonly given the words swelling and shrinkage. The assymptotes of these curves are straight, horizontal lines. Their ordinates indicate the amount of unit heaving h/H, of which laterally confined clay of this type is capable against the indicated applied pressures. The values determined are:

Pressure p	Unit heaving h/H
1.2 tons/sq.ft.	0.0084 cm/cm - 0.101 in/ft
2.4 "	0.0060 " - 0.072 "
3.6 "	0.0055 " - 0.066 "
4.8 "	0.0053 " - 0.063 "

When these values are plotted as pressure versus unit heaving, it is interesting to note that the strain values appear to approach a constant (approximately 0.0055 cm/cm) at pressures exceeding five tons/sq.ft. That is to say that this desiccated clay at higher pressures will shrink or swell by the constant amount of about 1/16 inch per foot of thickness, regardless of what the exact amount of applied pressure is. Since it seems unreasonable to believe that this statement holds true in an unlimited range, the writer feels that similar tests using pressures upward of 5 tons/sq.ft. would prove very interesting and are necessary before the final word on this subject can be said.

The fact that this clay initially failed to swell and changed its properties during repeated drying cycles, indicates that originally it may not have been desiccated. The sample having been taken from near the water table, this assumption is very likely correct.

Series E. The four samples of this series were obtained from the same location in Oklahoma City as those for series D and were prepared in an identical manner. Instead of ovendrying the specimens, however, they were dried in the machine, at room temperature, and under the following pressures:

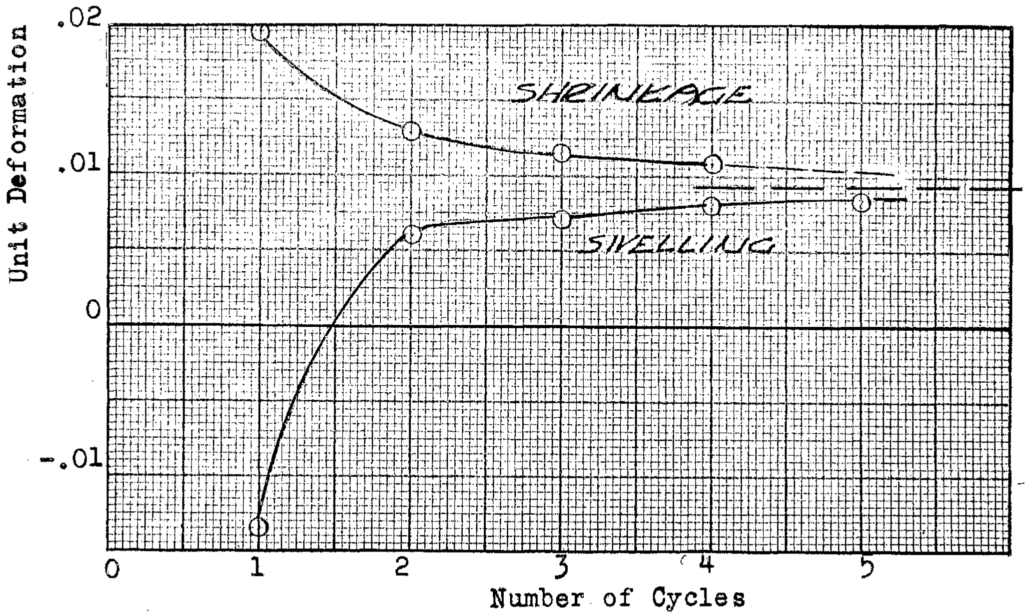


Figure 13. - Cyclic Swelling.
Sample D-1 Pressure p - 1.2 kg/cm^2

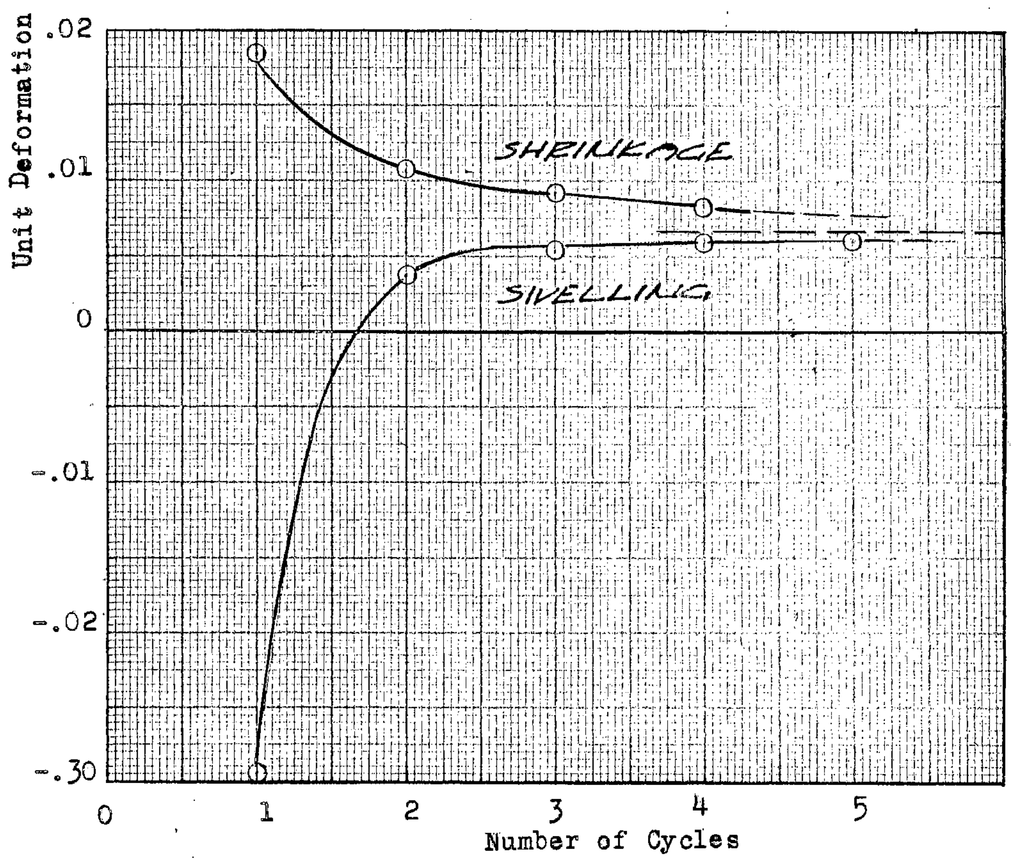


Figure 14. - Cyclic Swelling.
Sample D-2 Pressure p - 2.4 kg/cm^2

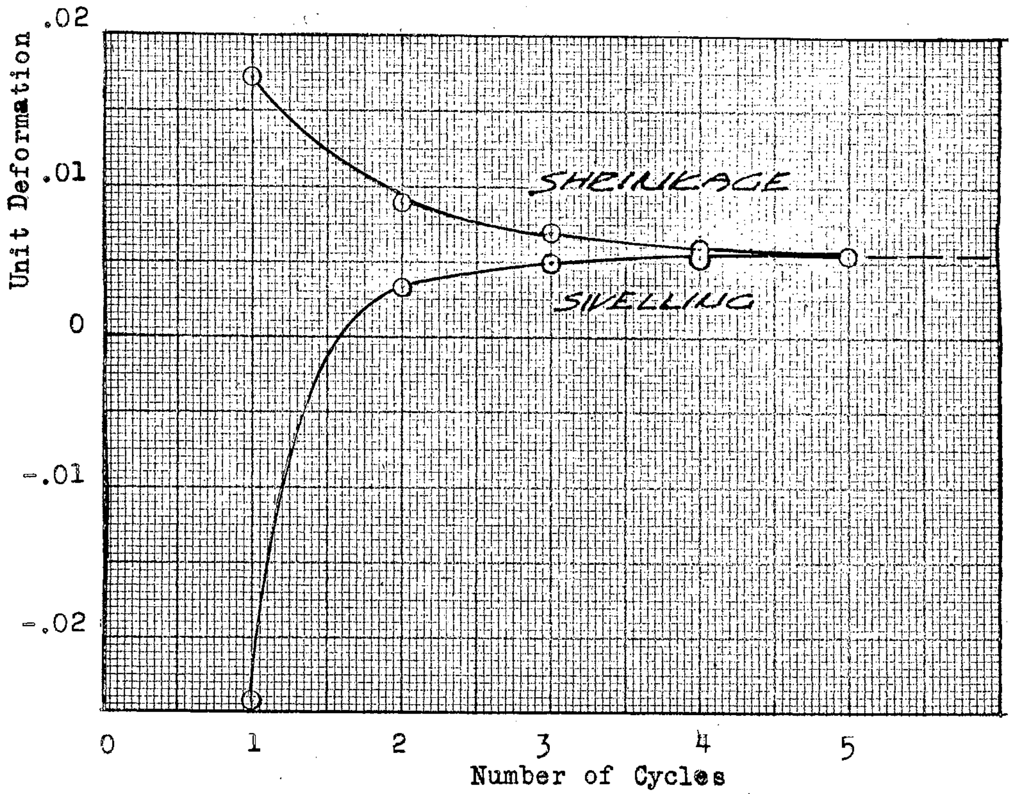


Figure 15. - Cyclic Swelling.
Sample D-3 Pressure p - 3.6 kg/cm^2

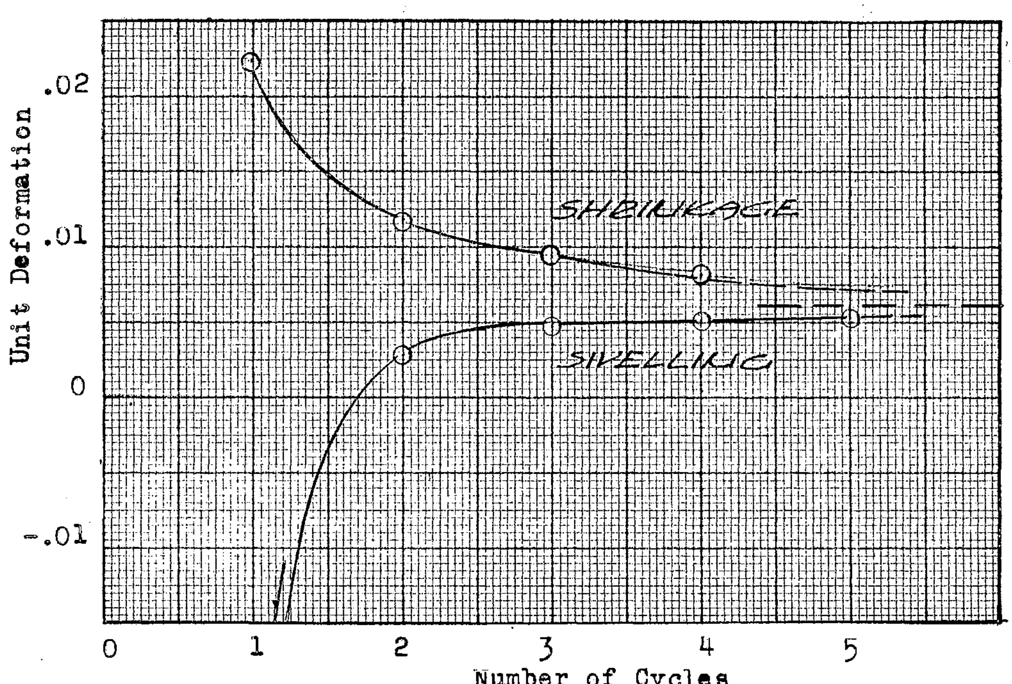


Figure 16. - Cyclic Swelling.
Sample D-4 Pressure p - 4.8 kg/cm^2

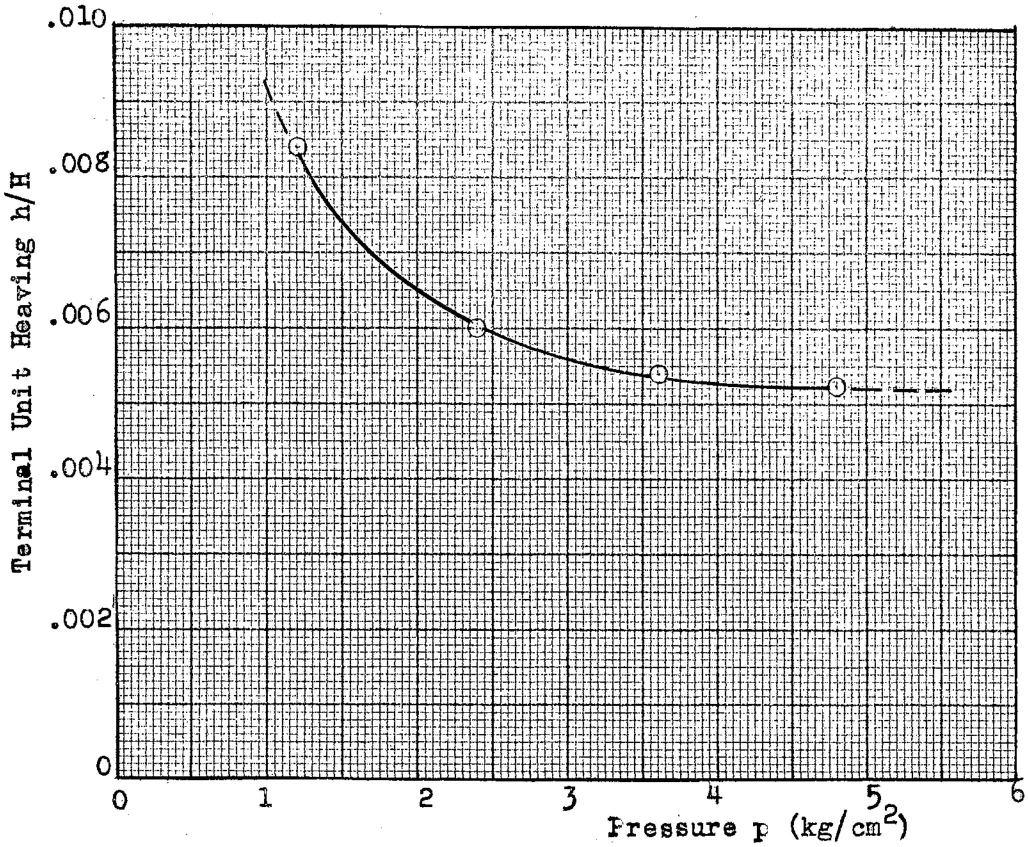


Figure 17.

Series D - Pressure-Heaving Curve.

Sample E-1	-	1.2 tons/sq.ft.	
E-2	-	2.4	"
E-3	-	3.6	"
E-4	-	4.8	"

When shrinkage had ceased, the samples were taken out of the machines in order to fill the shrinkage cracks around the perimeters. It was noted, however, that in contrast to the oven-dried specimens the cracks that had formed were so narrow as to make filling them seem impossible. They were nevertheless returned to the machine and flooded under load. As expected, heavy slaking of clay was observed during the first as well as two succeeding cycles. Hence, the data obtained were rendered almost valueless and the test was abandoned. Two observations, however, are significant.

Firstly, the shrinkage values obtained, not effected by slaking, bear a close resemblance to their counterparts of series D, thus substantiating the results of those tests. The significant data obtained from the two series are listed in table 1.

Secondly, the difference in the nature or amount of shrinkage between series D and E poses a question, which might be answered in one or both of two ways. Visual inspection revealed cracks in series D several times as wide as those in series E, while no information is available on the vertical shrinkage values of series D.

It is possible that temperature has an influence of no small significance on the amount of shrinkage, greater changes in volume being possible at higher temperatures. This would indicate that at any given temperature only a certain corresponding amount of double layer water

TABLE I

UNIT DEFORMATION DUE TO CYCLIC DRYING

Pressure (kg/cm^2)	Cycle No.	Series D Swelling (cm/cm)	Series D Shrinking (cm/cm)	Series E Shrinking (cm/cm)
	0			.0150
	1	-.0135	.0190	.0170
1.2	2	.0059	.0129	.0130
	3	.0070	.0115	
	4	.0081	.0109	
	5	.0084		
	0			.0163
	1	-.0295	.0182	.0176
2.4	2	.0038	.0106	.0119
	3	.0052	.0091	
	4	.0059	.0084	
	5	.0060		
	0			.0170
	1	-.0243	.0172	.0161
3.6	2	.0034	.0089	.0097
	3	.0050	.0070	
	4	.0052	.0054	
	5	.0054		
	0			.0170
	1	-.0539	.0222	.0174
4.8	2	.0029	.0117	.0097
	3	.0049	.0096	
	4	.0052	.0082	
	5	.0053		

can be driven off, so that the shrinkage potential rises with temperature.

But the applied pressure was also different in the two cases. It might be concluded that an externally applied pressure in one direction tends to aid shrinkage in the same direction by forcing out double layer water from beneath individual soil particles when these are forced closer together. No significant difference in the width of cracks between the various samples of series E was discovered by visual inspection of the samples, however, so that this theory can not be substantiated.

It is possible, of course, that both factors entered into the behavior of the clay, with the first one very likely having had greater influence.

<u>Series F.</u> The four samples of series D were used for this series after completion of the former; the individual specimens correspond in their numbers of designation.

After the last drying and swelling cycle of series D, all pressure was removed from the samples except the minute amount of 0.025 tons/sq.ft., and the clay was permitted to swell under this reduced load for one day. Drying was then initiated, and when shrinkage had ceased after two weeks the original pressures of 1.2, 2.4, 3.6, and 4.8 tons/sq.ft. were reapplied. After another day the samples were flooded, which proved to cause swelling of F-1, no volume change of F-2, and consolidation of the remaining two samples. At intervals of one day, i.e. 1440 minutes, the loads were increased up to the maximum pressure of 19.2 tons/sq.ft. and then gradually reduced to zero. The void ratio - pressure curves were plotted to the customary semi-logarithmic scale and included here as figures 18 to 21.

Since flooding of sample F-2 under 2.4 tons/sq.ft. led to neither swelling or consolidation, this value suggests itself as the preconsolidation load due to desiccation. A horizontal line through the dry void ratio of curve F-1 under 1.2 kg/cm^2 cuts the e - log p curve of that sample at 2.2 tons/sq.ft., and similar values would be arrived at if corresponding lines were drawn to intersect with the extensions of the upper portions of the e - log p curves of samples F-3 and F-4. It is therefore concluded that the preconsolidation pressure due to desiccation of this clay amounts to some value slightly higher than 2 tons/sq.ft. This figure is higher than that determined in series A, but much smaller than the preconsolidation pressure of 3.5 to 4.5 tons/sq.ft. which has repeatedly been found for typical desiccated clays of this region. Since clays in nature have dried out many more times than the number of drying cycles to which these specimens were subjected in the laboratory, it appears that additional drying cycles will further increase the preconsolidation load due to desiccation.

What causes this increase? Quite possibly each successive drying cycle urges the soil particles into denser and stronger structure. H. B. Seed (9) conducted repeated load tests on remolded clays and found increased resistance to deformation with an increased number of loading cycles. The causes to which he attributes these effects are increase in density, thixotropic action, or both. The problem at hand is quite different, but obviously the causes for increase in strength due to repeated loading and due to repeated drying are related.

Another question that arises is this: Why will a clay sample that has achieved equal and constant volume change due to drying and wetting after a sufficient number of cycles under a certain load show a different deformation due to flooding under the same load after the pressure has been relieved and reapplied? The test results show that this is the case, and that the new amount of swelling becomes zero at the preconsolidation pressure due to desiccation (which is thus determined) and increasingly more negative (consolidation) under even heavier pressures. It is also shown that the void ratio during repeated drying and wetting under a given pressure is generally lower than that obtained after this pressure has been relieved and then reapplied after swelling and shrinking have taken place under zero load.

The answer is probably, that when the load is removed, the soil particles will partially, but not entirely, relinquish their new dense structure,and repeated cycles will again be necessary to give the clay its former high strength and density.It is surmised that this process, too,is limited in that a constant condition would probably be approached after a sufficient number of cycles.

The practical meaning of these phenomena is that if the clay under a foundation were able to dry and swell periodically,as is not normally the case, not only would ways of combatting heaving become more difficult, but the heaving action itself would be more serious, because the volume change of the clay would be greater, as well as possible against heavier loads.

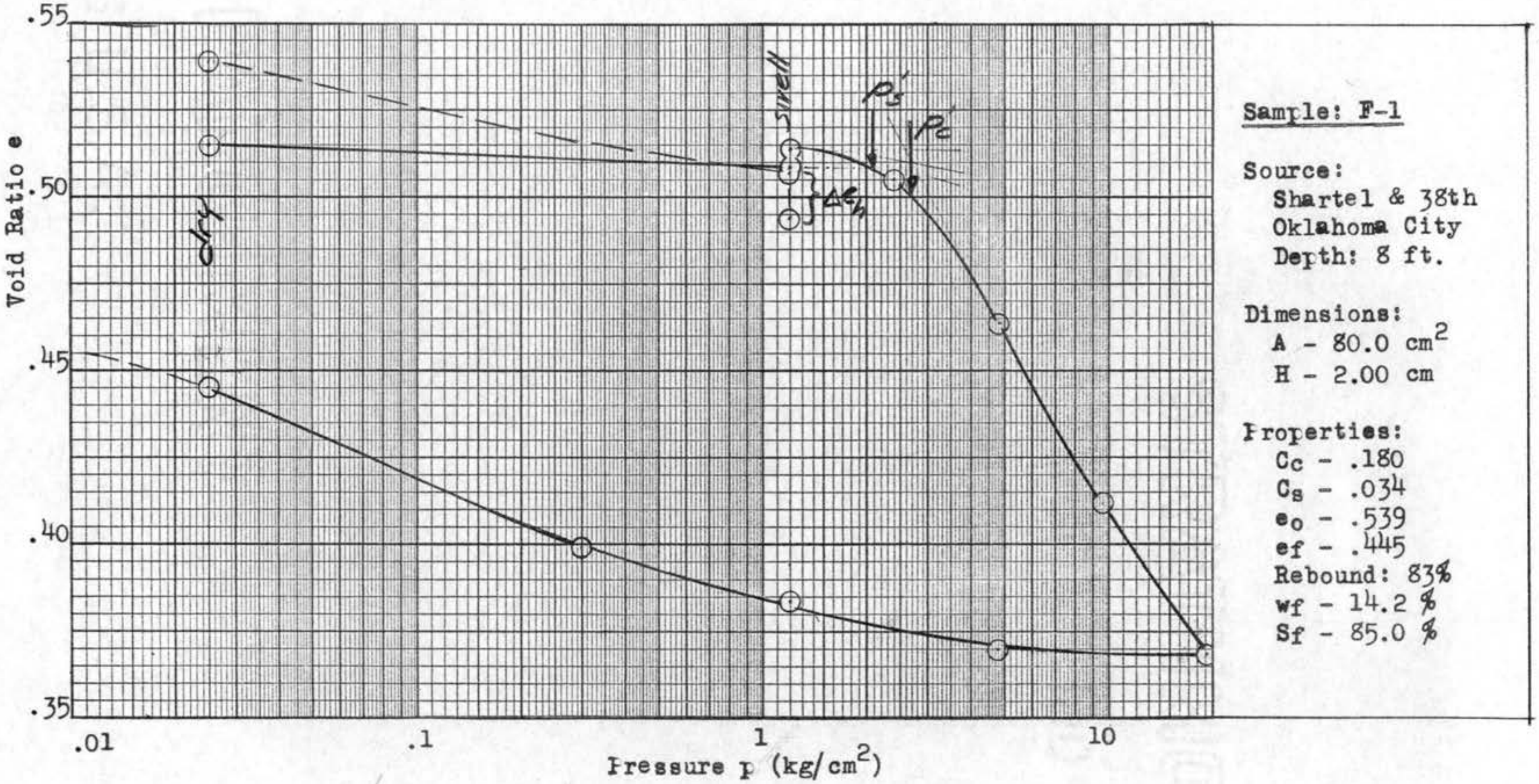


Figure 18.

Consolidation Test (e - log p Curve).

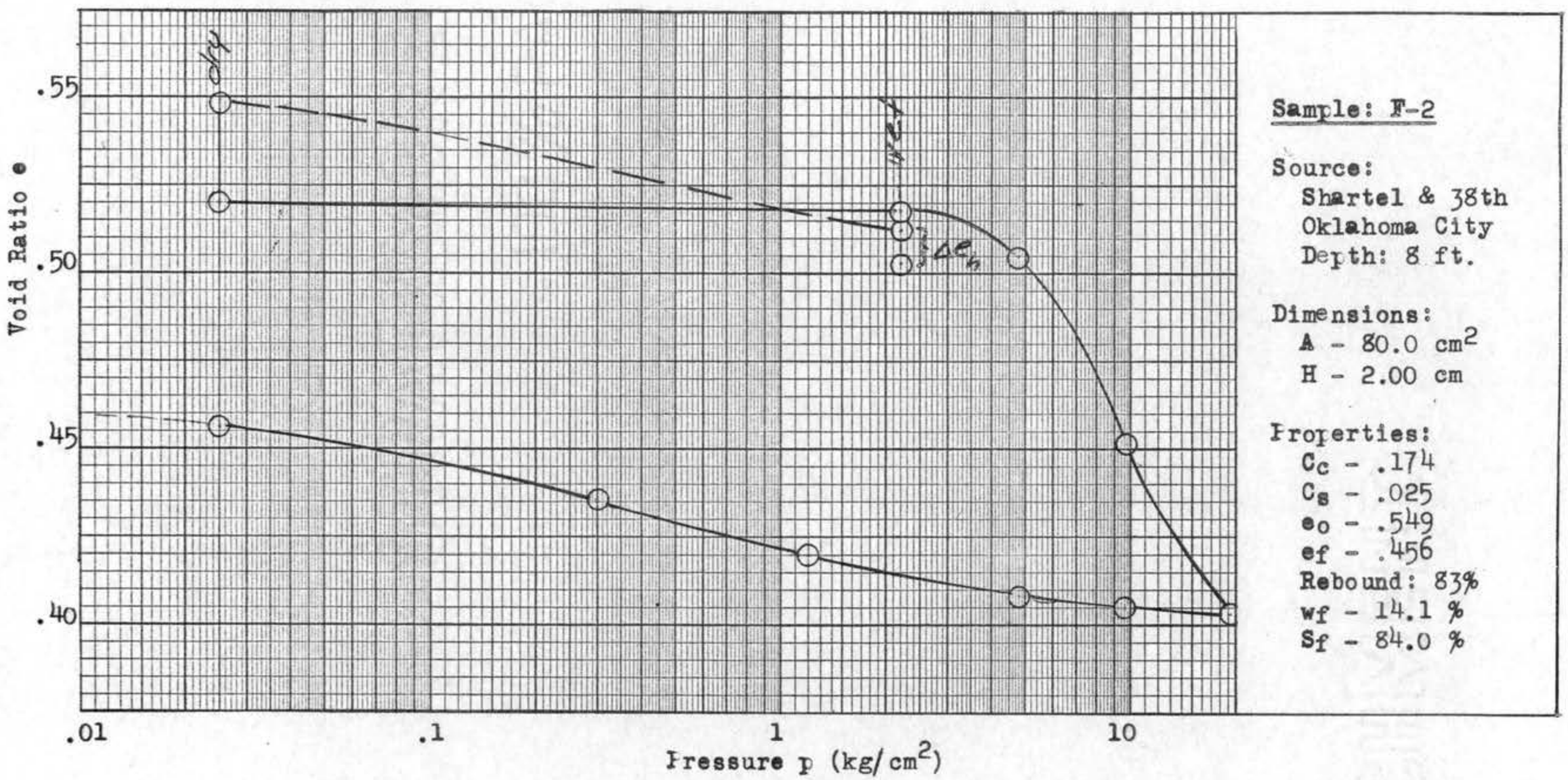


Figure 19.

Consolidation Test (e - log p Curve).

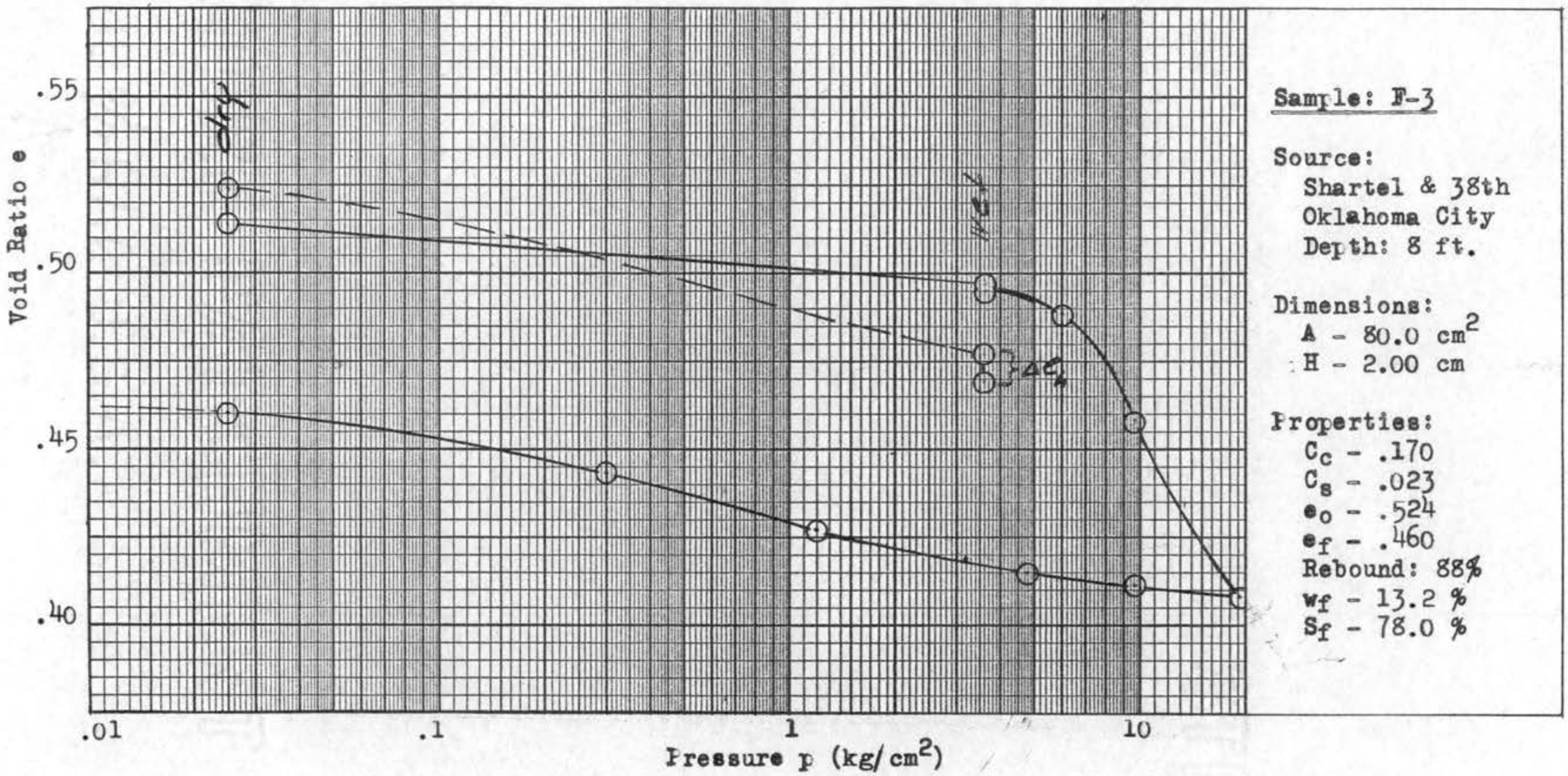


Figure 20.

Consolidation Test (e - log p Curve).

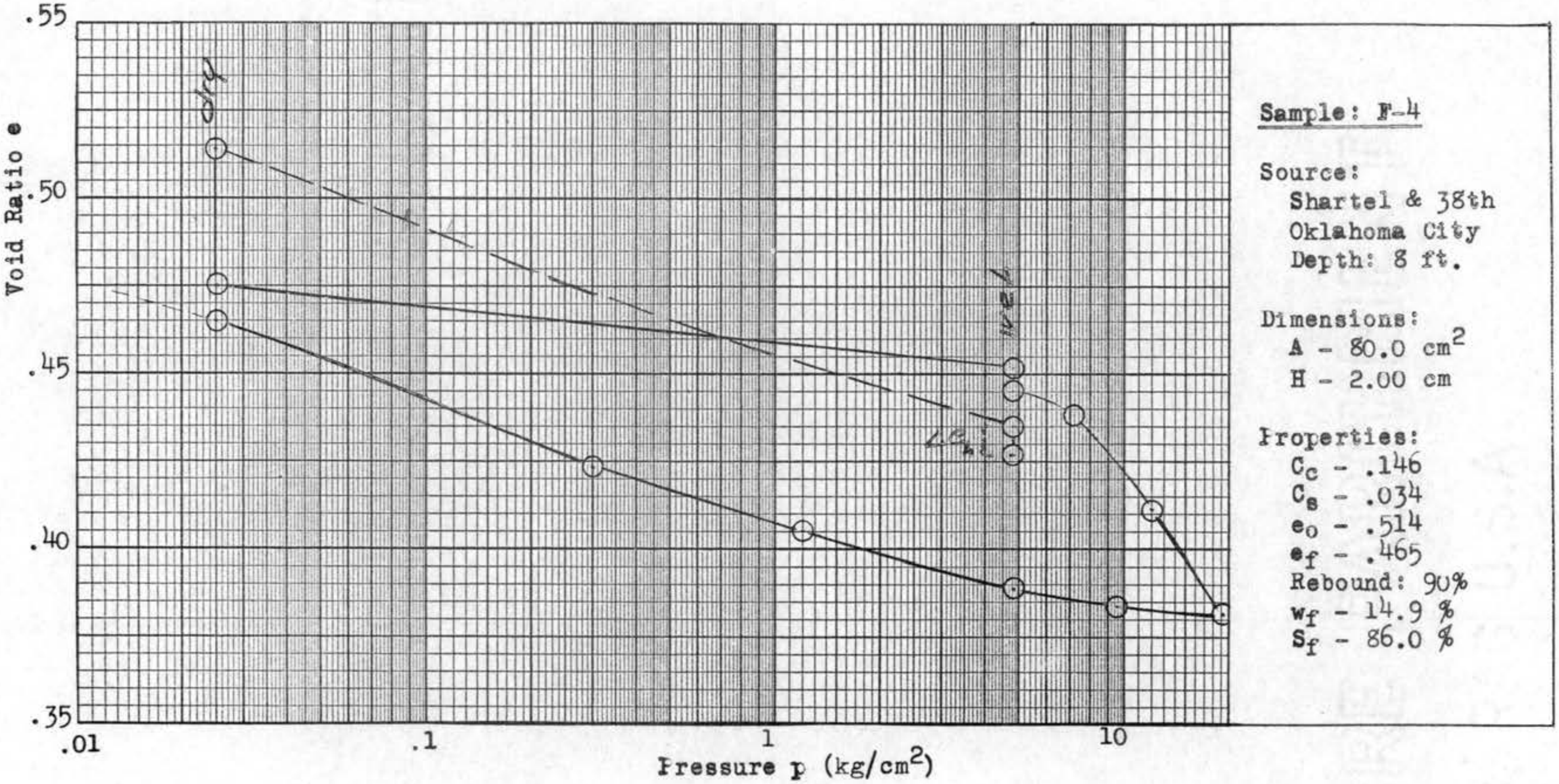


Figure 21.

Consolidation Test (e - log p Curve).

Intrusion Tests

It was now attempted to determine some values that relate to the ability of Permian clay to intrude into cohesionless materials. The various variables considered for these tests were time, pressure, type of gravel, and availability of moisture. The author failed, however, in finding a practical way of testing the influence of the latter, the high impermeability of the clay making the control of water supply exceedingly difficult. Several series of tests were conducted examining the influence of the remaining three factors and are described below.

Series G. These were extended tests designed to measure the intrusion of clay into cohesionless material with respect to time. The four clay samples involved were those previously used for the repeated drying tests of series E. For simplicity, steel balls rather than gravel were placed on the saturated samples in two layers. The diameter of the balls of samples G-1 and G-2 was 1.10 cm. and that of the balls of G-3 and G-4 was 1.58 cm. Loads were applied to the steel balls through porous stones resting on them, as shown in figure 22, with average pressures being exerted on the clay samples as shown:

Sample G-1	-	1.2	tons/sq.ft.
G-2	-	2.4	"
G-3	-	3.6	"
G-4	-	4.8	"

Each apparatus was immersed in water, and the deflections measured until all movement had ceased. Deflections were plotted versus time for each sample, and the graphs included here as figures 23 to 26.

It is seen that the initial deformations, measured after a few seconds, are responsible for the major portion of the total deformations, with the succeeding deformations amounting generally to less than 1 mm, a very slight amount when compared to heaving of several inches, which is common. Even the total intrusion appears to be of little help in compensating for swelling, and the primary, initial set must be disregarded, since it occurs much faster than swelling and would probably take place during construction or whenever water is made available to the top layer of clay. The secondary part, however, seems to approach roughly the shape of a typical swelling curve. If secondary intrusion, therefore, can be caused to be large enough to be comparable to heaving deformations, an effective method of stabilizing a foundation is possible. Possibly this can be

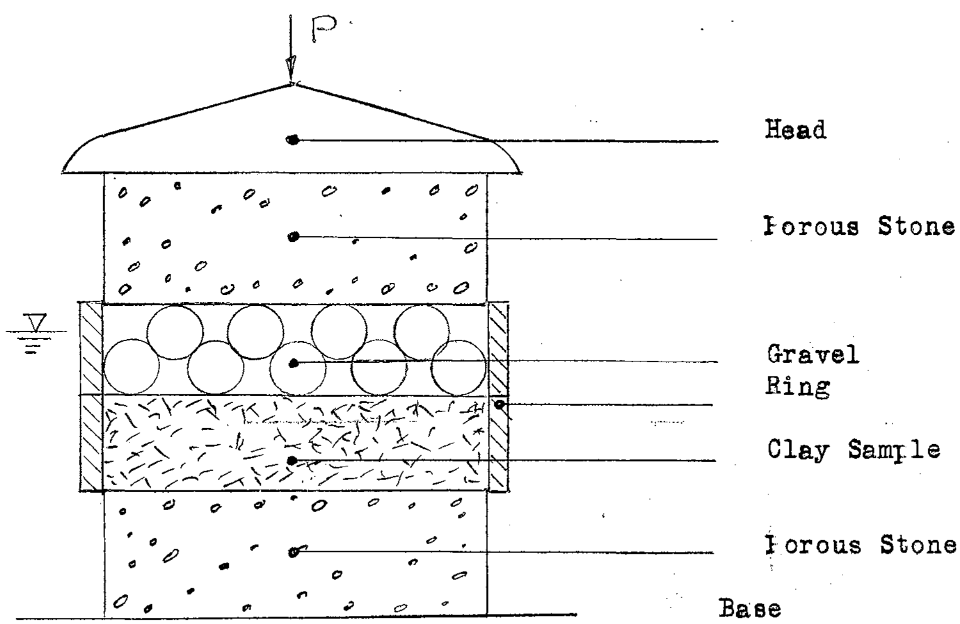


Figure 22.

Intrusion Test Apparatus.

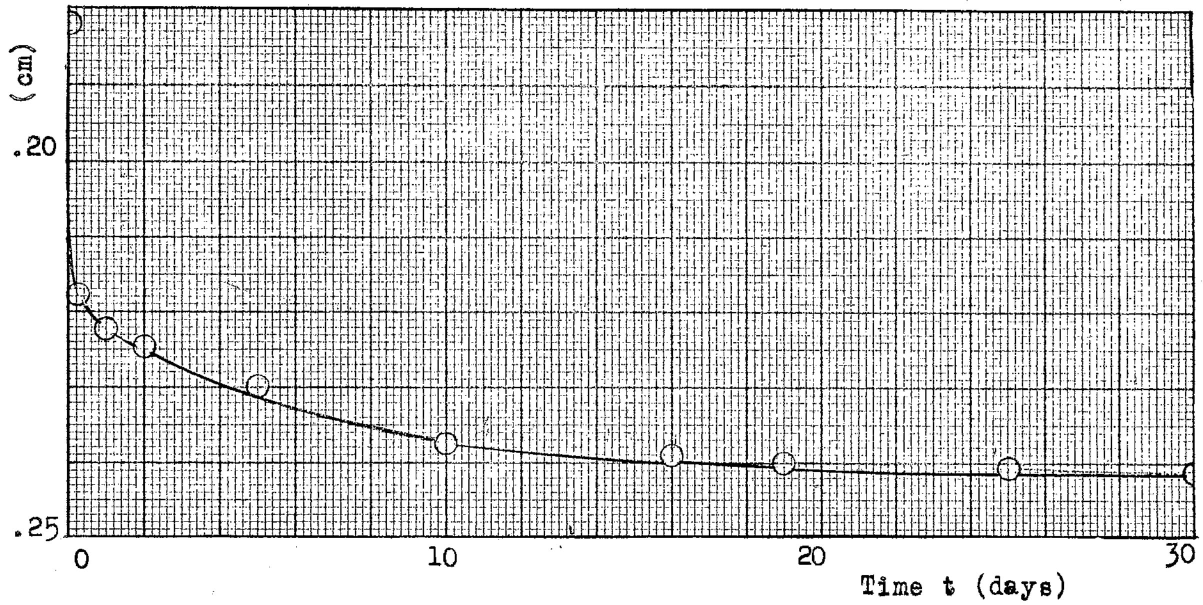


Figure 23. - Time-Intrusion Curve.

Sample G-1 - Balls: 1.10 cm dia.; Pressure: 1.2 kg/cm^2

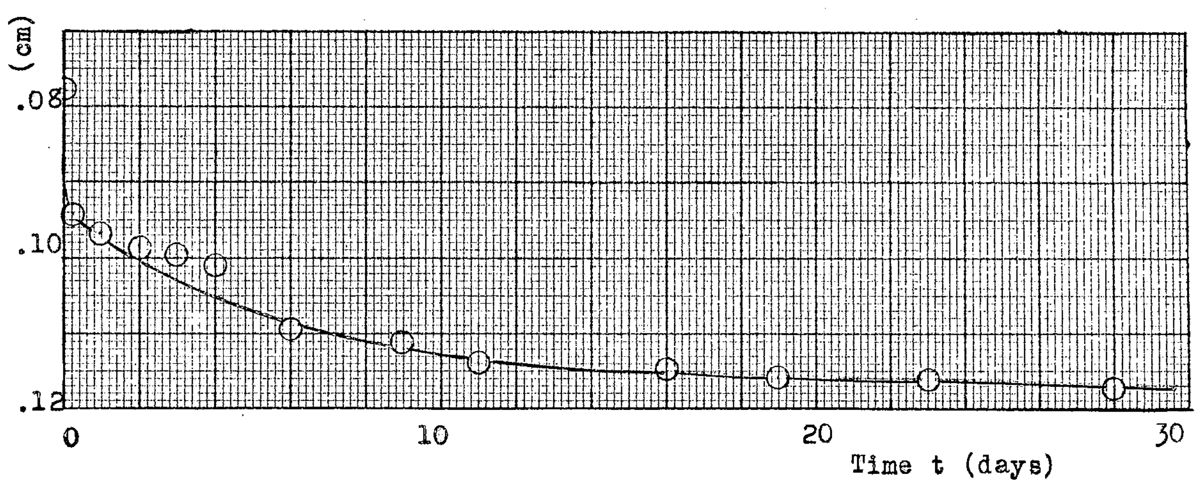


Figure 24. - Time-Intrusion Curve.

Sample G-2 - Balls: 1.10 cm dia.; Pressure p: 2.4 kg/cm^2

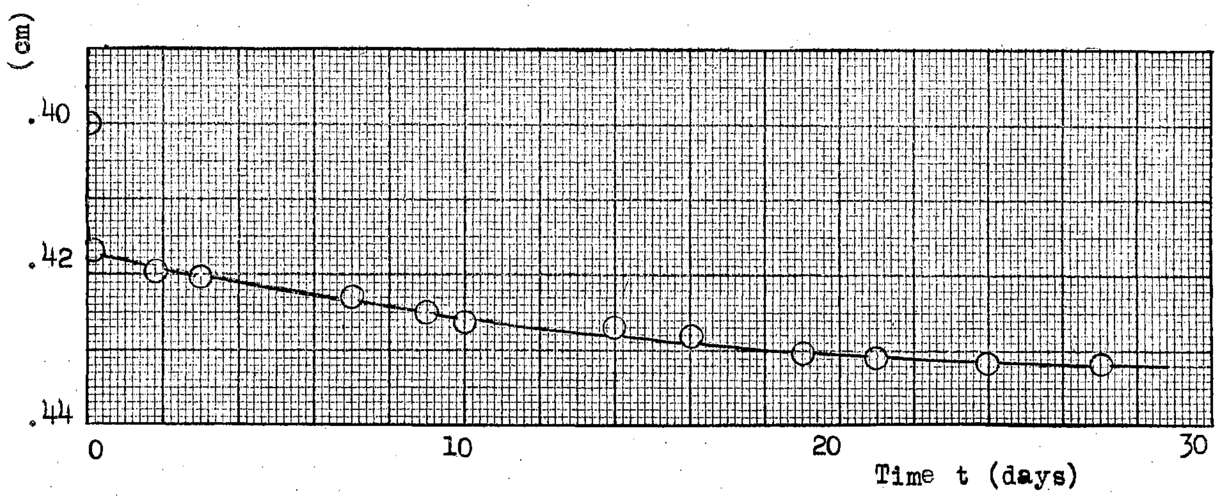


Figure 25. - Time-Intrusion Curve.

Sample G-3 - Balls: 1.58 cm dia.; Pressure p: 3.6 kg/cm^2

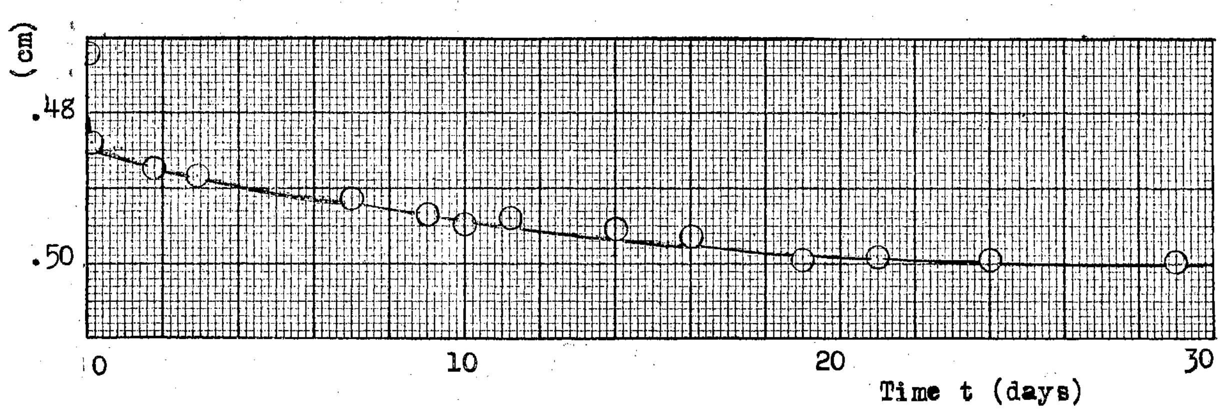


Figure 26. - Time-Intrusion Curve.

Sample G-4 - Balls: 1.58 cm dia.; Pressure p: 4.8 kg/cm^2

achieved by selecting much larger gravel for similar tests, which would more reasonably be performed in the field rather than in the laboratory.

<u>Series H.</u> These tests were designed to obtain some information on the relationship between pressure and initial intrusion for several types and sizes of gravel. The four samples previously used in series D and F were used. While their histories were not identical, their maximum preconsolidation loads were, and all had been allowed to swell under zero load for several days. Various types of gravel, as described below, having the void ratios indicated, were placed on the samples in thicknesses of about 4 cm.

Sample	Void Ratio	Type of Gravel
H-1	0.78	Steel balls (1.10 cm diameter)
H-2	1.15	Steel balls (1.58 cm diameter)
H-3	1.00	Subrounded gravel
H-4	1.13	Angular gravel

Both types of gravel were of a size comparable to the steel balls, passing a 3/4 inch sieve and being retained by a 1/2 inch sieve. The test apparatus was substantially the same as that of series G, but while free water was available at the surface of the clay, immersion was not deemed necessary.

By means of a consolidation machine, pressure was applied in increments of 1.2 tons/sq.ft. in each case, at intervals of one minute. It was previously reported that after that time the largest part of the total intrusion under a given pressure, especially a high pressure, has usually been accomplished. Making the simplifying, but somewhat erroneous, assumption that the gravel was incompressible in each case, and after correcting for machine deformations, the results of these tests are reported here in the form of stress - strain curves. During test H-3 it was noted that at a pressure of 14.4 tons/sq.ft. the subrounded gravel began to slip, as evidenced by sudden, erratic increases in deformation, and the test was therefore discontinued at this point. The four pressure - intrusion curves are shown in figures 27 to 30.

These graphs allow a number of observations:

1. The curves are steepest in the lower pressure range, i.e. the intrusion for a given pressure differential is largest for those, smaller, pressures that are dealt with in foundations work.

2. As expected, samples H-1 and H-2 show that larger sized cohesionless material results in larger intrusion values.

3. No large difference seems to exist between the behavior of different shapes of gravel. However, the smaller void ratio of sample H-3 probably more than offsets the advantage of the rounded particles over the angular gravel of sample H-4.

4. The curves of the two samples using natural gravels appear to offset under heavier pressures, indicating slippage and crushing. Had this not occurred, the curves would probably have more nearly followed the imaginary, dotted line.

5. It was possible to compare one sample, H-4, to a test conducted by Mr. Yie (12) on compacted clay of nearly the same water content using the same size and type of gravel. His pressure - intrusion curve shows a less pronounced change of curvature, but the intrusion

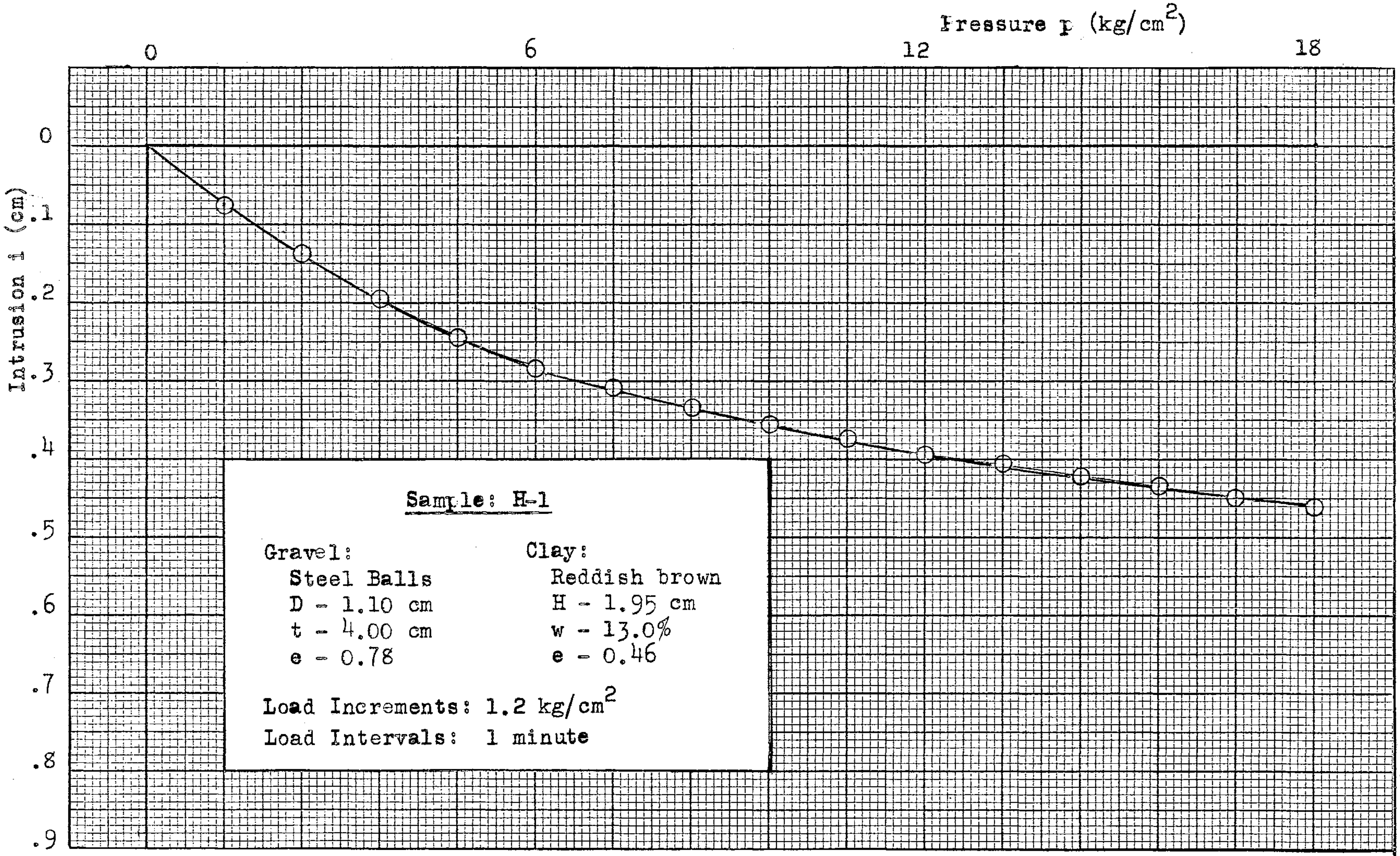


Figure 27. - Pressure - Intrusion Test (p - i Curve).

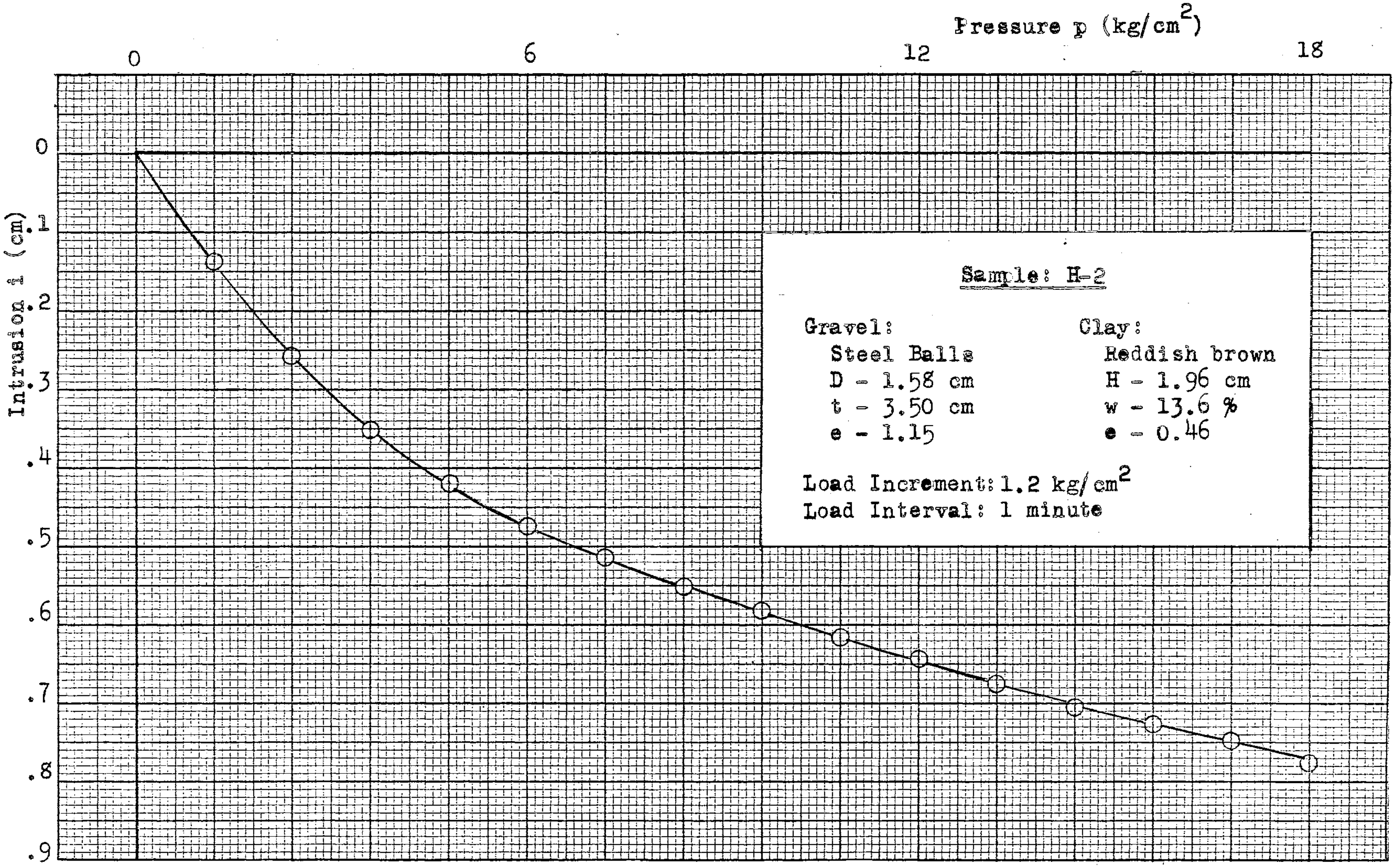


Figure 28. - Pressure - Intrusion Test (p - i Curve).

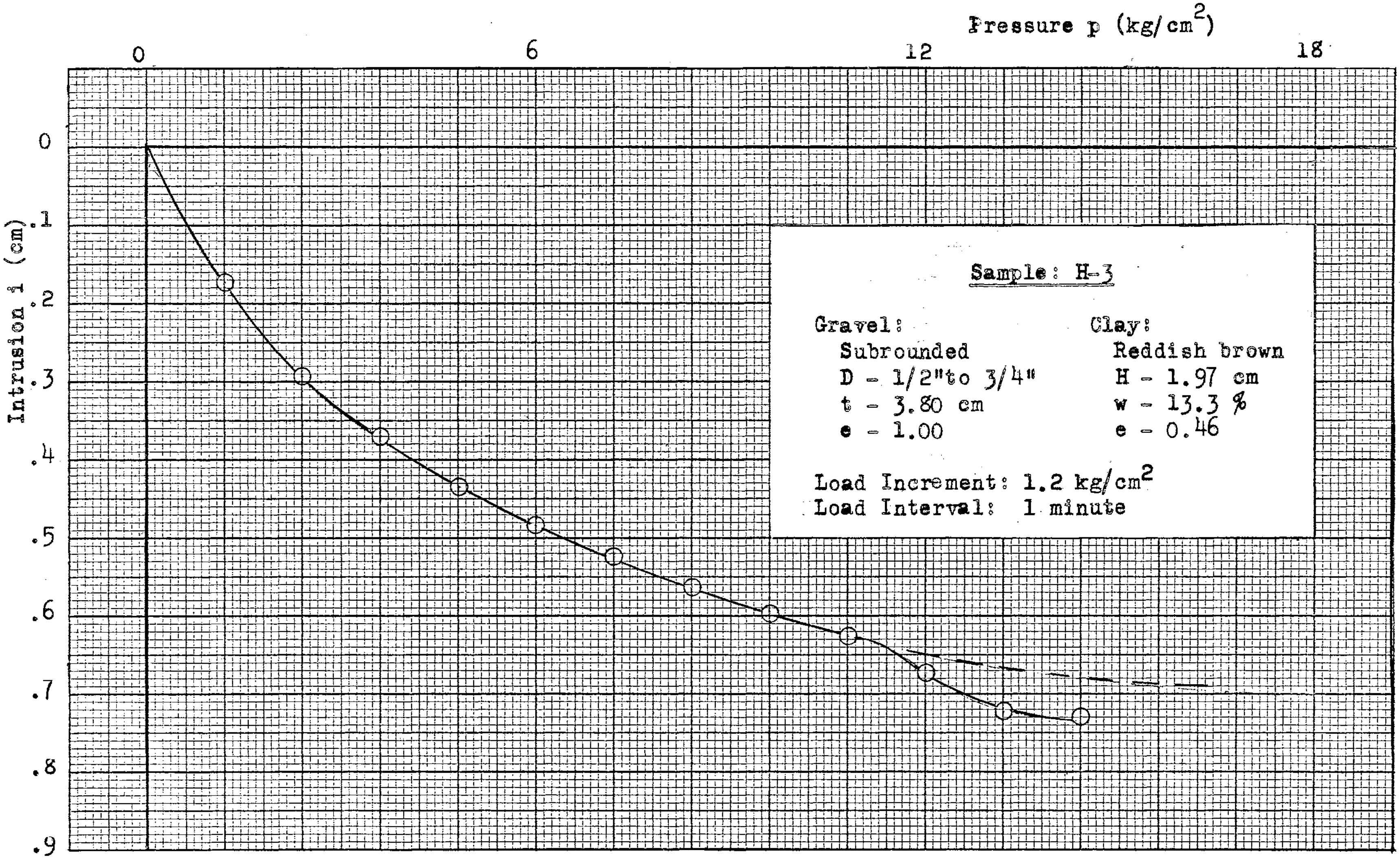


Figure 29. - Pressure - Intrusion Test (p - i Curve).

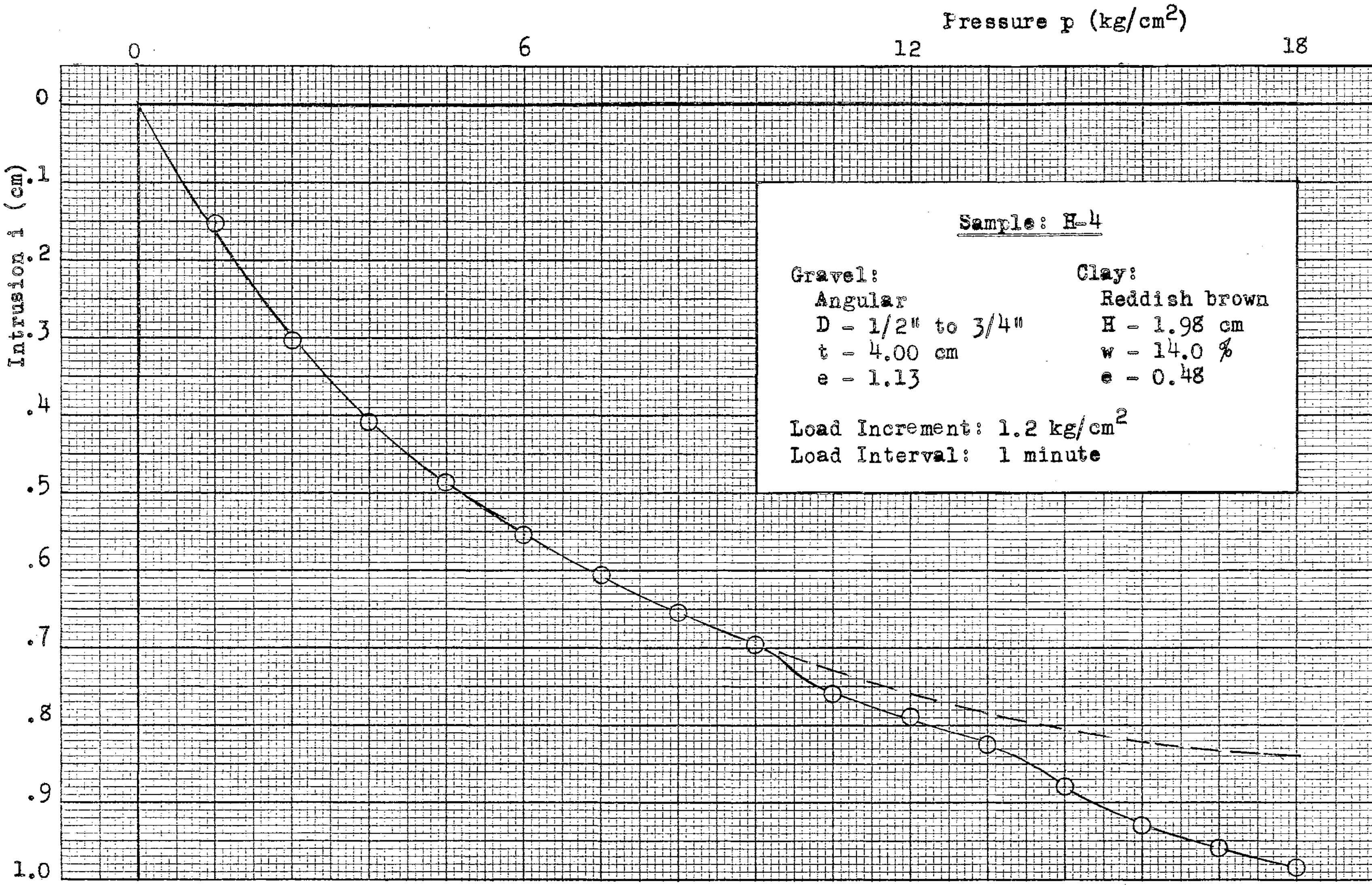


Figure 30. - Pressure - Intrusion Test (p - i Curve).

values are of the same order.

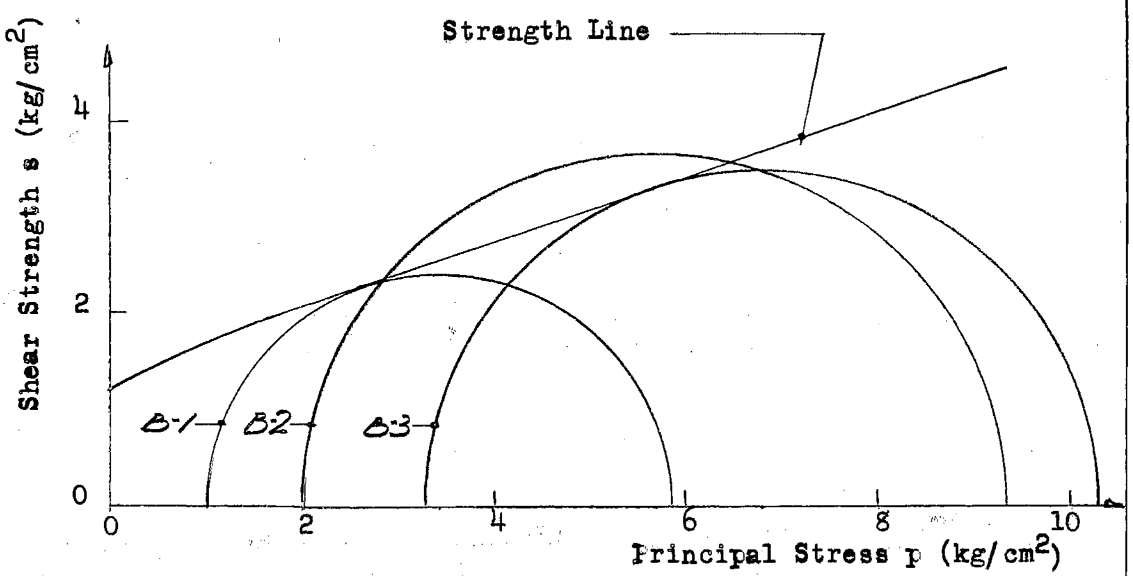
Future tests can probably be improved by confining them to lower pressures, and by increasing the time interval between loading increments. Both changes would serve to render the tests more realistic or practical.

Shear Strength

As stated in the chapter dealing with theoretical considerations, a variety of the properties of undisturbed clay influence its intrusive capabilities with respect to gravel. Foremost among these is shear strength, and while it is not possible to relate shear strength to intrusion in this thesis, three triaxial tests were conducted and designated as series B. It is hoped that the few values found will be of help in future investigations.

The high shear strength of this clay also affects a foundation on pier or pile footings, whether gravel sheets are used or not, by causing frictional heaving if adequate precautions are not taken. This problem will also be briefly discussed in this section.

Series B. The three samples were taken from 5 feet below grade at Ridge Road, Stillwater, Oklahoma, and were prepared as described earlier. It will be recalled that this clay was also used for test series A. The samples were consolidated under different pressures p_3 in triaxial testing machines. After one day loads were applied at the rates indicated until failure occurred at the vertical pressure p_1. The pertinent data and the resulting Mohr circles are presented in figure 31.



Sample	B-1	B-2	B-3
Height H (cm)	8.42	8.50	8.66
Area A (cm^2)	9.95	10.10	10.10
Void ratio e	0.42	0.44	0.47
Water content w %	14.2	14.0	14.9
Saturation S %	95.0	72.0	85.0
Rate of loading (kg/min.)	4.0	8.0	4.0
p_3 (kg/cm^2)	1.00	2.00	3.25
p_1 (kg/cm^2)	5.82	9.37	10.28

Figure 31.

Triaxial (Consolidated Quick) Test.

The very prevalent fissures in the clay, the rock impurities, the different rates of loading, or all these and other factors caused considerable inconsistency in the shear strengths indicated, so that a strength line can be only guessed at.

<u>Pier tests.</u> It has frequently been noted that clay not merely lifts a pier by swelling against the sole of the footing, but also bonds to the concrete and raises the foundation by virtue of high shear strength, sometimes to the extent that a rupture crack appears between pier and footing.

A preliminary test to measure such swelling, which, of course, would defeat the purpose of gravel sheets or any other method of combatting heaving if left unchecked, was conducted in the O.S.U. Soil Mechanics Laboratory under the supervision of J.V. Parcher. Permian red clay with low moisture content was compacted into a cylindrical bowl of 23 cm diameter and 8 cm height. A hole, 2 cm in diameter, was drilled into the center of the clay and filled with Portland cement paste. Water was made available to the clay through a tube connected to the bottom of the bowl, which contained a layer of sand under the clay, and the movement of the miniature concrete pier was observed on an extensiometer resting against its top. Steady heaving was noted which finally ceased after three months. At this time a total heaving deformation of 0.268 cm had taken place. When the water supply was shut off, shrinkage occurred, with the pier being lowered. At the time of this writing, six months after the beginning of the test, the pier has not reached its original position, nor has its movement ceased.

A corresponding test using full-scale piers is in the early preparation stage at the time of this writing. To be conducted by J. V. Parcher and D. W. Irby with the assistance of J. E. Catlin and the writer and under the guidance and counsel of R. E. Means, this test is to be conducted in three parts and will feature some refinements. Three piers are to be cast in pre-drilled holes, two of them with flared footings, the other without. The former are to be separated from their footings by a layer of sand or equivalent material with a reinforcing bar forming the only connection. One of these two piers is to be cast in a cylindrical tube made of foamy material having a very low shear strength, the other in contact with the clay. The third pier, without footing, is also to be cast in contact with the ground. Electric strain gages are to be mounted on the reinforcing bars of the first two piers in order to measure shear heaving force, while the amount of heaving of the third pier is to be read by means of a level.

It is thus hoped to gain some information as to the force required to restrain a pier when water becomes available to the surrounding desiccated clay, to the amount of possible heaving, and to the effectiveness of tubes of low shear strength material separating pier and clay. It will be attempted to use plaster-of-paris blocks to measure the moisture content of the clay during the test.

Due to the low permeability of the clay, these tests will be very lengthy, and publication of the results of this investigation will not be possible before 1960 or 1961.

CHAPTER V

SUMMARY AND CONCLUSIONS

Results of Research

If the basic question to be answered herein was whether it is possible to make use of the capability of clay to intrude into the void spaces of a cohesionless material in combatting heaving of foundations, the answer must be a qualified yes. The pressure - intrusion tests have shown that with the clay and gravel used, a certain rigidity of a building may well be able to overcome a small amount of differential heaving. While differential heaving is generally much smaller than the amount of total heaving, it may nevertheless often be necessary to increase the amount of intrusion. As was shown, this may well be done by selecting, for instance, a gravel of larger size.

Selection of a much coarser gravel will absolutely be necessary under normal conditions if it is desired to prevent heaving of a foundation altogether, as was demonstrated with time - intrusion tests. The amount of secondary intrusion, which alone can be considered to be of aid, was so small in every one of these tests as to make it valueless for practical purposes.

The question originally posed, however, requires more than an affirmative or negative answer. As to swelling, it was shown that under conditions where moisture is taken up by the clay only once after application of load, as must be the case if intrusion is to be effective, heaving is generally much less severe than in cases where the soil is allowed to swell and shrink repeatedly under an essentially constant pressure. Care must therefore be taken to seal the soil underlying a proposed structure from the surface, and thus to prevent periodic shrinking.

Intrusion tests showed, on the other hand, that the amount of intrusion of a natural clay of this type very much depends on the type, shape, and size of gravel used, as well as on the imposed pressure. More intrusion is caused by high pressure than low, by poorly graded gravel than well graded one, by smooth surfaces than rough ones, by large sizes of gravel than small ones. It is believed that these variables will make it possible for the engineer to select suitable conditions and to achieve a satisfactory foundation on Permian clay in many cases. First, however, a good deal more inquiry into the behavior of clay and gravel will be necessary.

Shortcomings of Tests

The tests related here were unsatisfactory in a good many ways. The assumption that thoroughly desiccated clays were being dealt with led to a selection of pressures for the swelling-consolidation tests of both types of clay which proved somewhat too large, making a number of tests less valuable than they could have been.

The selection of the size of gravel used for the intrusion tests was dictated by the equipment available for testing. It is evident that tests on much coarser gravels would have supplemented the information gained quite considerably.

The error caused by slaking of soil, when the dry clay samples were flooded, was later adjusted for. Subsequent tests were conducted on dry samples whose shrinkage cracks had been filled with wax. Some such method of filling cracks should be applied to all future tests of this nature.

Future Tests

At the end of the research program more questions are on the author's mind than were there at the beginning. Some of these may prove to be rewarding subjects for future investigations. Among them are:

a. How is repeated loading related to repeated drying of Permian clay?

b. What are the constant heaving values of these clays under pressures considerably higher than 5 tons/sq.ft.?

c. How is shrinkage related to temperature?

d. What is the numerical relationship between size of gravel and intrusion?

e. How is shear strength related to intrusion?

f. Can simplifying empirical formulae be found relating intrusion and the various variables that govern it?

g. How does moisture supply affect intrusion of saturated and dry clays?

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APPENDIX

The following letter symbols were adopted for use in this thesis, representing properties and having dimensions as indicated:

Symbol	Property	Dimensions
A	Area	cm^2, ft^2
C_c	Compression index	Dimensionless
C_s	Swelling index	Dimensionless
D	Diameter	cm, in.
e	Void ratio	Dimensionless
e_o	Original void ratio	Dimensionless
e_f	Final void ratio	Dimensionless
G_s	Specific gravity	Dimensionless
H	Height	cm, in., ft.
h	Total heaving	cm, in.
i	Total intrusion	cm, in.
k	Coefficient of permeability	cm/sec.
P	Total load	kg, lbs., tons
p	Unit pressure	kg/cm^2, tons/ft.^{2}
p'	Preconsolidation pressure	kg/cm^2, tons/ft.^{2}
S	Degree of saturation	Per cent
s	Shear strength	kg/cm^2, tons/ft.^{2}
t	a) Time	min., days
	b) Thickness of gravel	cm, in.
w	Water content	Per cent
w_f	Final water content	Per cent

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