SWELLING AND CONSOLIDATION CHARACTERISTICS

OF THE PERMIAN CLAY

By

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CHAPTER I

INTRODUCTION

General

The conditions and properties of the soil that support the structures designed today present a problem that requires a great amount of study. The phase of the study to be considered in this manuscript will be swelling and consolidation characteristics of the Permian clay.

Deformation due to consolidation is often the principal factor controlling the design of a structure. Settlement of most soils is accompanied by volume change of the soil. Unequal settlement under the structure introduces a movement of the foundation material which causes additional moments or stresses in the structure which could stress the members to the point of failure unless consideration of their magnitude has been given in the design.

Sufficient information should be obtained about the soils native to the construction site, and unless the information is reasonably accurate to the extent of obtaining the data necessary for the prediction of the behavior of the soil, the precision in the design of the structure is to no avail. For a long period of time, it has been the practice to obtain this stable foundation by several means other than the use of the soil as it exists in nature. Procedures such as driving piling, or continuing the footing to a depth sufficient to embed in rock, have been expensive construction methods which, at times, have been almost prohibitive in cost. To eliminate the use of unnecessarily expensive foundations would permit the construction of more structures with a given expenditure of money.

The problem of investigation of the soil to make certain of the soil conditions would entail the investigation of the following factors:

1. Bearing capacity of the soil with respect to failure and settlement.
2. Amount of settlement of the footing.
3. The possibility of plastic flow.
4. Lateral resistance of soil.
5. Possibility of scour.

The interpretation used to obtain the necessary information concerning a soil for a certain structure is based on several simplifying assumptions. Therefore, the accuracy of the predictions of soil behavior is possibly inaccurate to a degree. The purpose of these and similar tests is to provide data for more accurate prediction of the behavior of soil.

The properties of the soil can change within a very short distance. It is important to get good undisturbed samples at the site of the construction. These samples should be taken at various depths, and should represent the undisturbed conditions of the strata.

The theories and the mathematics of the characteristics and the mechanics of soils have been developed and proven, and this thesis will not include these developments except for the purpose of clarification in some individual problems. The interpretation of the information obtained from tests on soil samples will be the prime objective of this thesis.

The purpose of this investigation is to increase the knowledge of the physical properties of the permian red clays, which are common to large areas in Oklahoma.

A complete analysis of the properties of the permian clays is beyond the scope of this thesis. More specifically, the aim of this thesis is the study of the consolidation and swelling characteristics of the permian clay.

CHAPTER II

THEORIES RELATIVE TO CONSOLIDATION AND SETTLEMENT OF SOIL

Most of the structures built rest upon the soil, and the loads imposed are the cause of settlement. Failure of structures may occur, primarily, in two ways. The first may be introduced by a load that may produce shearing stress that exceeds the allowable shearing strength of the soil. The settlement would occur by virtue of the soil shearing laterally and at some angle with the horizontal. The second type of settlement could occur with the reduction of the volume of the soil mass resulting from the load placed upon it. This type of settlement is known as consolidation. The study of the first type of failure is beyond the scope of this thesis. The second type is our aim in this study.

The consolidation which occurs with the reduction of soil volume, due to water leaving the soil voids, is generally known as the primary consolidation. The total volume of the soil is composed of a volume of solids and of a volume of voids, the voids being filled with air and water. The water is forced under pressure to leave the voids, and the resulting action is settlement. The ratio of the voids content to solid content is known as the void ratio of the material.

The secondary consolidation occurs when the water has left the voids, and the pressure on the water has been transferred to the solid structure of the soil. This consolidation is of secondary importance since it occurs in a long period of time and is not significant to the life of the structure considered, with the exception of organic soils like peat and muck.

The void ratio of the soil, which is used to compute the consolidation, is the ratio of the void content to the solid content of the soil. Consider a volume of soil, which is composed of voids and solids, under an area, A. If the total depth of the volume is represented by the letter H_1, then the total volume will be H_1A. The solid depth is expressed by letter H_s, then the solid volume is H_sA.

When: V_s = volume of solid content

V_v = volume of void content

Then: $e = V_v / V_s$ (Equation 1)

$V_v = A(H_1 - H_s)$

Where: $H_1 - H_s$ = void depth

$V_s = AH_s$

Where: H_s = depth of solid

Then substituting in equation 1:

$$e = \frac{A(H_1 - H_s)}{AH_s} = \frac{H_1 - H_s}{H_s} = \frac{H_1}{H_s} - 1 \qquad \text{(Equation 2)}$$

The coefficient of consolidation can be obtained from the plot of the void ratio against pressure. The influence of time upon the development of compression strains, which is not indicated by the void ratio - pressure curve, is an important factor to the study of settlement. When the soil is subjected to a load, the water is forced under pressure to leave the voids in the soil in a direction providing the least amount of resistance. The direction and the number of directions of flow are significant in determining time for consolidation. The soil settles at a rate proportional to the rate at which the water leaves the voids, and this rate is a function of the coefficient of permeability.

The settlement is computed from the void ratio of the soil, before and after loading. The formula for the settlement, Δh, is derived from the illustration in Figure 1.

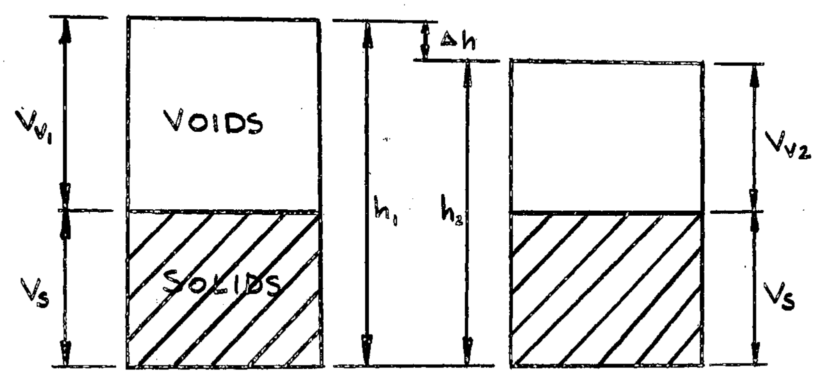

Fig. 1. Settlement of Soil Column

$$h_1 A = V_{v1} + V_s \qquad \text{and} \qquad h_2 A = V_{v2} + V_s$$

Then: $$h_1 = \frac{V_{v1} + V_s}{A} \qquad \text{and} \qquad h_2 = \frac{V_{v2} + V_s}{A}$$

$$\Delta h = h_1 - h_2 \qquad \text{(Equation 3)}$$

Then substituting h_1 and h_2 in Equation 3:

$$\Delta h = \frac{V_{v1} + V_s - V_{v2} - V_s}{A} = \frac{V_{v1} - V_{v2}}{A} \qquad \text{(Equation 4)}$$

When: $$\frac{V_{v1}}{V_s} = e_1 \qquad \text{and} \qquad V_{v1} = e_1 V_s$$

$$\frac{V_{v2}}{V_s} = e_2 \qquad \text{and} \qquad V_{v2} = e_2 V_s$$

Then substituting in Equation 4:

$$\Delta h = \frac{e_1 V_s - e_2 V_s}{A} = \frac{V_s \ (e_1 - e_2)}{A} \qquad \text{(Equation 5)}$$

And: $A = V_{v1} + \frac{V_s}{h_1}$

Then substituting for A in Equation 5:

$$\Delta h = \frac{h_1 \ V_s \ (e_1 - e_2)}{V_{v1} + V_s} \qquad \text{(Equation 6)}$$

Then divide Equation 6 by V_s:

$$\Delta h = \frac{h_1 \ (e_1 - e_2)}{\frac{V_{v1}}{V_s} + 1} \qquad \text{(Equation 7)}$$

Substitute $e_1 = \frac{V_{v1}}{V_s}$ in Equation 7:

$$\Delta h = \frac{h_1 \ (e_1 - e_2)}{e_1 + 1} \qquad \text{(Equation 8)}$$

The void ratio values of the initial and final loading condition of the soil, e_1 and e_2, are obtained from the void ratio - pressure curve drawn from the experimental data. The initial void ratio and final void ratio are found from the change in height of the sample after complete consolidation under a given pressure change.

The time for a percent of the total consolidation to occur is computed with the use of the variable factors, N and coefficient of consolidation C_v, and the solid depth of the strata, h_s. The time for the consolidation (1) to occur is obtained from the equation.

$$t = \frac{h_s^2 \ N}{2.47 \ C_v} \qquad \text{(Equation 9)}$$

Where: t = time in years

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(1) Indicates the reference number, see page 58.

c_v = coefficient of consolidation (cm. per min.) experimentally determined

h_s = solid height of the soil

N = factor depending on distribution of the consolidating pressure and particularly on the existence of multi-dimensional consolidation.

The value of N (2) for any particular percentage of consolidation depends on two factors pertaining to the installation for which a settlement analysis is being made. The first of these factors is the number of drainage faces of the compressible layer. The second factor upon which N depends is the character of the distribution of the consolidating pressure throughout the vertical height of the clay layer relative to the direction of outflow of the pore water during consolidation. In this thesis we assumed that the consolidating pressure is essentially the same at the top and the bottom of the clay layer and the pressure distribution is found to be rectangular in character (Fig. 2)

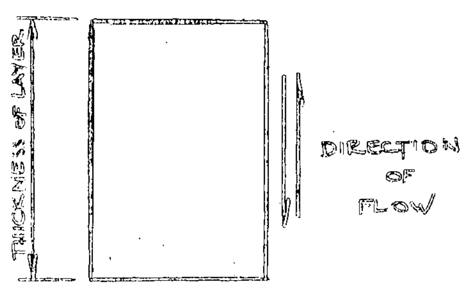


Fig. 2. Distribution of Consolidating Pressure

The coefficient of consolidation, C_v, is found for each of the experimentally applied loads, and the average value of the coefficient of the consolidation, which is approximately equal for all loads, is used to obtain the time of settlement.

The dry weight of the sample was measured after completion of the experiments and after the sample has been oven dried. The specific gravity of the soil was determined by estimation from previous tests, and the area was determined by the size of the ring of the consolidating unit.

The coefficient of consolidation is obtained from the following equations:

1) Square Root Fitting Method:

$$c_v = \frac{0.848H^2}{t_{90}} \qquad \text{(Equation 10)}$$

2) Log Fitting Method:

$$c_v = \frac{0.197H^2}{t_{50}} \qquad \text{(Equation 11)}$$

The coefficient of consolidation, obtained from the experiments conducted on the soil sample, is used to determine the time of settlement of the structure.

CHAPTER III

GENERAL DISCUSSION OF PHYSICAL PROPERTIES OF THE PERMIAN RED BEDS

Geology of the Permian Clay

The clays under consideration in this report are very stiff over consolidated red or reddish yellow clays in conjunction with the so-called Red Beds of Oklahoma and Texas. These beds of red and yellow sandstones and clays deposited during the Permian period of geologic history make up formations that are up to several hundred feet in thickness, consisting for the most part of intermingled layers of clays and sandstones. Occasional thin layers of disintegrated volcanic ash are found. The eastern edge of the outcrop of these Permian formations occurs along a north and south line some twenty or thirty miles east of Oklahoma City. East of this line, deposits of the Permian period are missing, the surface being covered with the older Pennsylvanian and Mississippian deposits. West of this line, the Red Bed deposits occur on the surface, becoming thicker toward the west until near the west edge of Oklahoma they disappear beneath the surface and are covered by the younger deposits of the Mesozoic era and later periods. The upper portion of the Permian formations which are left in the western part of the state include extensive deposits of salt and gypsum. This western edge of the outcrop is roughly indicated by the location of the gypsum industry at Southard in Blaine County, Oklahoma, and Sweetwater in Nolan County, Texas.

During the period of deposition the area covered by these Red Beds was the bottom of an inland sea surrounded by relatively high Mountains. Freshets pouring down from these mountains carried sand along with silt and colloidal clay particles which were deposited together forming flocculent marine clays. Most marine clays having a flocculent structure consist of very fine silt grains and flocks of colloidal size particles. A good many deposits of the clays under consideration are composed of sand and colloidal clay. Sometimes boulders of sandstone consisting of quartz sand grains cemented with reddish colored chert are formed in the clay deposits. These boulders vary in size up to several feet in diameter. The climate of the region during this period was arid. In normal seasons some of the streams seem to have dried up before reaching the sea, depositing their burdens as alluvialfans in the desert between the mountains and the sea. Such deposits were made over a large portion of the area as the sea dried up. During the Permian period the sea almost dried up three times, depositing three layers of gypsum. Because the deposits are complex and variable, the structure and composition of the clays is also quite variable and complex.

Physical Properties

The physical properties of these clays are affected by the mineral content. Those minerals such as mica, which is flat and scalelike in shape, give to the clay its plasticity. Most of the scalelike minerals are elastic. Clays composed of these minerals, therefore, are highly compressible and are capable of considerable elastic rebound. The mineral bentonite, which is formed by the disintegration of volcanic ash, is present to some extent in most of the clays in this area. Since bentonite takes up a great deal of water, about 400% at the liquid limit, 40% at the plastic limit and 16% at the shrinkage limit, clays containing considerable bentonite will exhibit the same property to a considerable extent. Such clays will in general shrink and swell a great deal as water is evaporated and absorbed.

Special consideration will be given to the Permian clays in this thesis because they cover a considerable portion of Oklahoma and the southwest and because a great deal of damage to buildings has resulted from the use of foundations not suited to soil conditions in these deposits and the climatic conditions that exist in this area. One should keep in mind that, in general, it is the climatic condition that causes similar damage to buildings on other clays than the Permian. But a very large portion of the buildings in Oklahoma are in the area covered by these Permian deposits.

CHAPTER IV

LABORATORY PROCEDURE AND DATA OBTAINED

Before the solution can be ascertained for a soil problem, the properties of the soil in question must be obtained from laboratory tests or other procedures. The properties of soils in general cannot be determined with as great a degree of accuracy as those of most other construction materials, and even though the properties were known for one sample of soil beneath an area, the properties of the entire soil affected would be only vaguely known, since materials may vary over a very wide range in a very small area. Experience, imagination and judgment are of considerably greater importance, and mathematical analysis is of less importance, in the design of earth structures than it is in the design of structures of most other materials. However, the information of the properties obtained from the tests provides sufficiently accurate data for the solution of the problem.

Consolidation Test

As a saturated soil is subjected to an increment of load, the load is carried by the water present in the pores of the soil, since the water is considered incompressible with relation to the soil structure. The water is forced from the soil, and the load is gradually transferred to the soil structure. The reduction of the volume of the sample is equal to the volume of water that has left the sample. This reduction in the volume of the sample is known as consolidation. With the application of calculated loads on a sample of definite area, the consolidation of the soil is measured.

In many instances the settlement of a structure is due to presence of one or more layers of soft clay located between layers of sand or stiffer clay. In the laboratory this condition is simulated most closely by confined compression or consolidation test.

During consolidation tests the specimen is completely confined by a metal ring, Fig. 3. The load is transmitted to the upper and lower faces of the specimen through two porous disks that permit water to flow into or out of the clay. The deformation is measured by means of a dial gage.

The following apparatus are used in the preparation of the sample and the test:

1) Consolidation unit
2) Timing equipment
3) Device for providing water to sample
4) Specimen trimming equipment
    a) Rotary lathe
    b) Knives and lathe tool
    c) Steel ring to hold sample (floating ring type)
5) Balances, sensitive to 0.01 and 0.1 grams.
6) Scale
7) Gages
8) Evaporating dish
9) Drying oven
10) Desiccator

The samples were obtained from the following two places:

1) From the east side of the new Telephone Building (Stillwater) at a depth of 10 feet below the surface. A sufficiently large sample was cut to insure an undisturbed condition for the sample to be tested. The soil sample was a dark red, sandy jointed clay (clay A).

2) From the foundation of the Civil Engineering Laboratory at Oklahoma State University. The soil sample was a reddish brown sandy jointed clay, and was taken at a depth 4 feet below the surface of a concrete floor slab which had been in existence for about 20 years. (Clay B).

The following procedures were used in cutting the sample:

1) The sample was trimmed to the approximate desired size while care was taken to prevent any disturbance such as cracks in the sample. The sample was trimmed to a diameter slightly larger than the base of the rotary lathe.

2) The sample was placed on the rotary lathe and trimmed to the desired diameter. The soil settled into the steel ring placed on the lathe when the proper diameter was attained.

3) The sample was trimmed flush with the top and the bottom of the ring container.

The sample area and depth were known from the predetermined size of the ring. The loading program was fixed according to the three ring sizes which were used for the experiment.

The system of loading was as follows:

1) The small ring sizes of diameter (4.03 cm.) were loaded in the following sequence: 0, 84, 96, 192, 384, 192, 96, 0, kg.

0, 72, 96, 192, 334, 192, 96, 0, kg.

2) The medium ring sizes (of 6.45 cm.) were loaded as follows:

0, 0, 6, 12, 24, 48, 384, 96, 24, 0 kg.

0, 18, 48, 96, 192, 384, 768, 192, 48, 0 kg.

0, 36, 48, 96, 192, 384, 768, 192, 48, 0 kg.

0, 54, 96, 192, 384, 768, 192, 96, 0 kg.

0, 72, 96, 192, 384, 768, 384, 96, 0 kg.

3) The large ring sizes (of 10.13 cm.) were loaded as follows:

0, 240, 384, 768, 1536, 768, 240, 0 kg.

0, 288, 384, 768, 1536, 768, 288, 0 kg.

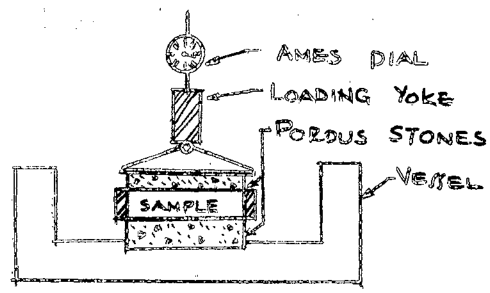


Fig. 3. Floating Ring Container

The sample was next weighed, and the weight recorded as tare plus soil wet at the beginning of test. The sample was next placed on the consolidation unit and the following procedures were used:

1) The sample was placed between the two porous stones, (Fig. 3), making sure that both the porous stones were properly centered on the sample surfaces and that the sample was centered under the loading device. The floating ring type was used for this test.

2) The loading unit was adjusted and lowered until it just made contact with the porous stones, and consequently with the sample itself.

3) The bar holder for the vertical dial was screwed into position and the gage set to register deformation of the sample. Volume changes during the test were registered by a vertical-deflection (Ames Dial).

4) The sample was left in the consolidating unit for several days

until it had dried, to determine the void ratio of the soil mass which assumes its lowest value at shrinkage limit. The dial reading was recorded every day, until there was no further reduction in volume. The total volume reduction corresponds to the volume of the water evaporated during the shrinkage process up to the shrinkage limit. After the sample dried out the initial load was carefully applied. Readings were recorded immediately after releasing the load, and when there was no volume change indicated by the dial reading, the sample was flooded and and readings of vertical deflection were taken for the time increments of 0, ½, 1, 2, 4, 8, 15, 30, 60 minutes etc. Thus the thickness of the sample could be determined at every elapsed time at which a reading was taken.

5) The load increments were added at intervals of sixty-four hours.

6) Upon completion of the test, the sample was removed from the consolidation unit and weighed; then it was placed in a drying oven where the moisture was removed, and the dry weight of sample was obtained.

7) After removing the sample from the consolidating unit, the machine calibration was determined using the same setup without the sample, and using the same porous stones as used in test in the machine, in order to determine the deformation of the machine and other parts for the same loads in the same order as used in test.

Time - Dial Reading Curves

Time - Dial reading curve was drawn for each of the load increments. (See Figures 5, 6, 7, 8, 9, 10, 11, and 12.) The curve was drawn on semi-logarithmic paper with the deformation reading plotted against the time in minutes on the logarithmic scale. Tangent lines were constructed from each of the straight portions of the curve, (3) and the intersection of the tangent lines taken as the point of 100 percent consolidation. The zero point was located by extending the curve as a segment of a parabola, and mathematically computing the zero reading. The 50 percent consolidation point was obtained as the halfway point between the zero reading and the 100 percent reading. The time for the 50 percent settlement to occur in the sample was then read from the scale. The value of t_{50} was used to determine the coefficient of permeability of that load increment, and this is shown in the following equation:

$$k_{\text{(for certain unit load)}} = \frac{T_s \gamma_w a_v H^2}{(1 + e_{av})\, t_{50}} \qquad \text{(Equation 12)}$$

in which:

k = coefficient of permeability

T = Time factor = (.2) for u = 50%

γw = unit weight of water

a_v = coefficient of compressibility = $\frac{\Delta e}{\Delta p}$

H = half of height of sample

e_{av} = average void ratio during load increment

t_{50} = Time for 50% settlement to occur

e - Log P Curve

The void ratio-pressure curve is a plot of the load increment against the void ratio computed for each of the loads after 100% consolidation (see Figs. 14 to 28).

The initial void ratios were computed from Equation 2:

$$e = \frac{h_1 - h_s}{h_s} \qquad \text{(Equation 2)}$$

Where:

h_1 = height of ring = Height of sample at start of test

$$h_s = \frac{W_s}{AG_s} = \text{Height of solid} \qquad \text{(Equation 13)}$$

The pressures were computed from load increments P divided by the area of the sample. Changes in void ratio, Δe, were determined from change in height of the sample, Δh, as follows:

$$\Delta e = \frac{\Delta h}{h_s} \qquad \text{(Equation 14)}$$

Discussion of procedure

Side Friction: Part of the load applied to a consolidation specimen is transferred to the container wall by friction between the wall and the specimen (Figure 4) illustrates the friction effect in both the fixed-ring and the floating-ring containers.

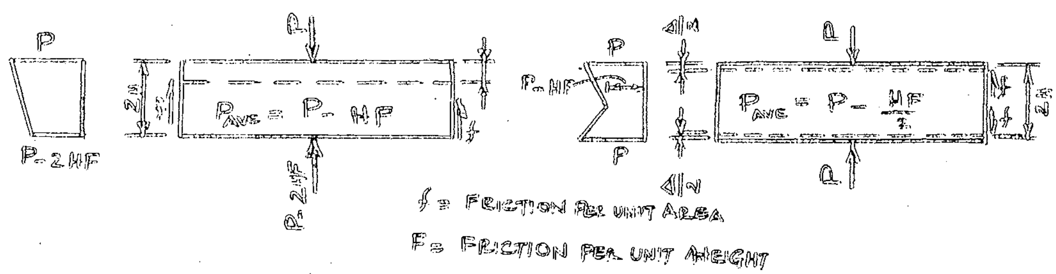


Fig. 4. Side Friction

In order to obtain the best results from a laboratory test, the magnitude of applied load should be increased in such a way that the average force within the specimen is of desired amount. However, this is not an easy thing to do because of the difficulty of determining the proper friction value.

Side friction is normally neglected in routine consolidation testing because of its minor effects and because of the difficulty of determining its magnitude. It has been suggested that an increase to the applied load by 10% - 20% would probably compensate for friction in this clay.

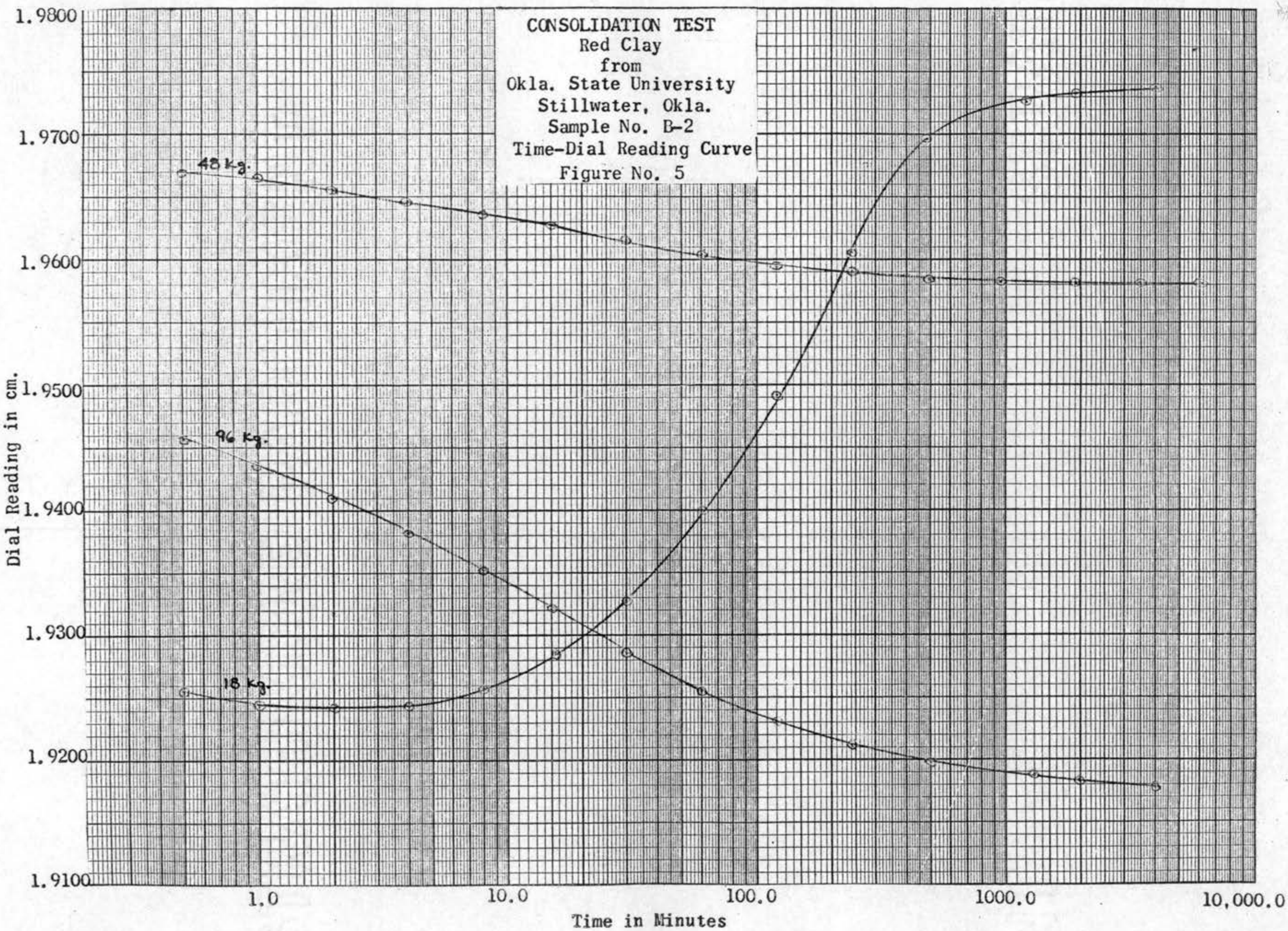
General Discussions: The duration for each load should be the same for an accurate void-ratio - pressure curve. If it is not uniform, secondary consolidation effects are more pronounced. For the clay under investigation each loading increment was allowed to act for approximately sixty-four hours. If any one increment were allowed to run for a longer time, a larger value for the difference in voids ratios would be obtained, resulting in irregularities in the e-log p curve. However, if each increment were allowed to stand for a longer interval, for example one week, then the resulting virgin curve would be merely displaced vertically downward without appreciable change of slope. The values of void-ratio change and compressibility coefficient are, therefore, independent of the time allowed for each loading increment, so long as time intervals are equal.

Test procedure provided for a consolidation time for every load increment of sixty-four hours. This period of time was established for this clay because compression or swelling of the sample was observed to be negligible beyond this point. This, however, was not the case at the end of each test when the sample was unloaded. Swelling was occurring slowly and would continue for a few days.

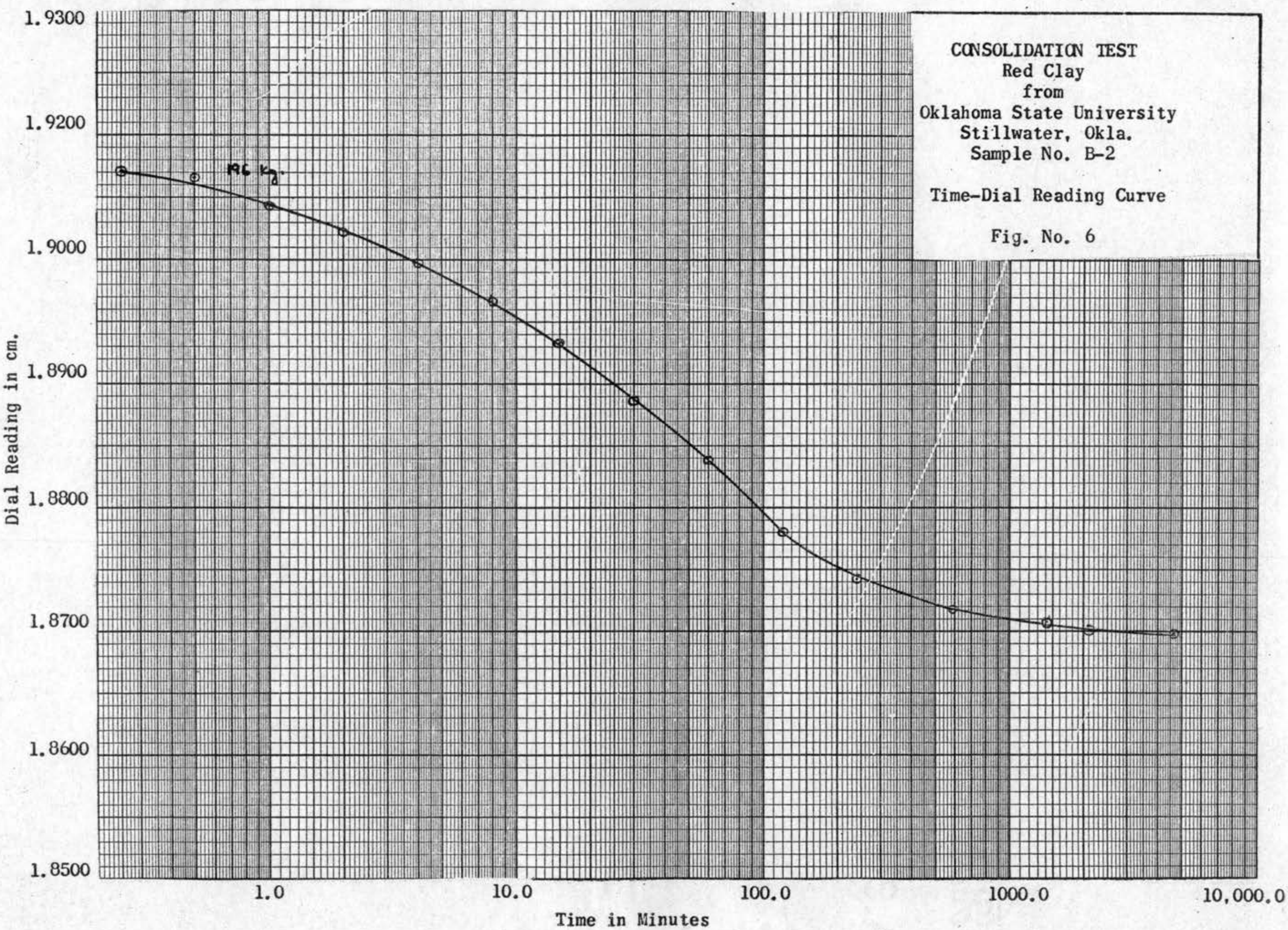
The results of laboratory tests depend on the size of the specimen (3) used. A series of tests on five widely different clays in which both 4.25 in. diameter by 1.25 in. thickness and 2.75 in. diameter by 0.85 in. specimens were used indicated that the pressure-void ratio curve was essentially independent of size, but the rate of compression was greatly dependent on size because:

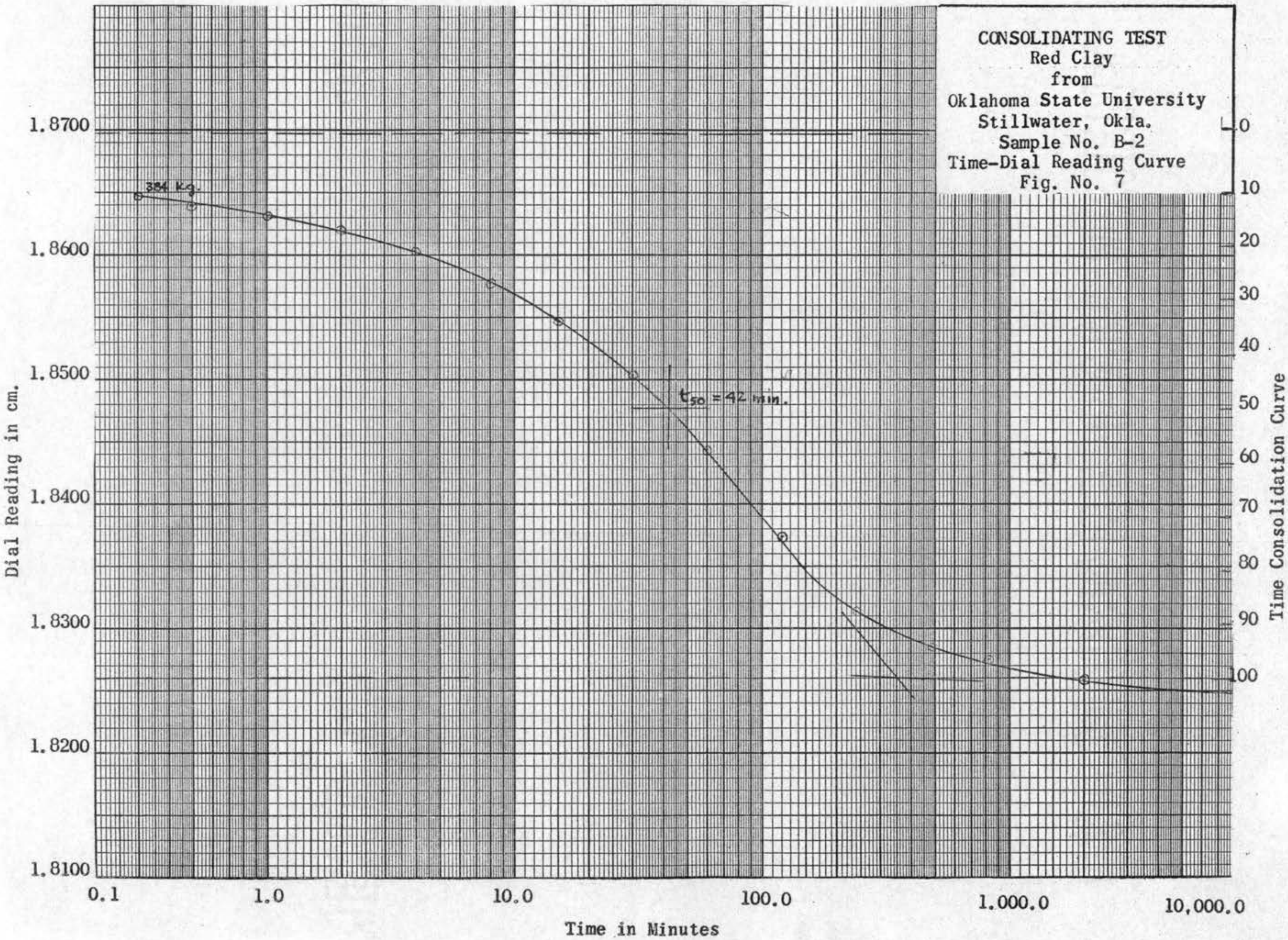
1) Side friction is less for the smaller specimen than for the big specimen.
2) The smaller specimen size is usually more disturbed than the larger one.

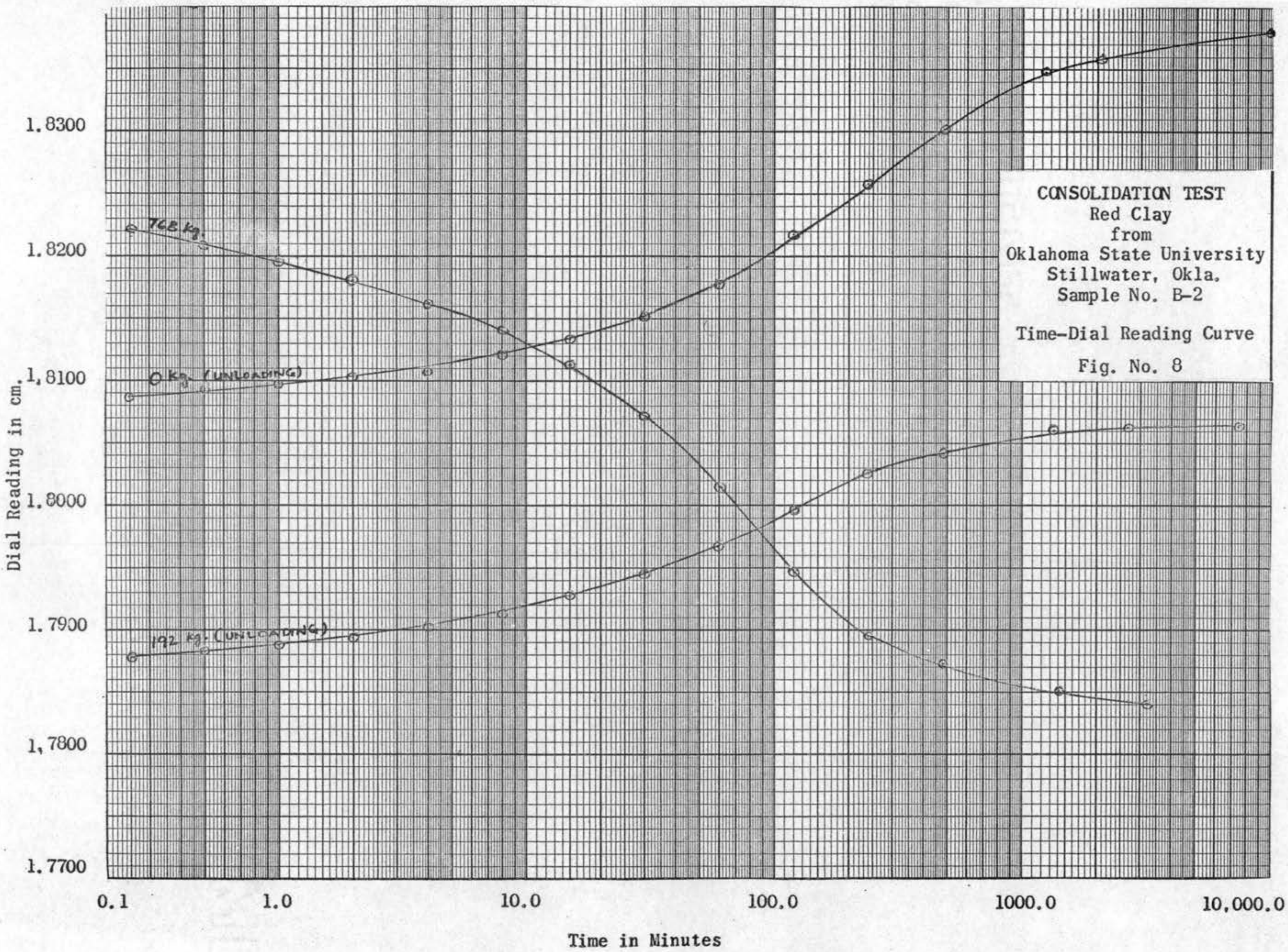
Higher coefficients of consolidation were obtained on the larger specimens. A ratio of specimen diameter to thickness of about three to four is recommended. Diameters greater than 2 1/2 to 2 3/4 inches are desirable.

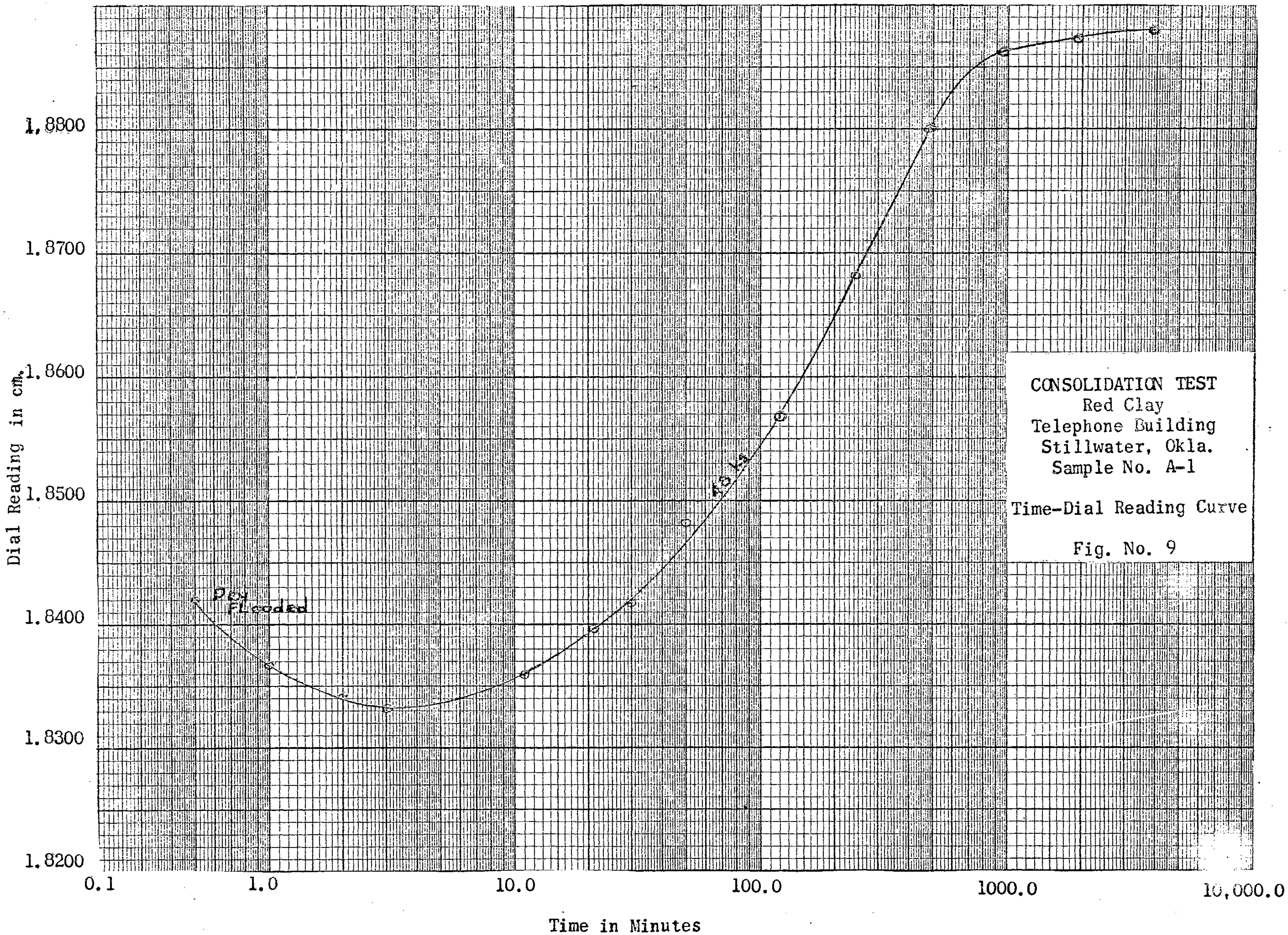


CONSOLIDATION TEST
Red Clay
from
Okla. State University
Stillwater, Okla.
Sample No. B-2
Time-Dial Reading Curve
Figure No. 5
48 Kg.
96 Kg.
18 Kg.
1.9800
1.9700
1.9600
1.9500
1.9400
1.9300
1.9200
1.9100
Dial Reading in cm.
1.0
10.0
100.0
1000.0
10,000.0
Time in Minutes


CONSOLIDATION TEST
Red Clay
from
Oklahoma State University
Stillwater, Okla.
Sample No. B-2
Time-Dial Reading Curve
Fig. No. 6
196 kg.
1.9300
1.9200
1.9000
1.8900
1.8800
1.8700
1.8600
1.8500
Dial Reading in cm.
1.0
10.0
100.0
1000.0
10,000.0
Time in Minutes


CONSOLIDATING TEST
Red Clay
from
Oklahoma State University
Stillwater, Okla.
Sample No. B-2
Time-Dial Reading Curve
Fig. No. 7
384 kg.
t_{50} = 42 min.
1.8700
1.8600
1.8500
1.8400
1.8300
1.8200
1.8100
Dial Reading in cm.
0
10
20
30
40
50
60
70
80
90
100
Time Consolidation Curve
0.1
1.0
10.0
100.0
1,000.0
10,000.0
Time in Minutes

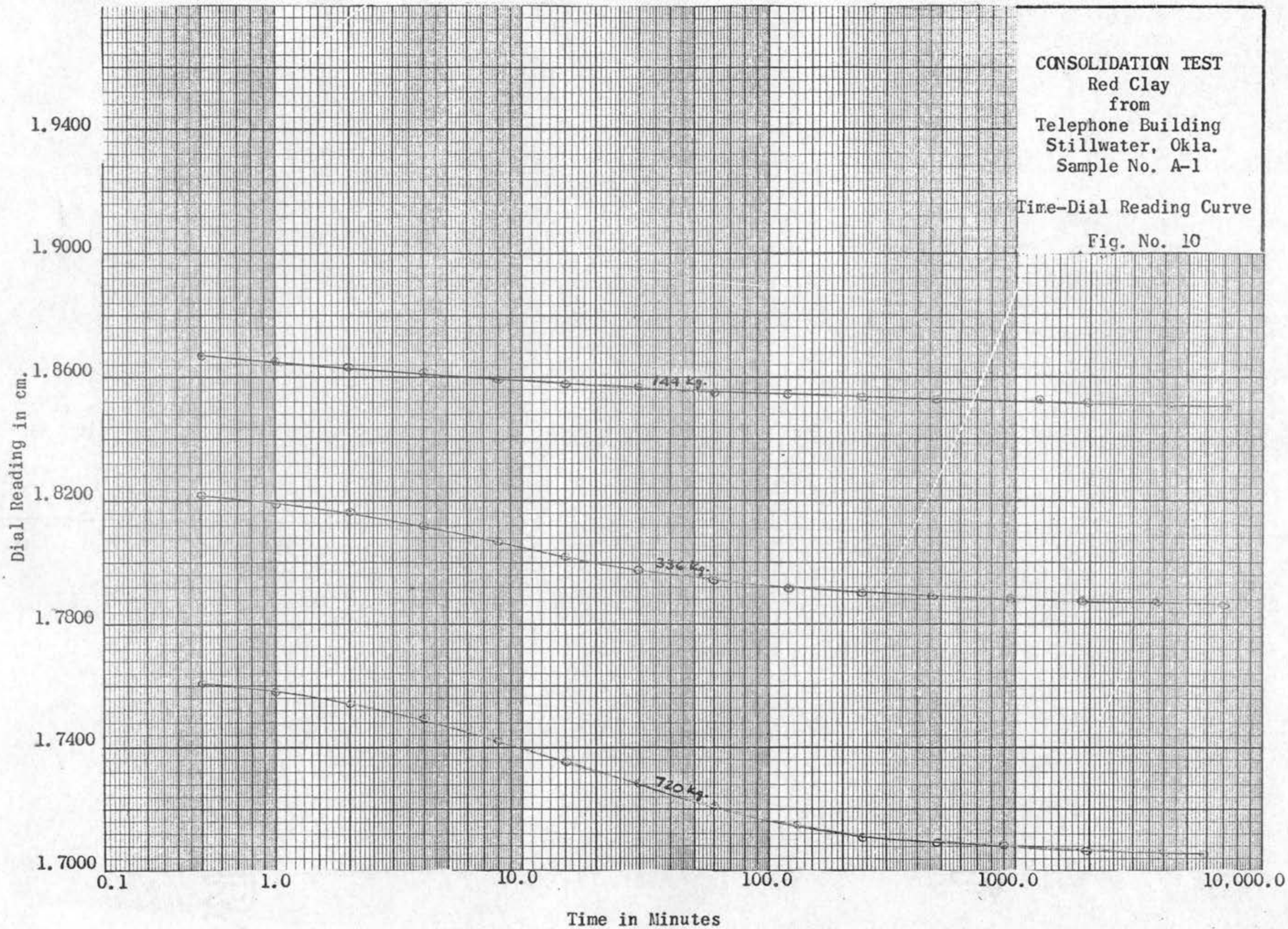

CONSOLIDATION TEST
Red Clay
from
Oklahoma State University
Stillwater, Okla.
Sample No. B-2
Time-Dial Reading Curve
Fig. No. 8
768 kg.
0 kg. (UNLOADING)
192 kg. (UNLOADING)
Dial Reading in cm.
1.8300
1.8200
1.8100
1.8000
1.7900
1.7800
1.7700
0.1
1.0
10.0
100.0
1000.0
10,000.0
Time in Minutes

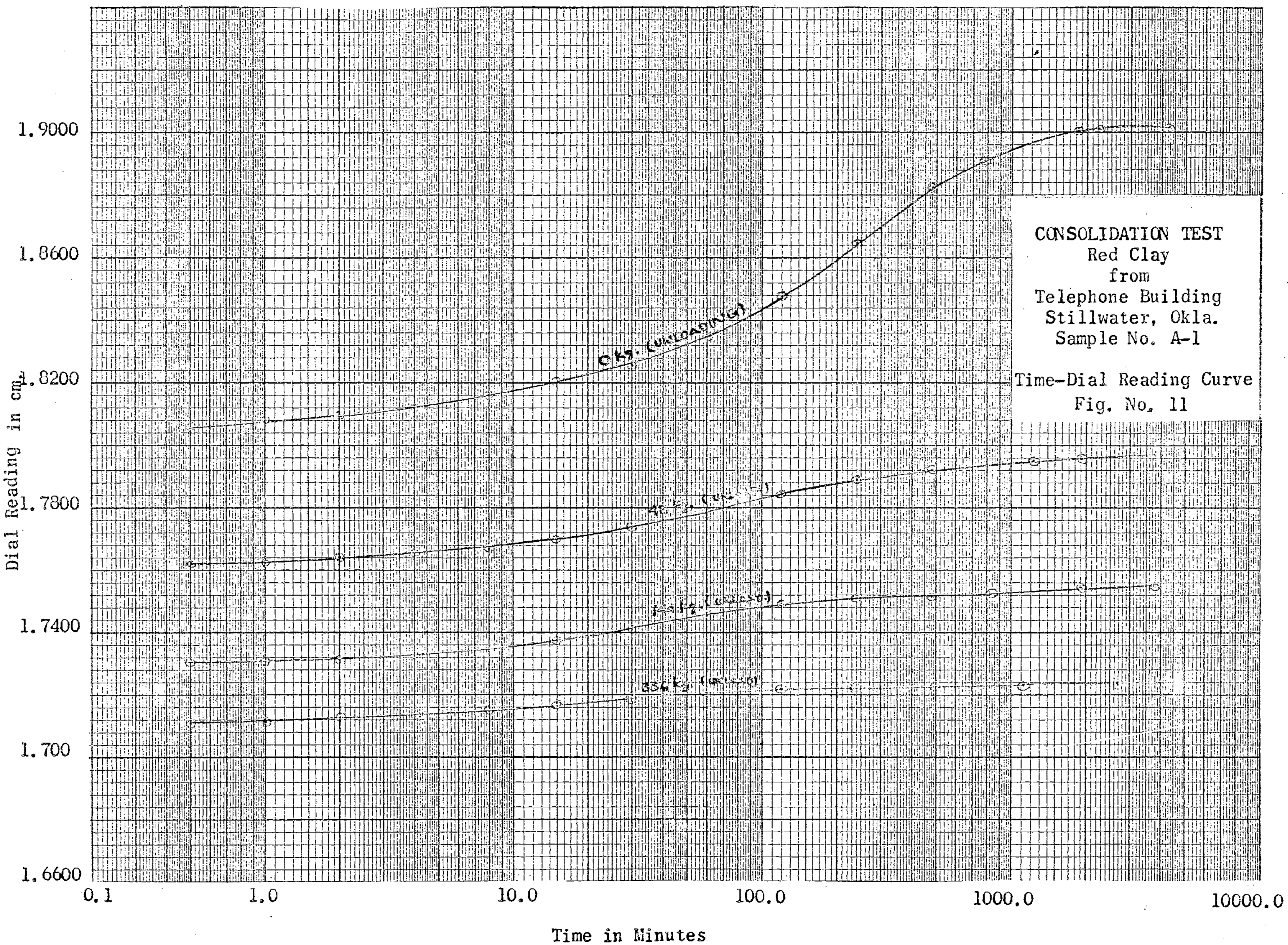


CONSOLIDATION TEST
Red Clay
Telephone Building
Stillwater, Okla.
Sample No. A-1

Time-Dial Reading Curve

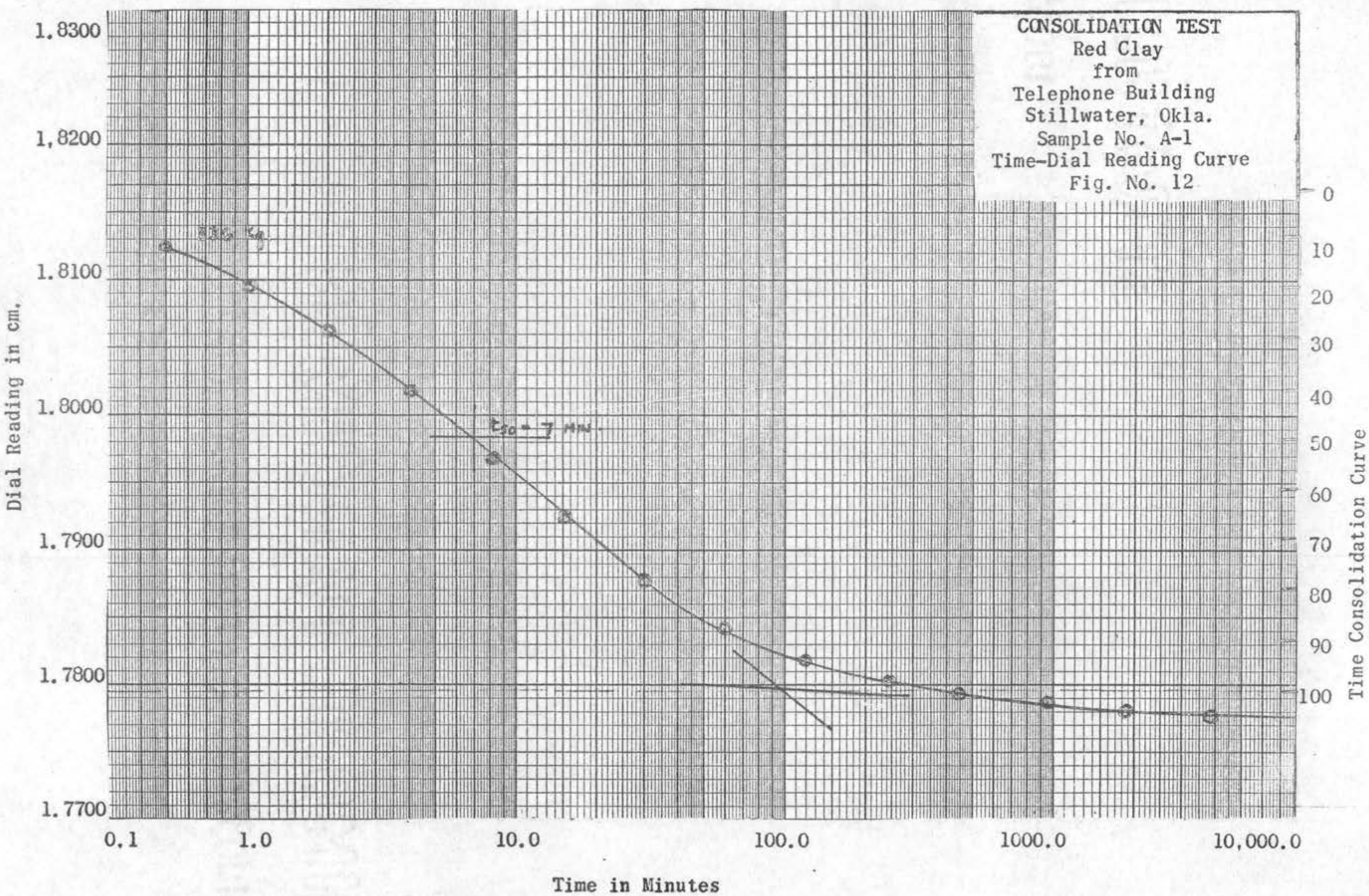
Fig. No. 9


CONSOLIDATION TEST
Red Clay
from
Telephone Building
Stillwater, Okla.
Sample No. A-1
Time-Dial Reading Curve
Fig. No. 10
1.9400
1.9000
1.8600
1.8200
1.7800
1.7400
1.7000
Dial Reading in cm.
144 kg.
336 kg.
720 kg.
0.1
1.0
10.0
100.0
1000.0
10,000.0
Time in Minutes



CONSOLIDATION TEST
Red Clay
from
Telephone Building
Stillwater, Okla.
Sample No. A-1

Time-Dial Reading Curve
Fig. No. 11



CONSOLIDATION TEST
Red Clay
from
Telephone Building
Stillwater, Okla.
Sample No. A-1
Time-Dial Reading Curve
Fig. No. 12
Dial Reading in cm.
1.8300
1.8200
1.8100
1.8000
1.7900
1.7800
1.7700
0.1
1.0
10.0
100.0
1000.0
10,000.0
Time in Minutes
Time Consolidation Curve
0
10
20
30
40
50
60
70
80
90
100
t50 = 7 MIN.

CHAPTER V

ANALYSIS OF TEST RESULTS

The results are plotted in the form of a curve representing the final void ratio corresponding to each increment of pressure as a function of the total pressure. It is convenient to plot the pressure to a logarithmic scale. The diagram is then known as an e-log P curve (see Figures 14 to 23)

The engineer is interested in the e-log P relationship for the clay in the field, not in the laboratory. Therefore, he requires some procedure for interpolating from the results of the laboratory tests to those representative of conditions in the field. This could be achieved by good judgment and practical experience. The design of ordinary soil-supporting or soil-supported structures is necessarily based on simple imperical rules, but these rules can be used safely only by the engineer who has a background of experience.

Since personal experience is necessarily somewhat limited, the engineer is compelled to rely at least to some extent on the records of the experiences of others. If these records contain adequate descriptions of the soil conditions, they constitute valuable information. Otherwise, they may actually be misleading.

According to information obtained from e-log P curves, sample A-1 swelled from the shrinkage limit under a load 1.47 tons per sq. ft. From a void ratio of 0.3686 to 0.3974. This corresponds to an increase of

.224 in per ft. (see Fig. 23). Sample B-1 was allowed to air dry like the rest of the samples to a water content below the shrinkage limit prior to its preparation for testing. A confined compression test was made on the material. The semi-log plot of the relationship existing between the pressure and void ratio is shown in Figures 14 to 28.

The e-log P plot for Sample B-1 (Fig. 14) is typical in most respects to all samples.

A study of void ratio-pressure curves for clay B (see Figures 14 to 22) indicate that the swelling decreases with the increase of the pressure. Finally, under higher loads, no swelling will take place, and settlement will take place as indicated in Figures 14 to 17. This is due to the increase of water content in the clay under the loaded area due to softening of the clay. The direction and magnitude of movements of this clay will depend upon the load applied.

From the study of 6 tests of clay A, it may be seen that a given increment of pressure produces a smaller change in void ratio along the precompression branch of the curve (see Fig. 23) than the same change in pressure would cause when applied along the virgin portion of the e-log P curve. This holds true also for clay B. Therefore, additional loads less than the preconsolidation pressure on overconsolidated clays will cause relatively little settlement.

The e-log P curves for both clays indicate that these soils were preconsolidated clays, and clay B differs from clay A probably by being redeposited in a recent geological era. This is why the preconsolidation load is found to be smaller, comparatively, for clay B than for clay A.

In order to determine the approximate preconsolidation load two methods were applied:

1) The graphical construction method by A. Casagrande (7).

2) By extending the dry void ratio line to the void ratio pressure curve (6).

In this thesis we shall discuss the second method for determining the preconsolidation load. The pressure determined by this method is assumed to be the preconsolidation pressure due to dessication.

This method says: If a line is drawn from the void ratio of the clay below the shrinkage limit, $e_{(dry)}$, to the e-log P curve, the intersection should indicate the pressure P_s required to compress the soil to the same void ratio as the tension in capillary water at the shrinkage limit. The dry clay loaded to a pressure equal to P_s (pressure at intersection of $e_{(dry)}$ with e-log P curve) 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.

To check this assumption and to get a relationship between pressure and amount of swelling, a series of 7 tests were run in the Soil Mechanics Laboratory at the Oklahoma State University (6) during 1956 on a sample of brown clay from 6 feet below the surface at Blackwell, Oklahoma. This soil will identify in this report as sample C. This sample had dried out after several years in a casing of cheese cloth and paraffin. Consolidation tests were run on this sample. Each of the specimens was subjected to a different pressure, flooded and allowed to swell, and loaded in increments as a consolidation test up to about 24 tons per sq. ft. Also another two series totaling 15 tests were run in 1957-1958 by the author of this thesis and the results of these are shown in Table I. The relationship between pressure and swelling for sample C is shown in Fig. 13.

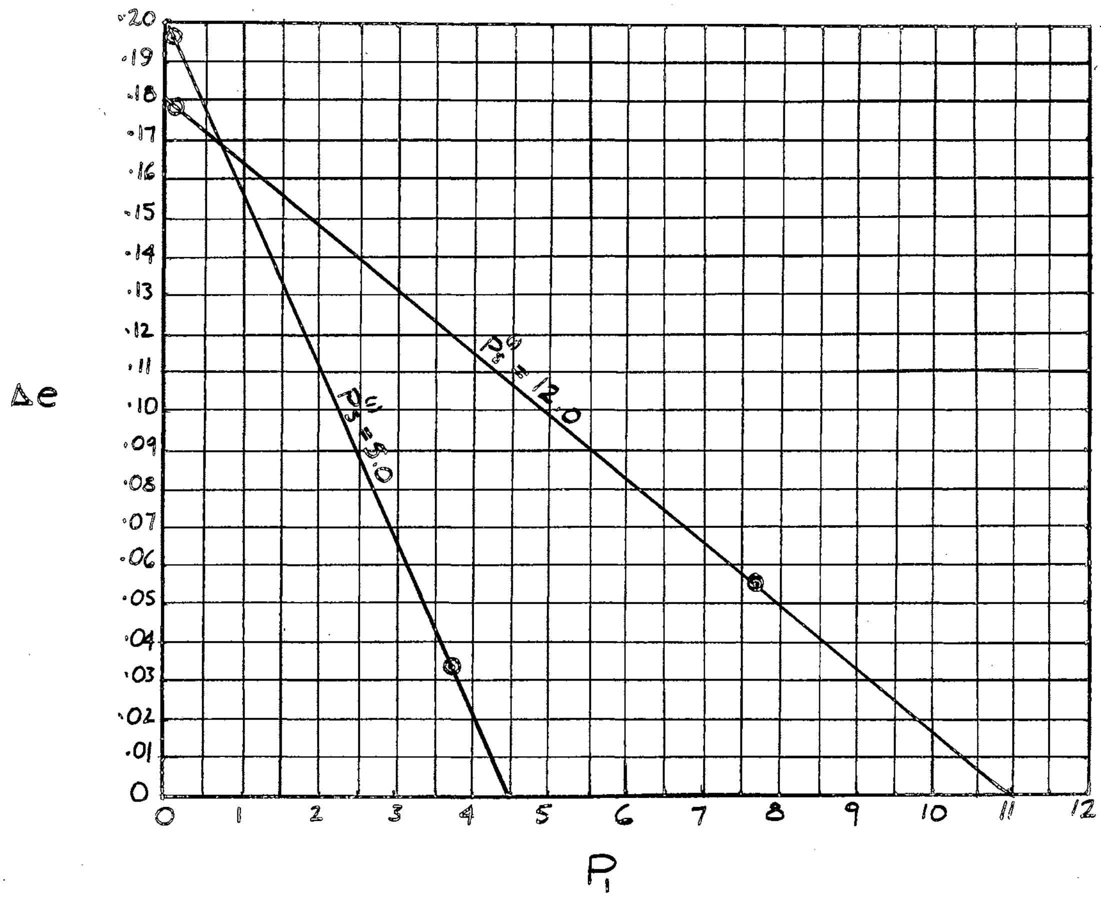


Fig. 13. Relationship Between Pressure and Swelling

At first glance one is led to discount the reliability of the tests because of the wide variation in the value of P_s for different specimens of the sample of clay. When pressure is plotted against swelling and lines drawn through points indicating swelling of the specimens with the same P_s as done in Fig. 13, it can be seen that the lines extended intersect the line of zero swelling at approximately the indicated P_s. This suggests that possibly the method of determining the shrinkage pressure as indicated in e-log P curve is fairly reliable and that the swelling is inversely proportional to the applied pressure.

Table I indicates the results of methods one and two for determining the approximate consolidation loads for soils A and B. The values of $P_s^{(1)}$ and $P_s^{(2)}$ are close to each other for clay A and B, as shown in Table I. When pressure P_1 is plotted against Δe in order to determine $P_s^{(1)}$ (as shown in Figures 29 and 30) for these samples. The curves did not pass through the corresponding points which were determined from the e-log P curves, and did not indicate a linear relationship but the evidence of these tests was inconclusive. This might have occurred because of several things.

1) Because of the disturbance of the sample during the test procedure one could see (Table I) that sample A-3 was consolidated under $P_1 = 4.41$ tons-ft.$^{-2}$, while sample A-6 swelled under $P_1 = 4.62$ tons-ft.$^{-2}$. Although clay B was not disturbed as much as sample A, both samples were disturbed.

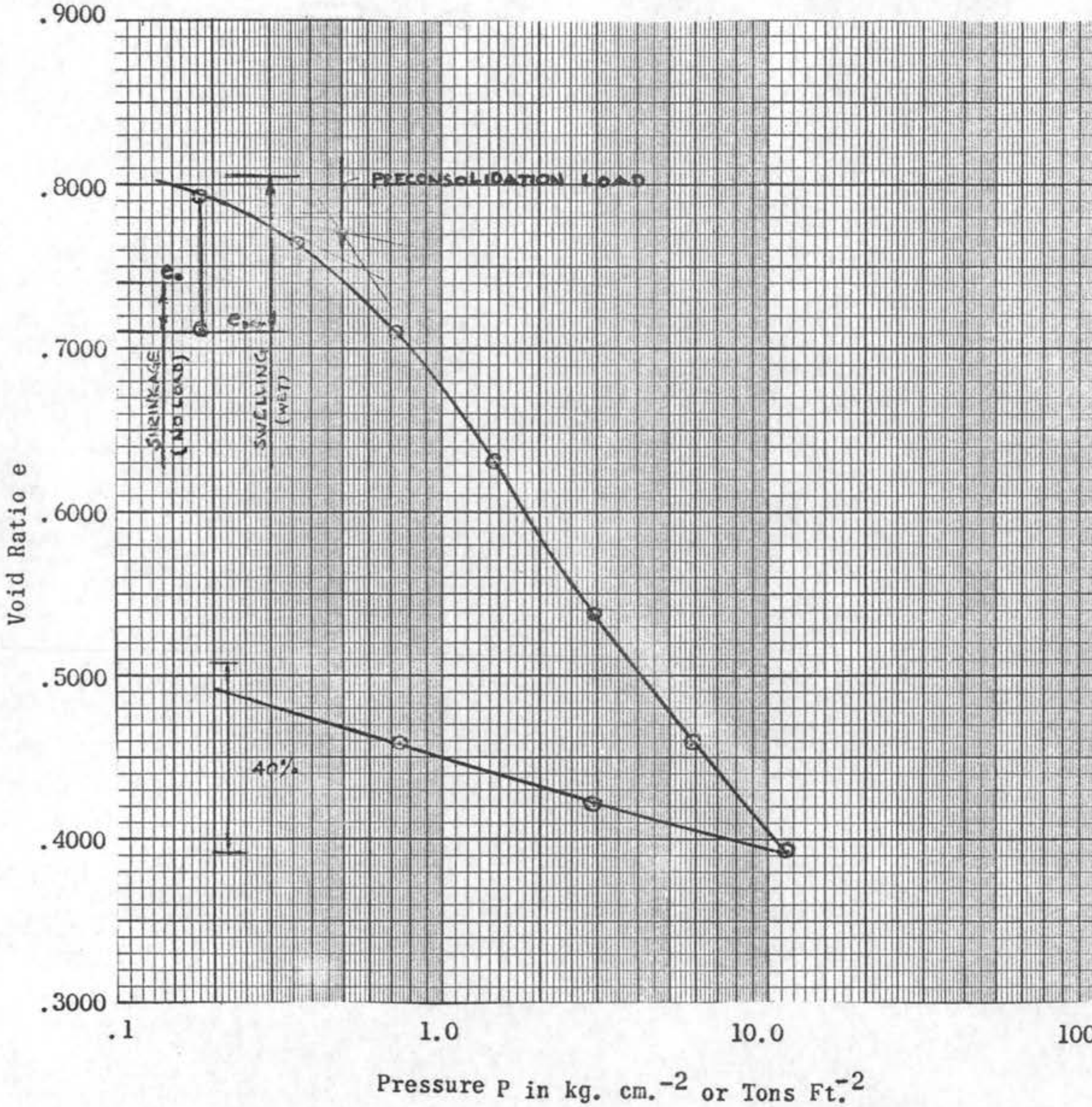
2) There were not enough values taken to derive swelling characteristics for both clays at smaller increments of loading, which enable us to justify an exact determination for this relationship. Thus, Figures 29 and 30 lack enough points to present this relationship properly.

3) Clay A was subjected to a heavy overburden pressure, and had been subjected to many cycles of wetting and drying. Clay B had not been subjected to as heavy an overburden pressure as sample A.

From the results of the observation of e-log P curves, the consolidation load of clay A is found to be approximately 4.5 to 5.0 tons-ft.$^{-2}$ and for clay B is approximately 2.0 - 2.5 tons-ft.$^{-2}$

The value of k was found to be about 1×10^{-9}. This compared favorably with previous results obtained for this soil. The value of k indicates that this soil is as impervious as other high plastic clays.

For each loading increment, the values of coefficient of consolidation c_v and the primary compression ratio, c_c are computed using the log fitting method. The coefficient of swelling c_s was also computed and tabulated in Table II.



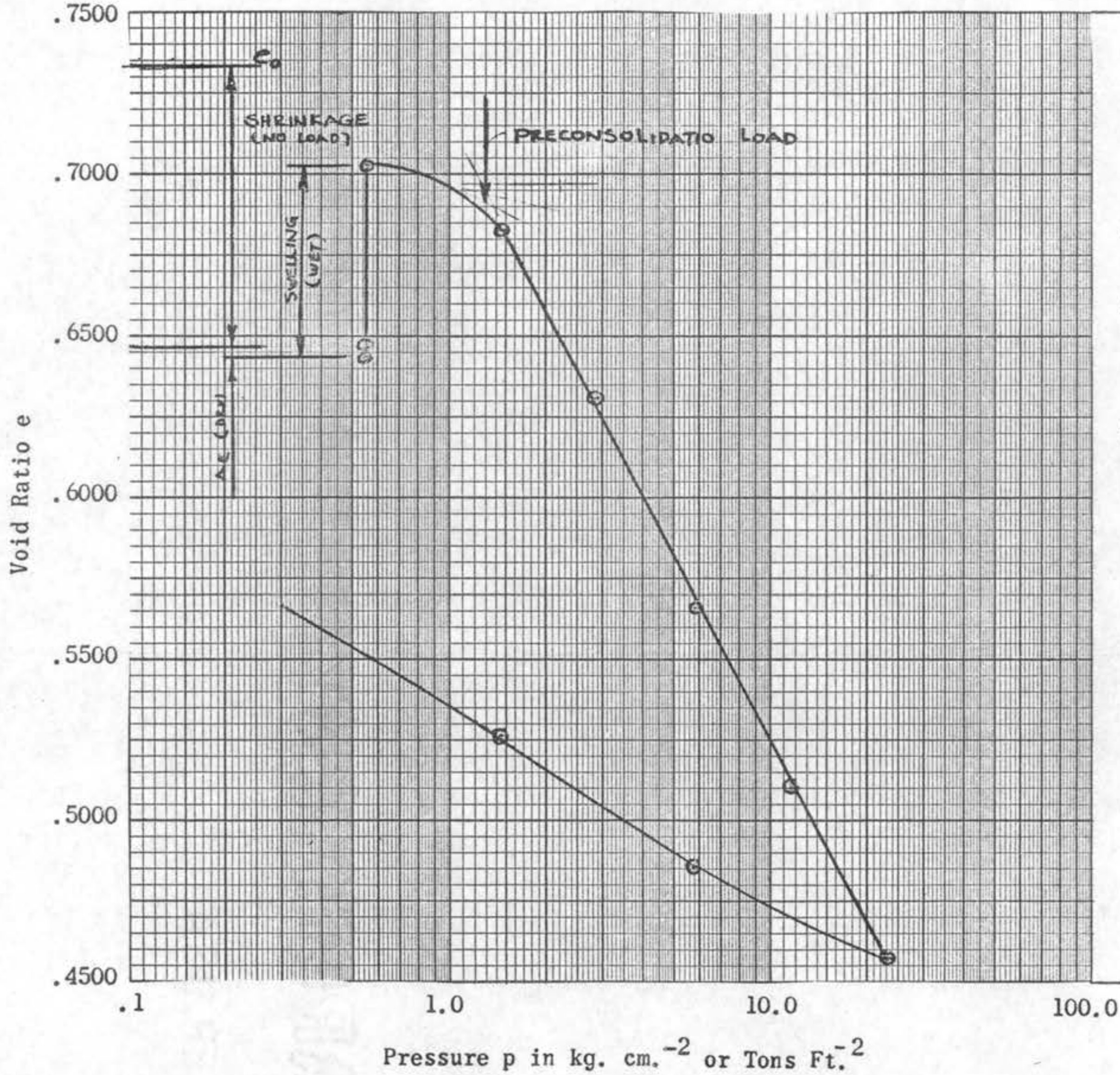
Machine No. 2

CONSOLITATION TEST
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-1
Area = 32.7 cm.^{2}
Height = 1.27 cm.
t_{50} (192) = 22 min.
a_v = .272 x 10^{-4}
k_{192} = .11 x 10^{-9} cm. sec.$^{-1}$
c_c = .268
c_s = .042

Fig. No. 14



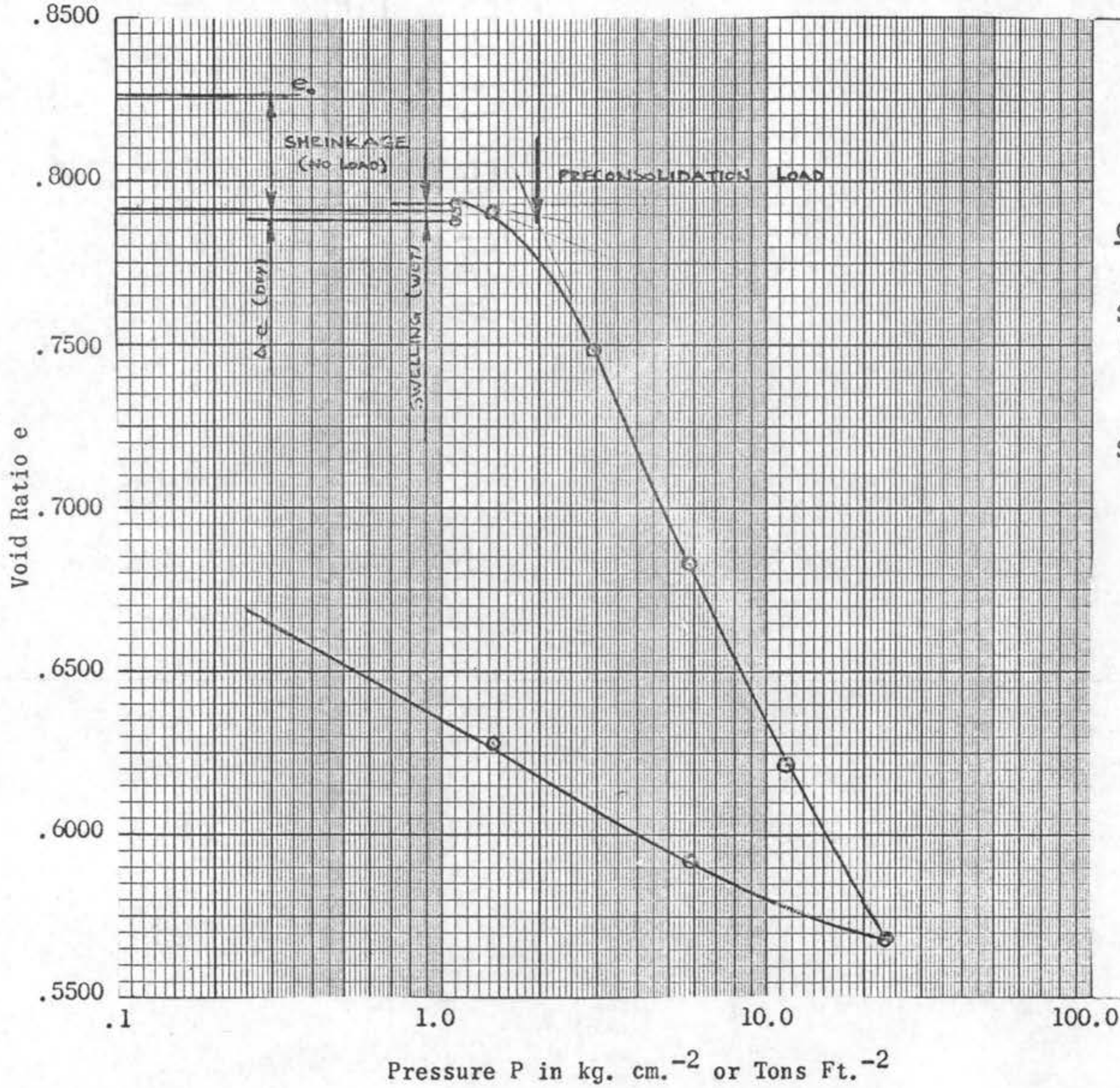
Machine No. 4

<u>CONSOLIDATION TEST</u>
e-p curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-2
Area = 32.7 cm.^{2}
Height = 1.28 cm.
t_{50} (384) = 42 min.
a_v = .953 x 10^{-5}
k_{384} = .202 x 10^{-9} cm. sec.$^{-1}$
c_c = .1895
c_s = .098

Fig. No. 15



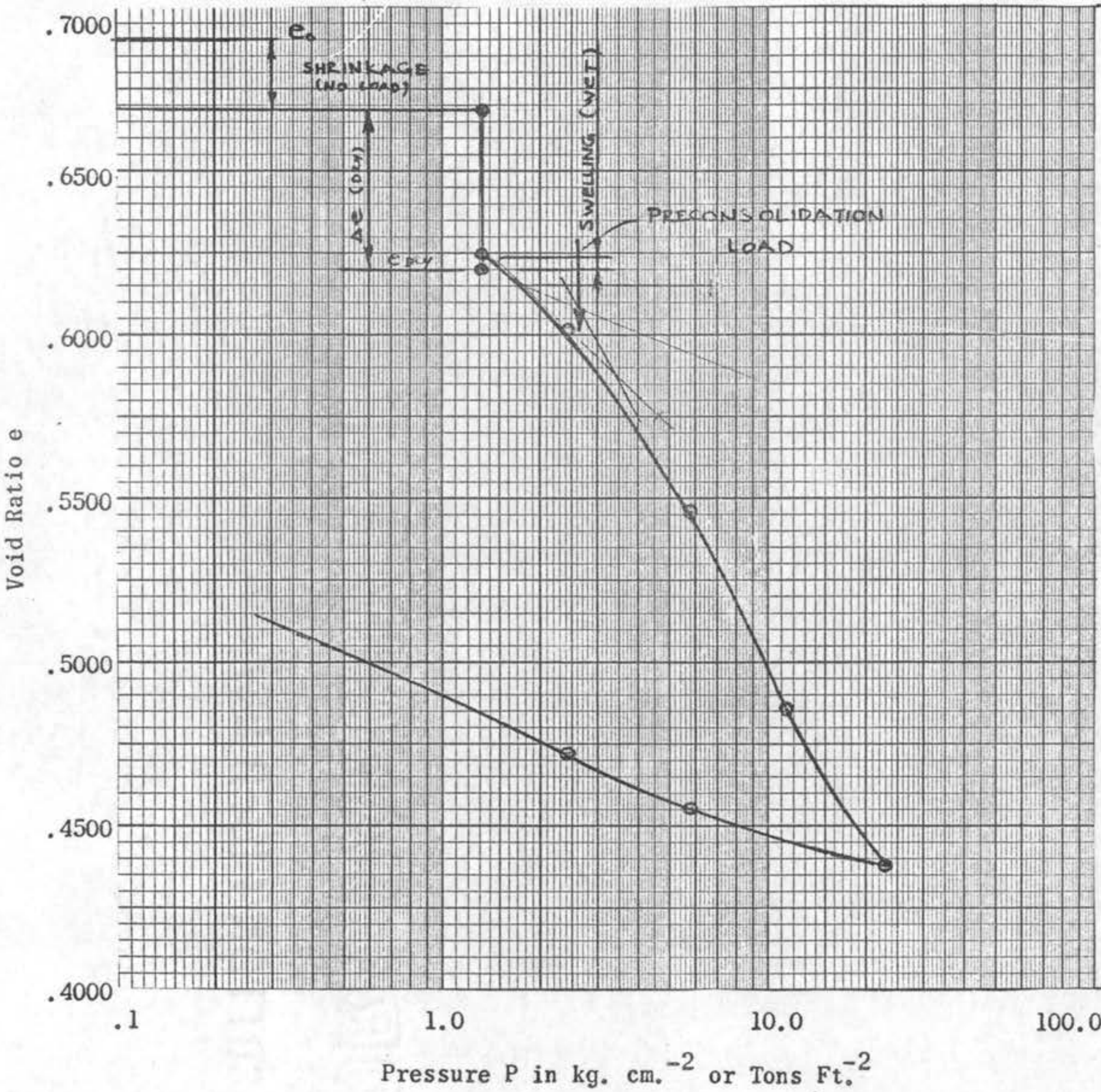
Machine No. 6

CONSOLIDATION TEST
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-3
Area = 32.7 cm.^{2}
Height = 2.1 cm.
t_{50} (384) = 62 min.
a_v = .105 x 10^{-4}
k_{192} = .376 x 10^{-9} cm. sec.$^{-1}$
c_c = .190
c_s = .032

Fig. No. 16



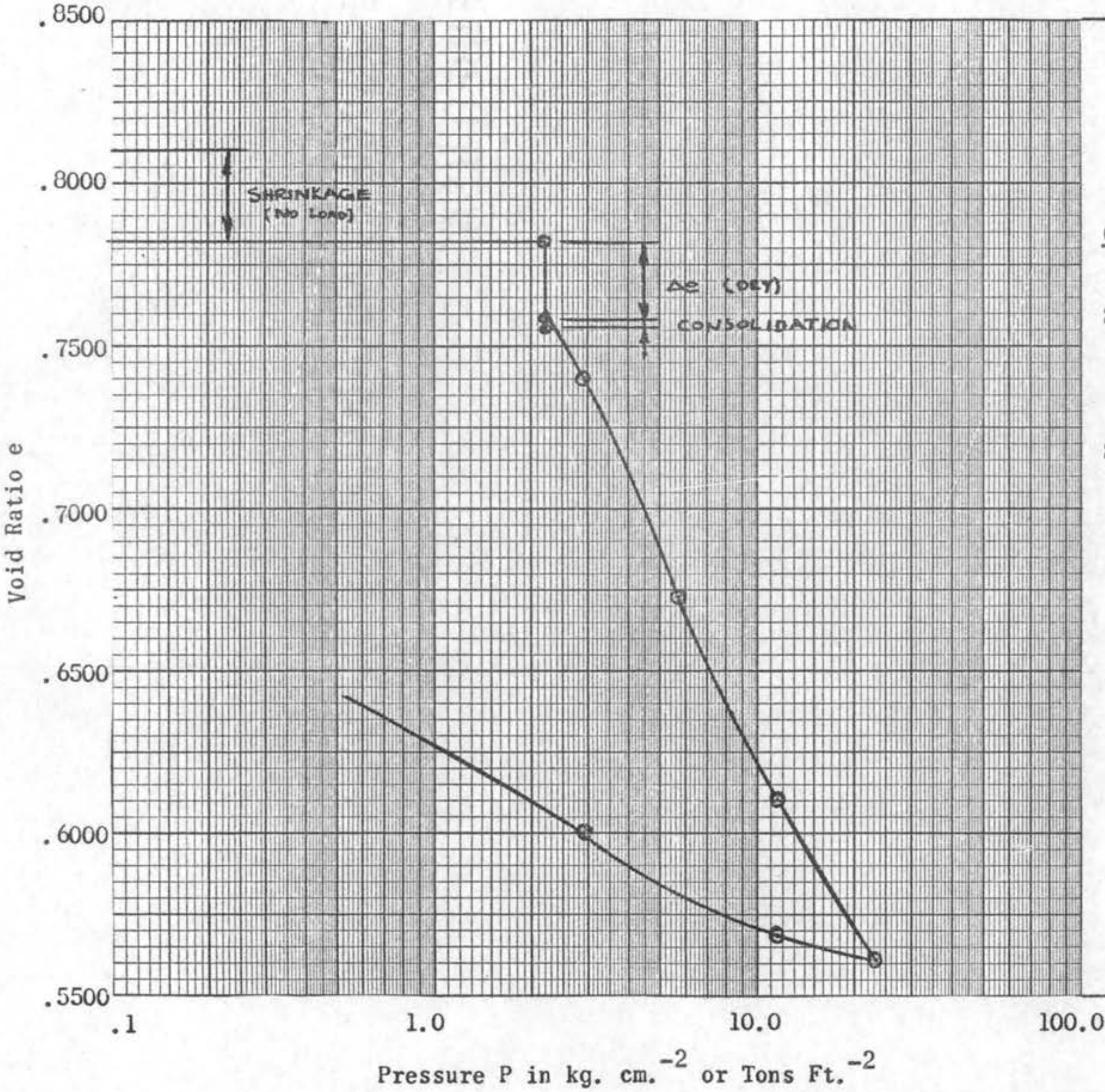
Machine No. 8

CONSOLIDATION TEST
e–p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-4
Area = 32.7 cm.^{2}
Height = 2.55 cm.
t_{50}(334 = 40 min.
a_v = .101 x 10^{-5}
k_{334} = .905 x 10^{-9} cm. sec.$^{-1}$
c_c = .157
c_s = .013

Fig. No. 17



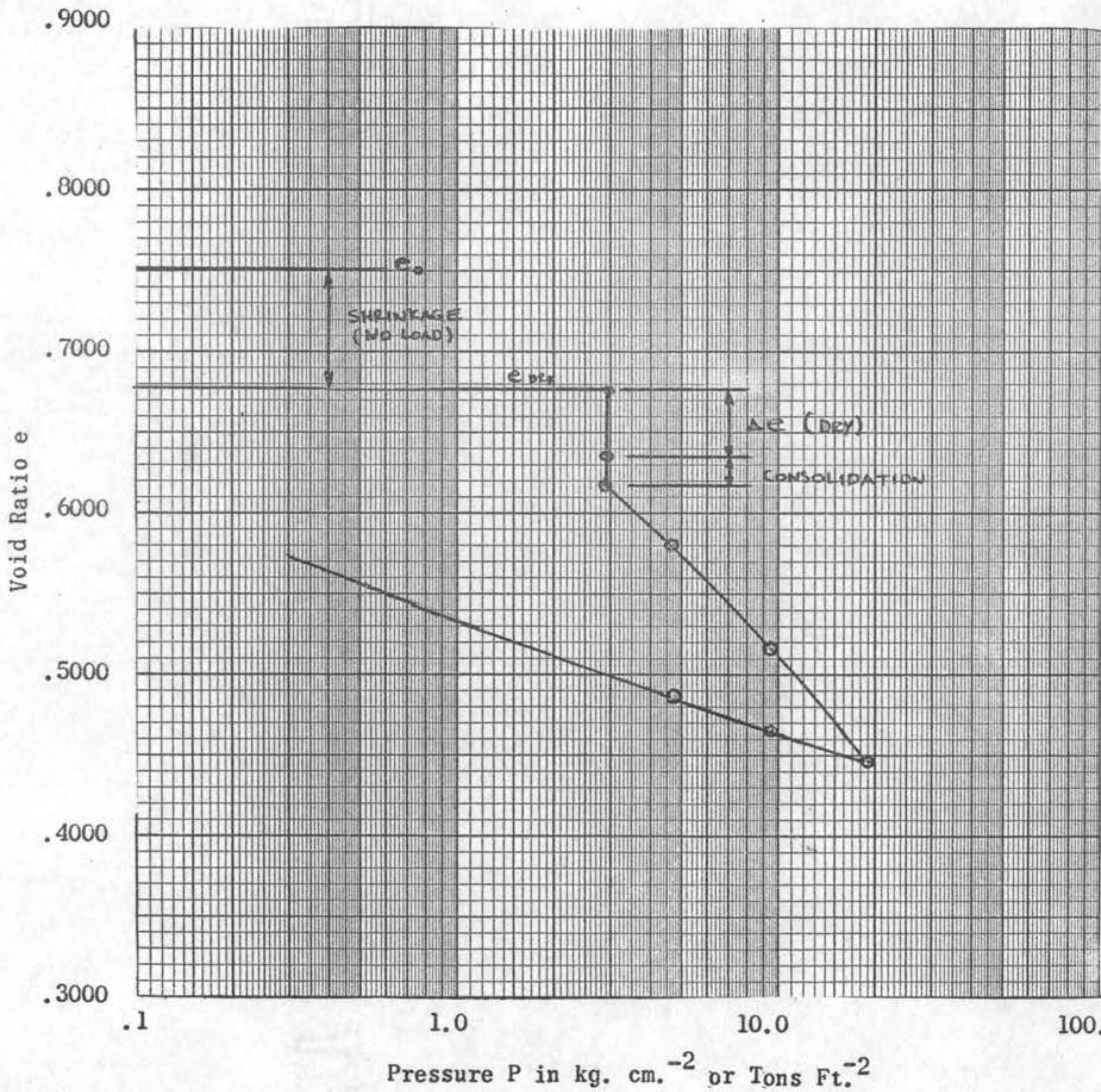
Machine No. 1

<u>CONSOLIDATION TEST</u>
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-5
Area = 32.7 cm.^{2}
Height = 2.55 cm.
t_{50}(384) = 70 min.
a_v = .102 x 10^{-4}
k_{384} = .47 x 10^{-9} cm. sec.$^{-1}$
c_c = .253
c_s = .037

Figure No. 18



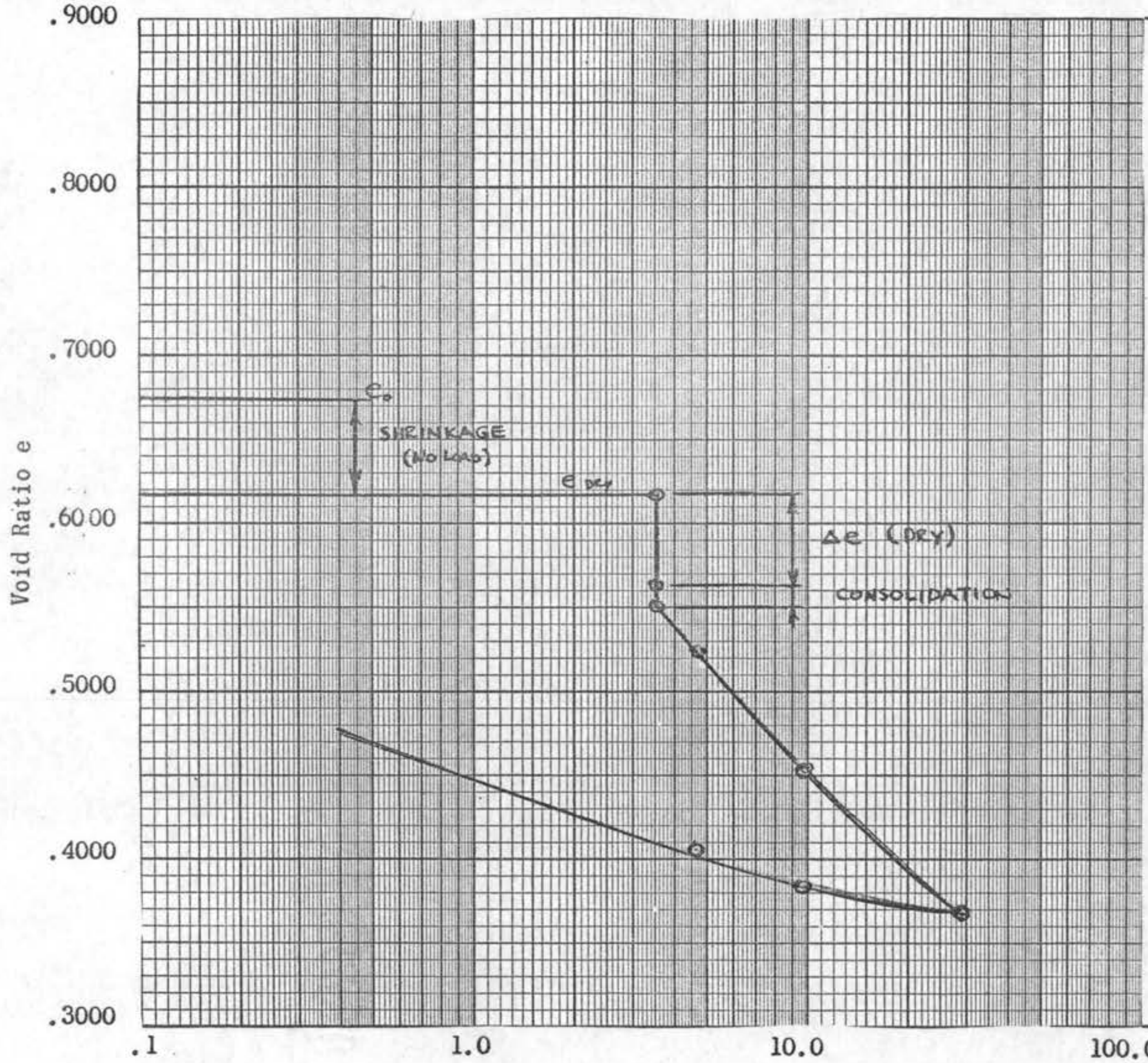
Machine No. 3

CONSOLIDATION TEST
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-6
Area = 80.5 cm.^{2}
Height = 1.9 cm.
t_{50}(384) = 4.6 min.
a_v = .22 x 10^{-4}
k_{384} = .9 x 10^{-9} cm. sec.$^{-1}$
c_c = .212
c_s = ,042

Fig. No. 19



Pressure p in kg. cm.$^{-2}$ or Tons Ft.$^{-2}$

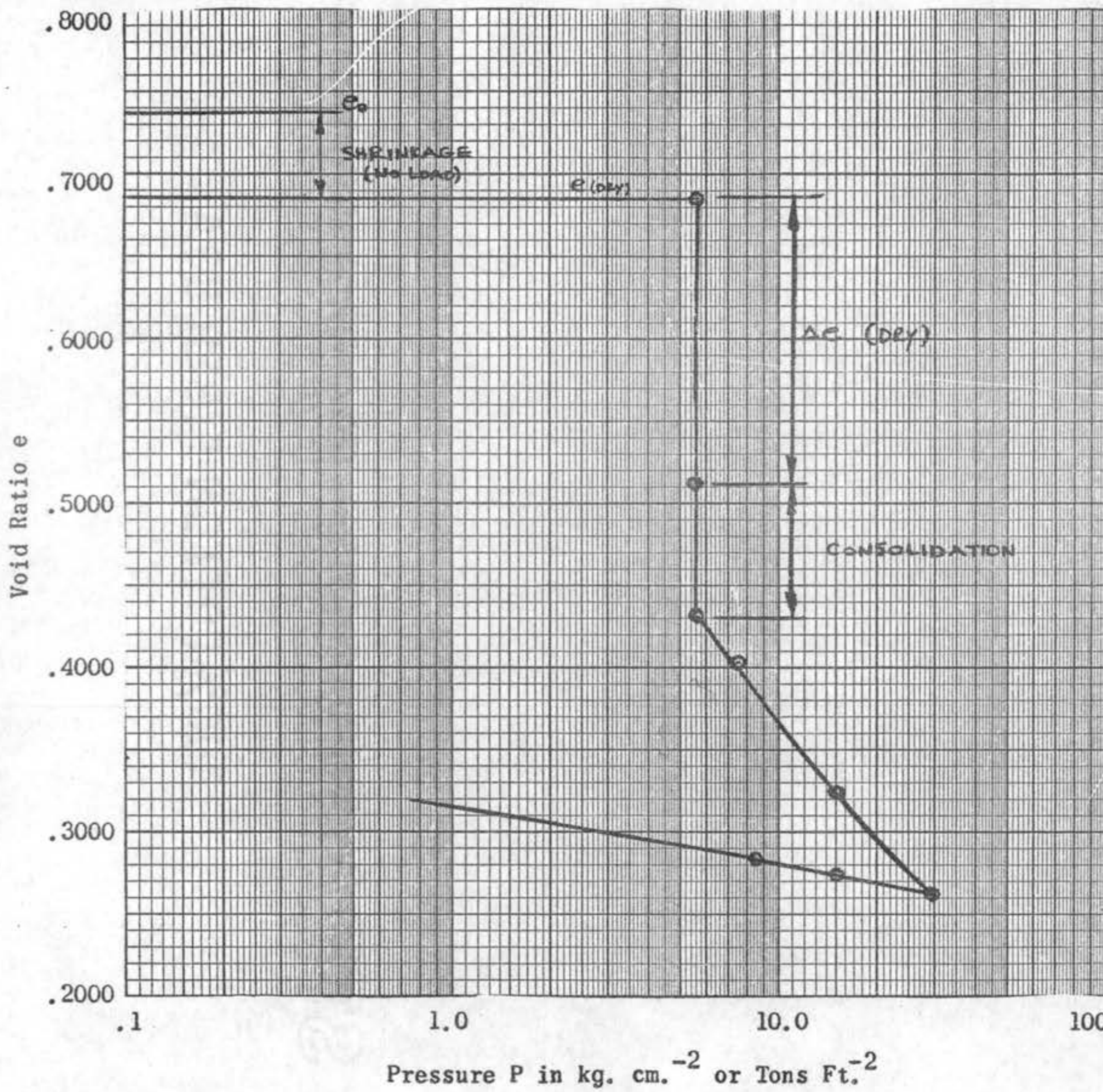
Machine No. 5

CONSOLIDATION TEST
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.

Specimen:
Sample No. B-7
Area = 80.5 cm.^{2}
Height = 1.91 cm.
t_{50}(768) = 7.8 min.
a_v = .157 x 10^{-5}
k_{768} = .348 x 10^{-9} cm. sec.$^{-1}$
c_c = .208
c_s = .053

Fig. No. 20



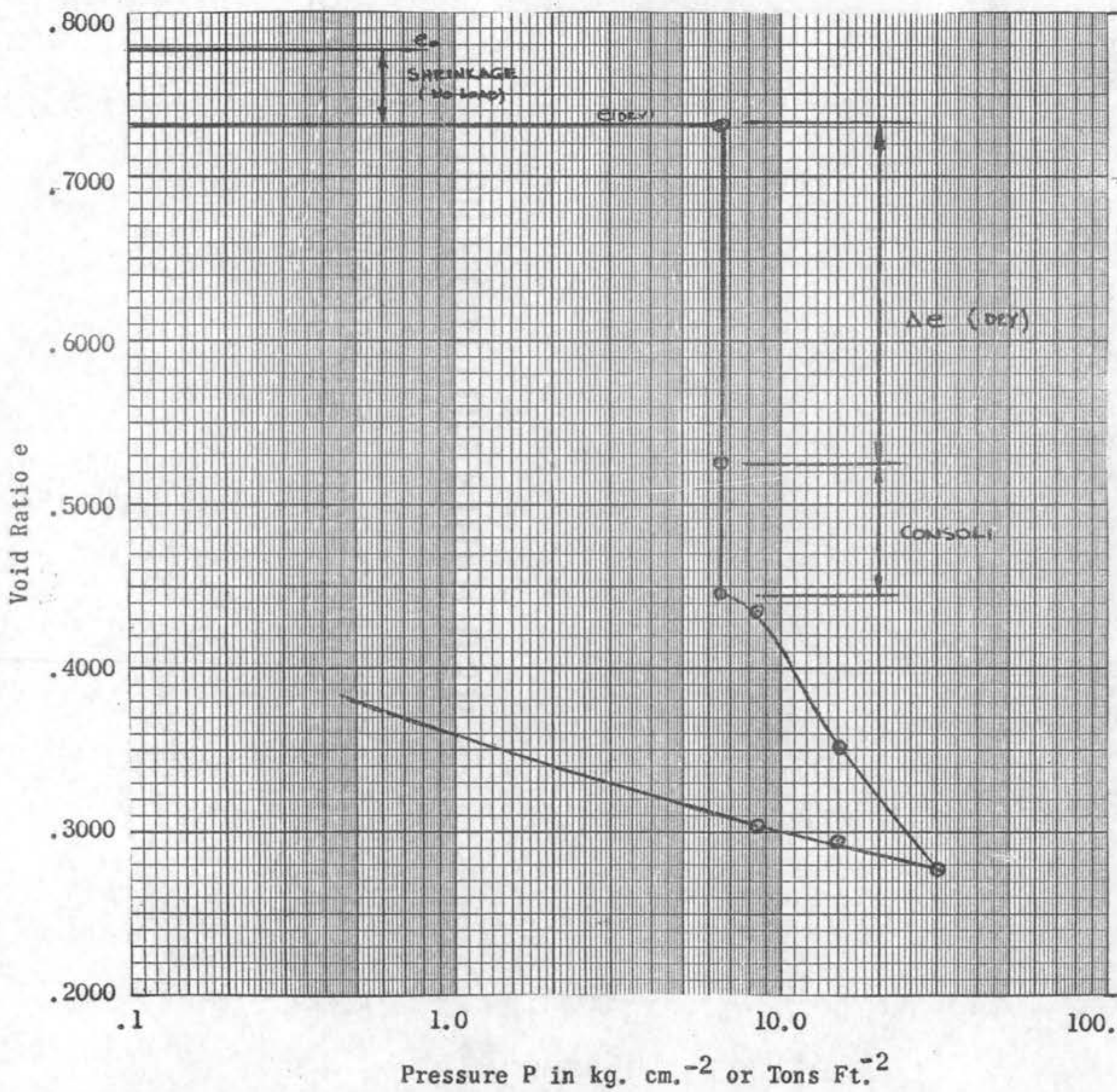
Machine No. 7

<u>CONSOLIDATION TEST</u>
e-p curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-8
Area = 12.78 cm.^{2}
Height = 1.58 cm.
t_{50}(192) = 3.6 min.
$a_v = .106 \times 10^{-4}$
$k_{192} = 45 \times 10^{-9}$ cm. sec.$^{-1}$
$c_c = .236$
$c_s = .020$

Fig. N. 21



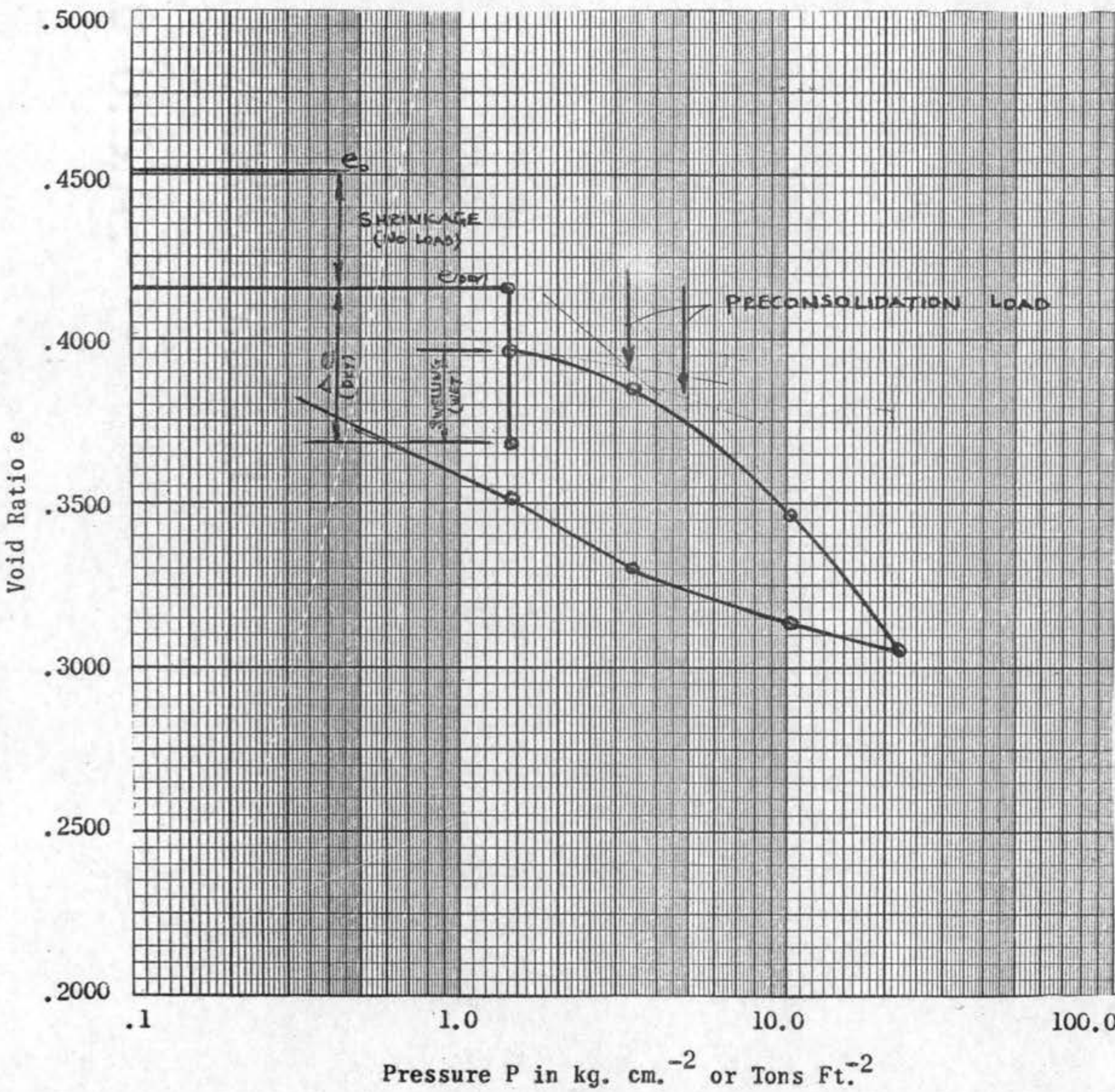
Machine No. 9

CONSOLIDATION TEST
e-p log curve

Sample:
Reddish brown sandy jointed clay
C.E. Testing Lab. Found.
(Stillwater, Okla.)

Specimen:
Sample No. B-9
Area = 12.78 cm.^{2}
Height = 1.6 cm.
t_{50} (192) = 3.6 min.
a_v = .113 x 10^{-4}
k_{192} = .48 x 10^{-8} cm. sec.$^{-1}$
c_c = .282
c_s = .048

Fig. No. 22



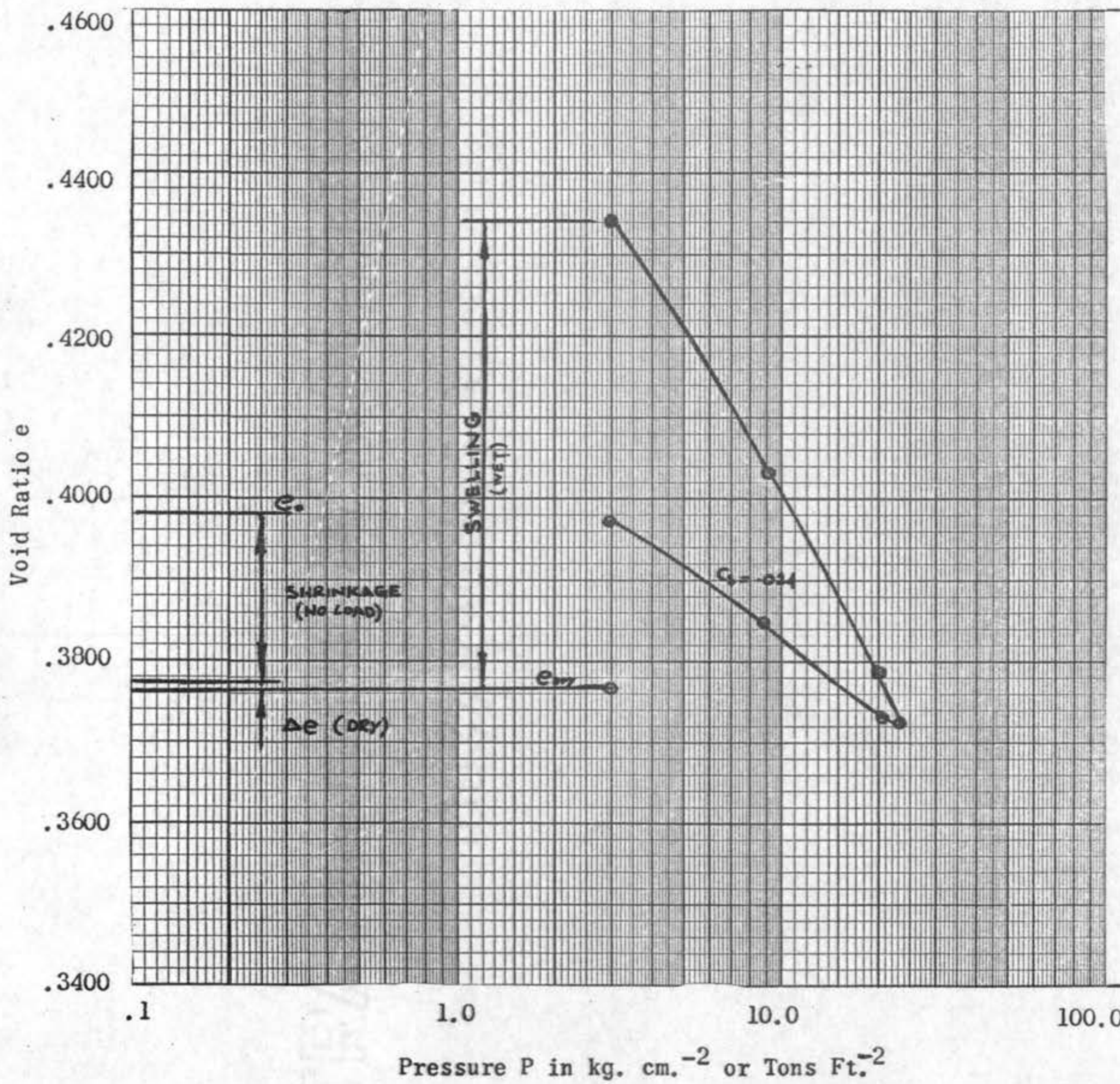
Machine No. 2

<u>CONSOLIDATION TEST</u>
e-p log curve

Sample:
Dark red sandy jointed clay
New Tele. Bldg. Found.
(Stillwater, Okla.)

Specimen:
Sample No. A-1
Area = 32.7 cm.^{2}
Height = 2.55 cm.
t_{50}(336) = 7 min.
a_v = .554 x 10^{-5}
k_{336} = .312 x 10^{-8}cm. sec.$^{-1}$
c_c = .082
c_s = .078

Fig. No. 23



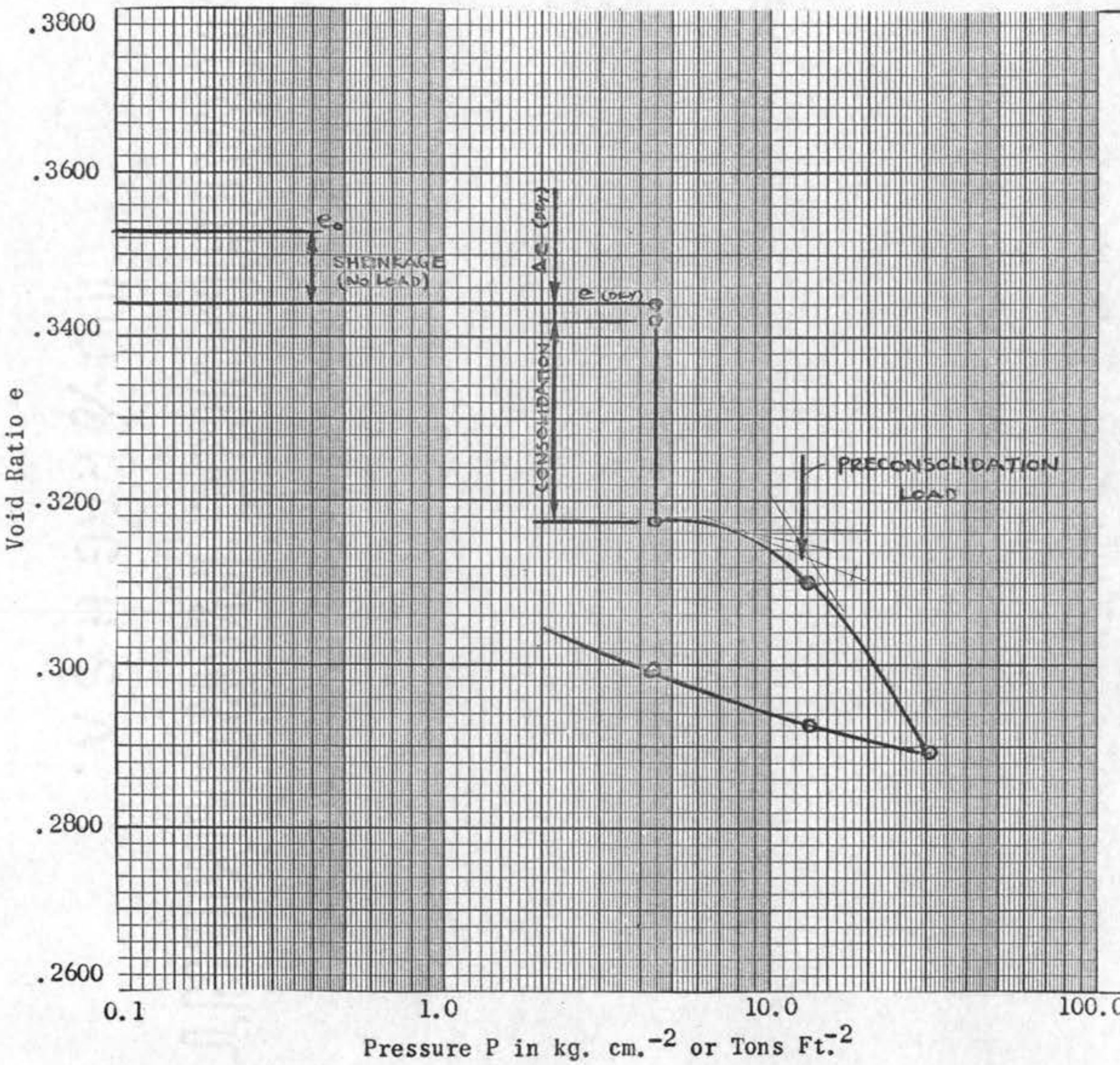
Machine No. 4

CONSOLIDATION TEST
e-p log curve

Sample:
Dark red sandy jointed clay
New. Tele. Bldg. Found.
(Stillwater, Okla.)

Specimen:
Sample No. A-2
Area = 32.7 cm.^{2}
Height = 2.53 cm.
t_{50} (672) = 12 min.
$a_v = .208 \times 10^{-5}$
$k_{672} = .665 \times 10^{-9}$ cm. sec.$^{-1}$
$c_c = .068$
$c_s = .024$

Fig. No. 24



Machine No. 6

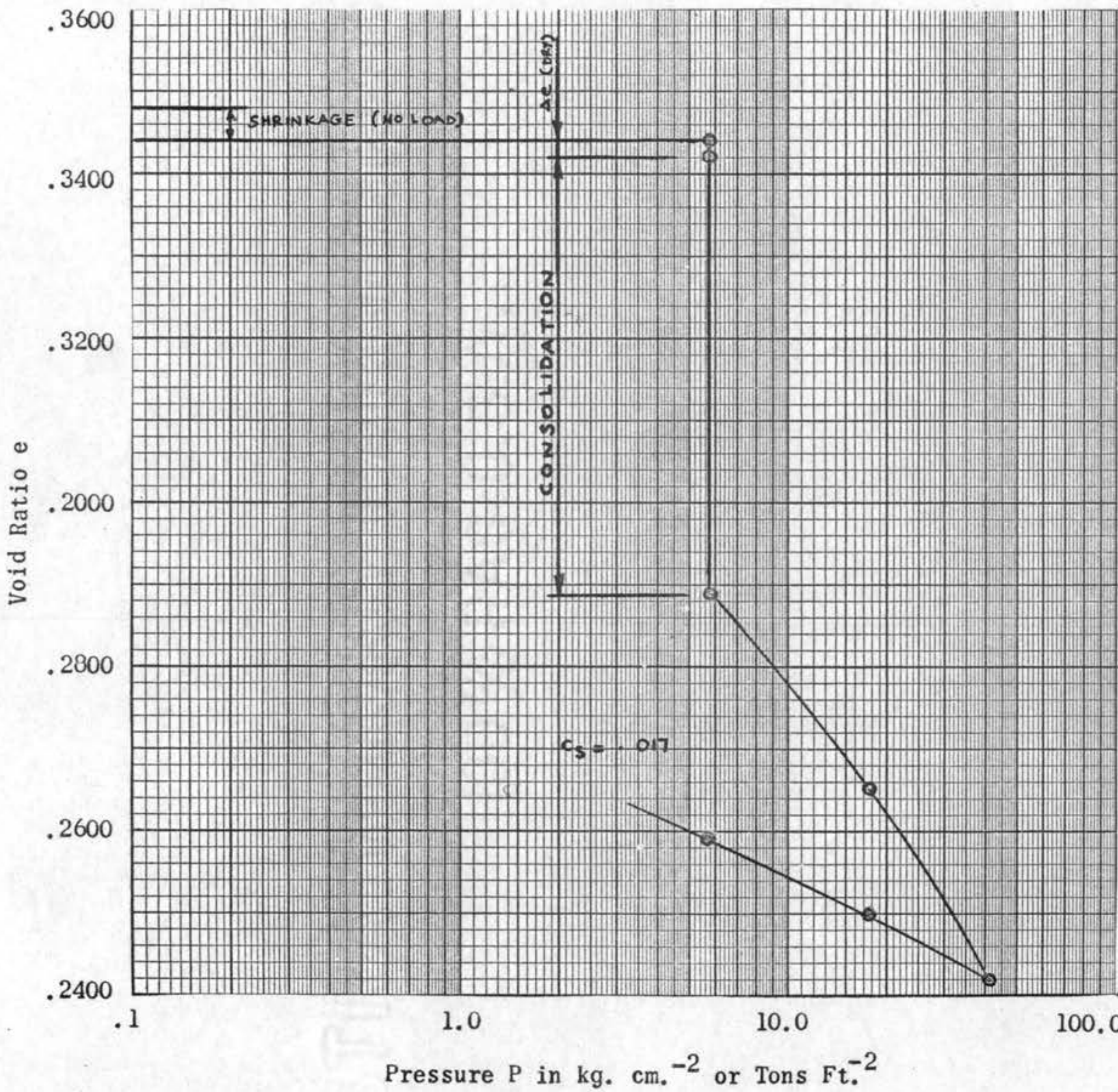
<u>Consolidation Test</u>
e-p log curve

Sample:
Dark red sandy jointed clay
New Tele. Bldg. Found.
(Stillwater, Okla.)

Specimen:
Sample No. A-3
Area = 32.7 cm^2
Height = 2.0 cm.
t_{50} (432) = 9 min.

$a_v = \frac{e}{p} - .33 \times 10^{-6}$
$k_{432} = .324 \times 10^{-9}$ cm. $sec.^{-1}$
$c_c = .046$
$c_s = .005$

Fig. No. 25



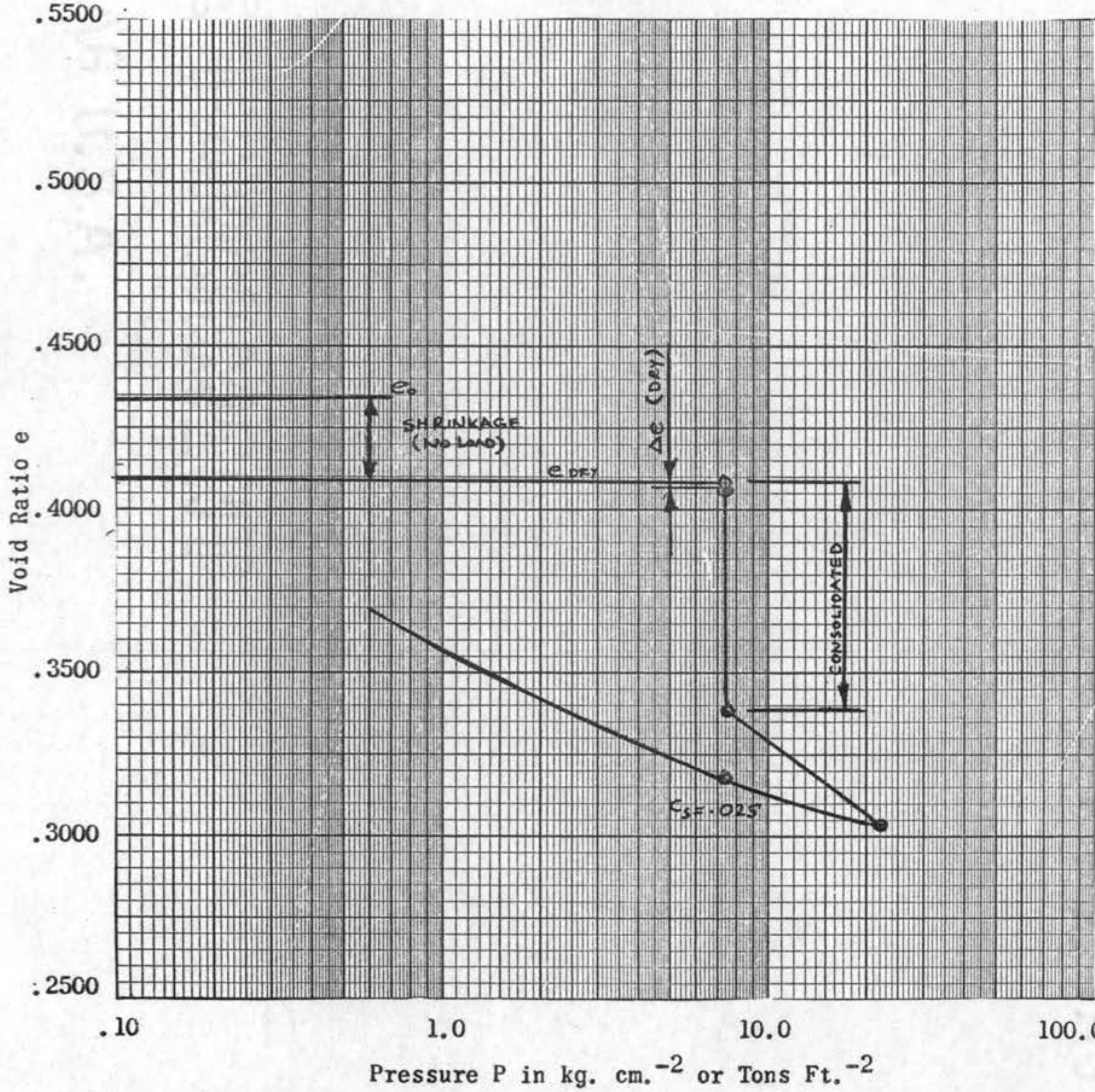
Machine No. 8

CONSOLIDATION TEST
e-p log curve

Sample:
Dark red sandy jointed clay
New Tele. Bldg. Found.
(Stillwater, Okla.)

Specimen:
Sample No. A-4
Area = 32.7 cm.^{2}
Height = 1.30 cm.
t_{50} (576) = 6 min.
A_v = .193 x 10^{-5}
k_{576} = .353 x 10^{-9} cm. sec.$^{-1}$
c_c = .053

Fig. No. 26



Machine No. 6

<u>CONSOLIDATION TEST</u>
e-p log curve

Sample:
Dark red sandy jointed clay
New Tele. Bldg. Found.
(Stillwater, Okla.)

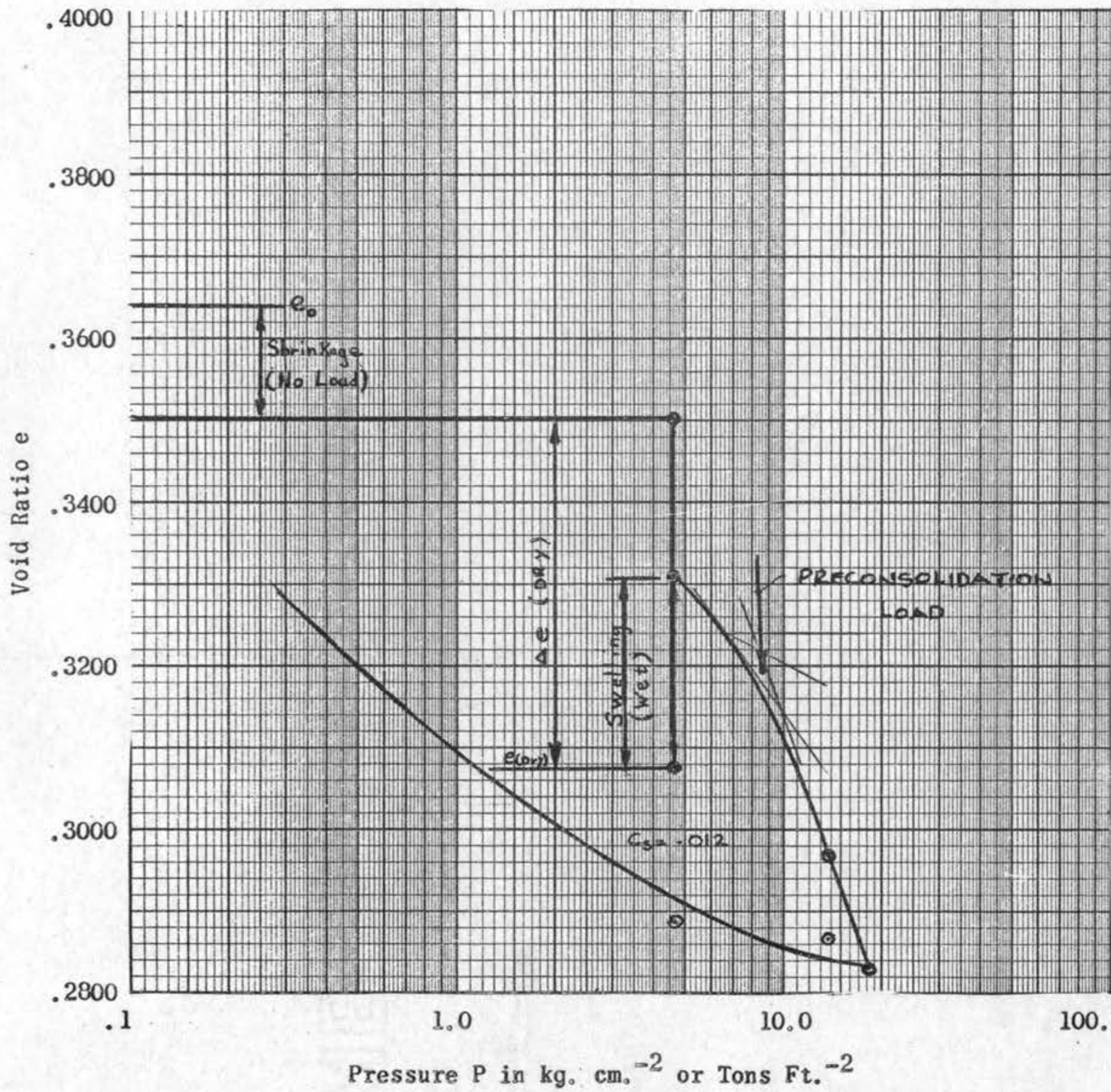
Specimen:
Sample No. A-5
Area = 32.7 cm.^{2}
Height = 1.27 cm.
$t_{50\,(720)}$ = 4 min.

$a_v = \frac{e}{p} = .23 \times 10^{-5}$

$k_{720} = .445 \times 10^{-9}$ cm. sec.$^{-1}$

$c_c = .072$

Fig. No. 27



Machine No. 7

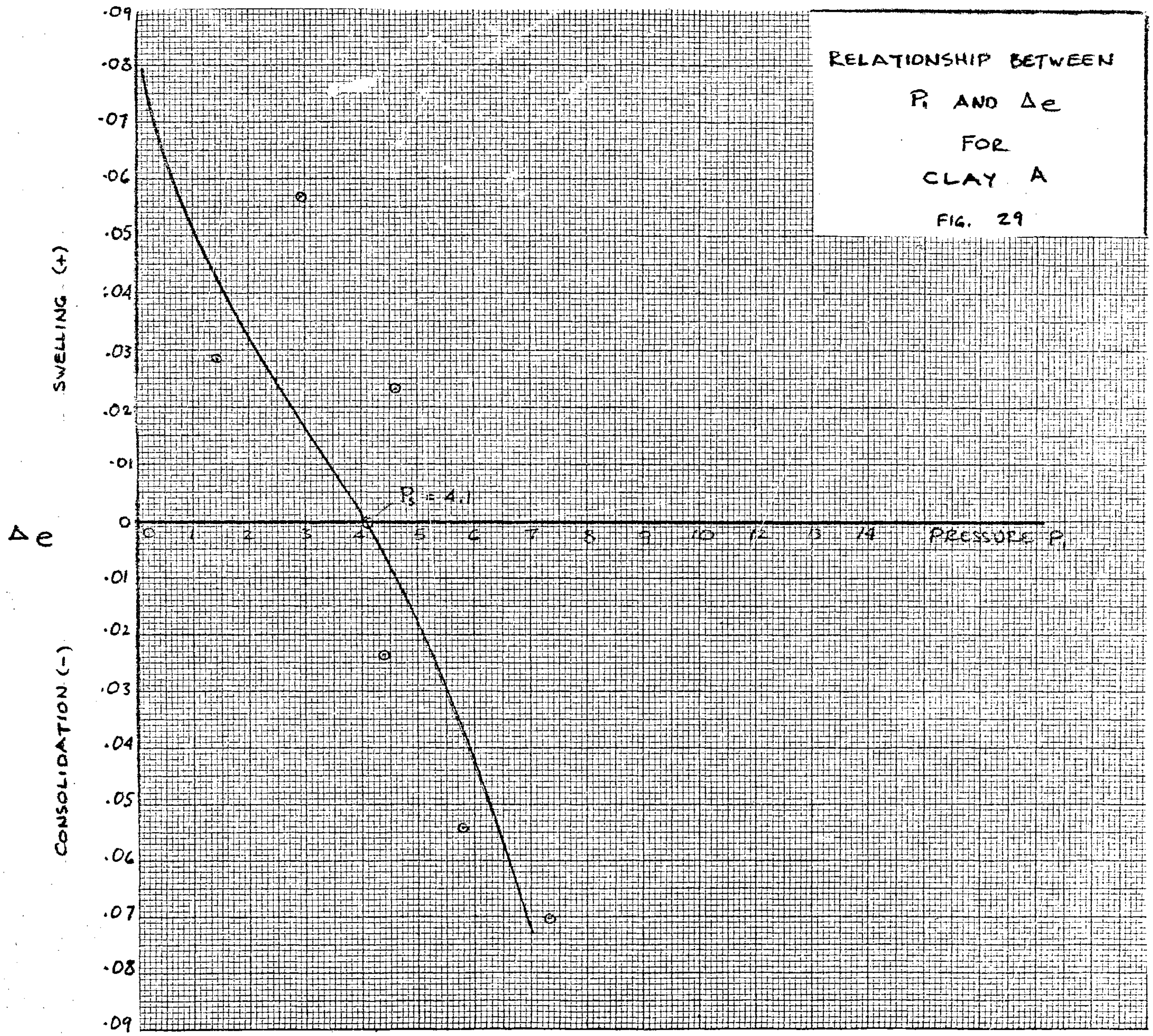
CONSOLIDATION TEST
e-p log curve

Sample:
Dark red sandy jointed clay
New Tele. Bldg. Found.
(Stillwater, Okla.)

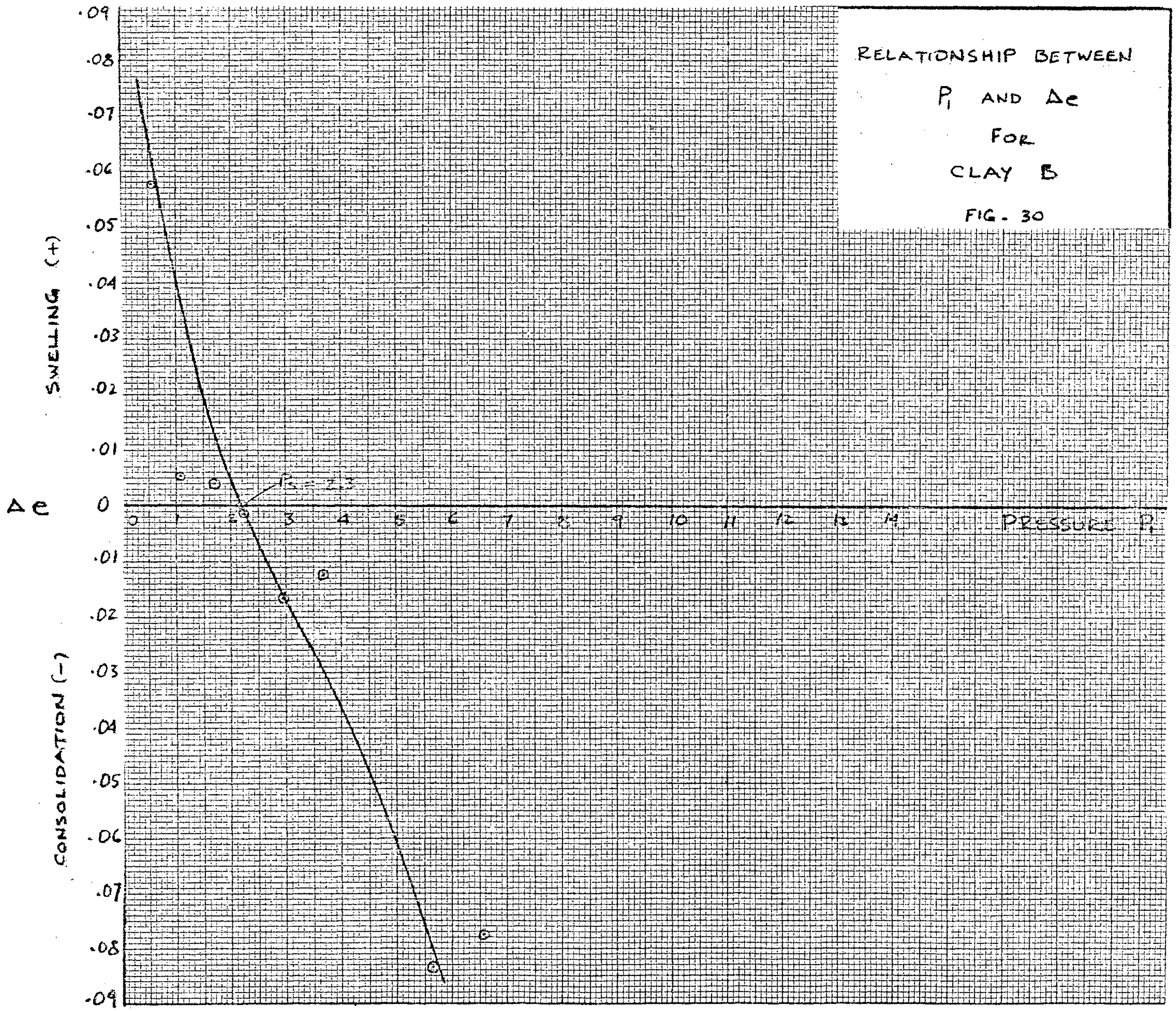
Specimen:
Sample No. A-6
Area = 80.5 cm.^{2}
Height = 1.9 cm.
t_{50} = 8 min

$a_v = .374 \times 10^{-5}$
$k_{(1116)} = 0.32 \times 10^{-9}$ cm. sec.$^{-1}$
$c_c = .083$

Fig. No. 28



RELATIONSHIP BETWEEN
P_1 AND Δe
FOR
CLAY A
FIG. 29



RELATIONSHIP BETWEEN
P_1 AND Δe
FOR
CLAY B
SWELLING (+)
CONSOLIDATION (−)
Δe
PRESSURE P_1
$P_s = 2.2$
.09
.08
.07
.06
.05
.04
.03
.02
.01
0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIG. 30

TABLE I

RESULTS FOR PRESSURE AND SWELLING RELATIONSHIP

Test No.	e_{dry}	e_F	Δe	P_1	P_s (1)	P_s (2)	Determined c_c	Computed c_c
A - 1	0.3686	0.3974	+0.0288	1.47	6.0	4.8	0.082	0.0815
A - 2	0.3767	0.4340	+0.0573	2.94	21.0	N.D.	0.068	0.063
A - 3	0.3413	0.3176	-0.0237	4.41	N.D.	12.5	0.046	0.037
A - 4	0.3426	0.2882	-0.0544	5.84	N.D.	N.D.	0.053	0.0396
A - 5	0.4086	0.3385	-0.0701	7.35	N.D.	N.D.	0.072	0.0722
A - 6	0.3078	0.3310	+0.0232	4.62	11.0	8.5	0.083	0.039
B - 1	0.7104	0.8033	+0.0929	0	0.74	0.5	0.268	0.238
B - 2	0.6449	0.7038	+0.0579	0.55	2.40	1.3	0.1895	0.232
B - 3	0.7885	0.7939	+0.0054	1.10	1.50	2	0.190	0.184
B - 4	0.6200	0.6246	+0.0046	1.65	1.50	2.6	0.157	0.210
B - 5	0.7589	0.7576	-0.0013	2.2	2.2	N.D.	0.253	0.274
B - 6	0.6360	0.6194	-0.0166	2.98	2.9	N.D.	0.212	0.243
B - 7	0.5631	0.5509	-0.0122	3.58	N.D.	N.D.	0.208	0.202
B - 8	0.5144	0.4300	-0.0844	5.64	N.D.	N.D.	0.236	0.232
B - 9	0.5239	0.4463	-0.0776	6.58	N.D.	N.D.	0.282	0.258
C - 1	0.5335	0.7290	+0.1955	0.093	5.0	N.D.	0.231	0.232
C - 2	0.5330	0.7118	+0.1788	0.184	12.0	N.D.	0.204	0.221
C - 3	0.5000	0.6867	+0.1367	0.373	7.2	N.D.	0.210	0.208
C - 4	0.6180	0.7902	+0.1722	0.741	9.2	N.D.	0.266	0.265
C - 5	0.5530	0.6544	+0.1014	1.470	N.D.	N.D.	N.D.	N.D.
C - 6	0.5560	0.5925	+0.0365	3.820	5.2	N.D.	N.D.	N.D.

P_1 is initial pressure in kg. cm.$^{-2}$ or tons Ft.$^{-2}$.

e_F is void ratio after swelling or consolidating under P_1.

P_s(1) is pressure at intersection of e_{dry} with e-log P curve.

P_s(2) is pressure which is found by Casagrande method obtained from e-log P curve.

c_c determined is slope of e-log P curve.

c_c computed is value from $c_c = .54\ (e_n - .3)$.

TABLE II

RESULTS OF THE CONSOLIDATION TEST

Sample No.	Natural Void-Ratio e_n	Coefficient of Compressibility a_v	Coefficient of Swelling c_s	Compression Index c_c	Coefficient of Consolidation c_v	Coefficient of Permeability k
A - 1	0.4510	0.554×10^{-5}	0.078	0.082	0.64×10^{-3}	0.312×10^{-8}
A - 2	0.3980	0.208×10^{-5}	0.024	0.068	0.44×10^{-3}	0.665×10^{-9}
A - 3	0.3510	0.083×10^{-5}	0.005	0.046	0.37×10^{-3}	0.324×10^{-9}
A - 4	0.3480	0.193×10^{-5}	0.017	0.053	0.23×10^{-3}	0.353×10^{-9}
A - 5	0.4340	0.230×10^{-5}	0.025	0.072	0.33×10^{-3}	0.445×10^{-9}
A - 6	0.3640	0.374×10^{-5}	0.012	0.083	0.56×10^{-3}	0.840×10^{-9}
B - 1	0.7400	0.272×10^{-4}	0.042	0.268	0.60×10^{-4}	0.110×10^{-9}
B - 2	0.7340	0.953×10^{-5}	0.098	0.1895	0.32×10^{-4}	0.202×10^{-9}
B - 3	0.8260	0.105×10^{-4}	0.033	0.190	0.59×10^{-4}	0.376×10^{-9}
B - 4	0.6900	0.101×10^{-5}	0.013	0.157	0.13×10^{-3}	0.905×10^{-9}
B - 5	0.8100	0.102×10^{-4}	0.037	0.253	0.76×10^{-4}	0.47×10^{-9}
B - 6	0.7510	0.220×10^{-4}	0.042	0.212	0.64×10^{-3}	0.90×10^{-9}
B - 7	0.6740	0.157×10^{-5}	0.053	0.208	0.74×10^{-3}	0.348×10^{-9}
B - 8	0.7360	0.106×10^{-4}	0.020	0.236	0.57×10^{-3}	0.480×10^{-9}
B - 9	0.7780	0.113×10^{-4}	0.048	0.282	0.58×10^{-3}	0.480×10^{-9}

CHAPTER VI

SUMMARY AND CONCLUSIONS

The experiment was conducted on samples of Permian clay from the vicinity of Stillwater, Oklahoma. The prime purpose of this investigation was to determine the pressure which would prevent swelling of the dry clay as it absorbs water and to determine the relationship between amount of swelling and pressure against which the swelling takes place.

Swelling and consolidation could be eliminated when the applied pressure on the soil is equal to the consolidation load due to desiccation. Because of the wide variations in the physical properties of the Permian clay, no claim is made that the results herein reported are representative of the Permian clay. The research performed requires more extensive research to follow.

The following results were noted:

1) Clay A would not swell against a load of 4½ to 5.0 tons per sq. foot, and clay B would not swell against pressures of 2 to 2½ tons per sq. foot, but both clays will lift lighter loads to a distance inversely proportional to the pressures.

2) The amount of swelling from the shrinkage limit under different loadings was measured for several samples and values were shown in Table I, but these values were not enough to determine a reliable relationship between pressure and swelling in order to determine the

preconsolidation load for this clay as shown in Figures 29 and 30. Also the disturbance of the sample during the test procedure might have an effect upon the results of this relationship. This explains why the curve, in Figures 29 and 30 did not pass through the corresponding points which were determined in Table I.

Summary

Successful buildings which will not be damaged by the action of the clay can be built economically in this region by obtaining good information about this soil from people with sufficient experience in this field. As Professor Means put it, "It is the exercise of common sense based on a general knowledge of properties of the clay and its action in this climate which is of greater importance than attempts to design for future laboratory determined values of the physical properties." This does not mean that laboratory determined values of the physical properties are useless, but that they should be used intelligently, keeping in mind that such values are determined for only a very small sample of the soil. Although they may be fairly accurate for this small sample, the over-all characteristics of the soil under footings may be quite different. Careful observations of existing structures in the neighborhood and intelligent interpretation is probably of greater importance than laboratory tests. It should be kept in mind that the damaging action of the clay is caused by climatic conditions. The same clay, with exactly the same physical properties, will react under a load entirely differently below the water table than above, where it is subjected to change in water content.

Future Investigations

For future investigations more research should be conducted in this field, because of the necessity of obtaining a better understanding of the physical properties of Permian clay deposits.

In determining a safe supporting pressure, the preconsolidation load should be known, and the method of intersection of e_{dry} with e-log P curve in determining the approximate maximum swelling pressure should be studied carefully, because the author feels that there was not enough information ahead of time in order to determine the reliability of this method. In future investigations one should start, with different loading procedures and with smaller initial loads, to obtain more information about the swelling property of this soil.

No special care was provided for preventing air bubbles. This should be taken into consideration in the future by flooding the sample half way, so that entrapped air bubbles will be released.

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APPENDIX

The following letter symbols were adopted for use in this thesis:

Symbol	Property	Dimension
A	Area	cm.^{2}
a_v	Coefficient of compressibility	cm^2 gm^{-1}
c_c	Compression index [(computed = (.5A) (e_n-.3)]	Dimensionless
c_s	Swelling index	Dimensionless
c_v	Coefficient of consolidation	cm^2 Sec^{-1}
e_n	Natural void ratio	Dimensionless
e_F	Void ratio after swelling under p_1	Dimensionless
Q	Specific gravity	Dimensionless
H	Height, or thickness	cm.
k	Coefficient of permeability	cm. sec^{-1}
P	Total force or load, pressure	kg. cm.$^{-2}$
s	Degree of saturation	Dimensionless
T	a) Temperature	Degrees
	b) Time factor	Dimensionless
t	Time	Sec.
W	Weight	gm
w	Water content	Dimensionless
γ	Unit weight	gm. cm.$^{-3}$
Δ	Displacement or deformation	cm.

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