SUGGESTED NEW PROCEDURE FOR SOIL CONSOLIDATION TEST

by

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ABSTRACT

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The Standard Soil Consolidation Test (ASTM D 2435-70), adopted by most laboratories today entails some disadvantages and inconveniences. These include an increase in the time needed for the sample preparation, and most importantly, the unavoidable specimen disturbances caused by cutting, trimming and fitting. These greatly influence the results obtained.

In an attempt to accomplish the test more accurately and effectively, Dr. John N. Cernica developed a new method for the consolidation test. Using this method, instead of extracting and trimming, the specimen is tested within the shelby tube with the tube acting as a floating ring consolidometer. This makes the test more expedient and, as indicated later, more accurate.

The contents of this thesis were based upon the results obtained from eighteen sets of comparative tests. Test data were recorded, void ratio - pressure relationships (e - log p) were plotted and the comparison of the results from the two methods were also discussed. A conclusion was subsequently obtained and presented in the final chapter.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

CHAPTER I

INTRODUCTION

Virtually all stationary structures rest upon soil through some type of foundations. Also, every material undergoes a certain amount of strain when subjected to stress; so does soil. Unlike a steel rod or to a lesser extent a column, when a soil is loaded or compacted, the strains are not proportional to applied stress increments. Furthermore, the strain may not be necessarily reduced when the load or a portion of the load is removed.

When a clayey (fine-grained), saturated soil is subjected to compressive load, the volume change in the soil is primarily due to the expulsion of water from the voids. According to Terzaghi: "Every process involving a decrease in the water content of a saturated soil without replacement of the water by air is called a process of consolidation. The opposite process is called a process of swelling, which involves an increase in the water content due to an increase in the volume of the voids". "The change in the volume of voids and the volume of water in the voids is brought about either by the weight of the soil itself, or from static loads. Thus, consolidation is the process of adjustment of the volume of the soil pore water (volume change) under an applied load on soil."[1]

Generally speaking, the entire consolidation process can be divided into two distinct parts: primary and secondary. During this earlier stage after the external load has been applied, air and pore water are stressed and gradually expelled from the voids of the soil. This phenomena is referred to as primary consolidation. With time and under sustained load, plastic deformation and other surface phenomena then become active. This is generally referred to as secondary consolidation. These processes are very slow and rather hypothetical in nature; at present time there is still no accurate method available to deal with secondary consolidation.

Of the two types, the primary consolidation is by far the more significant of the two, accounting perhaps for 95% or more of the total consolidation. In practice, only the primary consolidation is usually dealt with. Since the primary consolidation of a soil mass is the result of a decrease in the volume of the void spaces in the soil, it is convenient to express the stress-strain relationship in terms of void ratio and unit pressure instead of unit stress and unit strain used in the case of most other engineering materials.

By means of a device termed the consolidometer, the consolidation test is actually a compression test on a laterally-confined soil sample. The main purpose of this test is to obtain information on the compressive properties of saturated soils, especially clayey, and subsequently in determining the magnitude and rate of settlement of structures.

Physical and mechanical properties such as its structure, density, porosity, water content and the stress condition in the soil will influence the accuracy of the result of the consolidation test. Thus, one should attempt as much as possible to maintain the soil sample under an undisturbed condition. However, the term "undisturbed" is only a relative one. This is because it is almost impossible to obtain a soil sample, to transport it, to handle and prepare it for tests in the course of laboratory investigation without disturbing it to some degree.

For the standard test method which was adopted by most of the soil laboratories (ASTM D 2435-70), the soil sample is ejected from the sampler (e.g. shelby tube). Then the sample is trimmed to the inside diameter of the consolidometer ring, placed in the ring and trimmed flush with the plane surface of the ring. Actually the removal of any sample from its original environment is certain to disturb the sample to some extent. Also during this preparing process, the stress condition of the sample is first disturbed and changed by ejecting the soil from the shelby tube. Then during the process of trimming, the sample is again disturbed. Therefore the state of a typical soil sample used for standard test method is far from that of the original undisturbed condition on which an accurate result is based.

In an attempt to eliminate those unnecessary disturbances during the sample handling, Dr. John N. Cernica developed an alternate method to maintain the sample as "undisturbed" as possible during the test. By this modified method, the shelby tube is cut to a desired length with an original soil sample in it. Instead of ejecting, cutting, trimming and fitting into the consolidometer, the new method uses the shelby tube itself as a floating ring consolidometer. This not only eliminates the disturbances of extracting, cutting and trimming but also reduces the time consumption of the preparation. Furthermore, since the sample does not have to be removed from the shelby tube, the new method prevents coarser non-cohesive soils from falling apart.

In the following chapters, one finds that void ratio-pressure curves obtained from the new method are very similar to those from the standard method. Actually each set of comparison curves are almost parallel to each other, with more than 90% of the curves from the new method falling above those from the standard method.

CHAPTER II

PERFORMANCE OF SOIL UNDER COMPRESSIVE LOAD AND THE CONSOLIDATION PROCESS

2-1. Settlement and consolidation

Settlement of a structure resting on soil may be caused by two distinct kinds of action within the foundation soil. In one case the load imposed may cause shearing stresses to develop within the soil mass which are greater than the shearing strength of the material. When this occurs, the soil fails by sliding downward and laterally, and the structure settles and perhaps tips out of vertical alignment.

In the other case, a structure settles by virtue of the compressive stress and the accompanying strain which are developed in the soil as a result of the load imposed upon it. This strain is a normal phenomenon and in no sense is to be thought of as a failure of the soil, although it may cause the structure to fail if the settlements are excessive and nonuniform. This time-related decrease involving compression, stress transfer and drainage is called consolidation.

2-2. Causes of consolidation

Some of the main factors contributing to consolidation of soils are:

(1) static loads on soil;

(2) dynamic forces from vibrations excited by machinery, traffic, pile driving operations, explosions, earthquake, and various impacts on soil due to collapse of structures and earthworks. These factors loosen the structural strength of the soil, particularly the strength of non-cohesive soils;

(3) fluctuations in the elevation of the groundwater table;

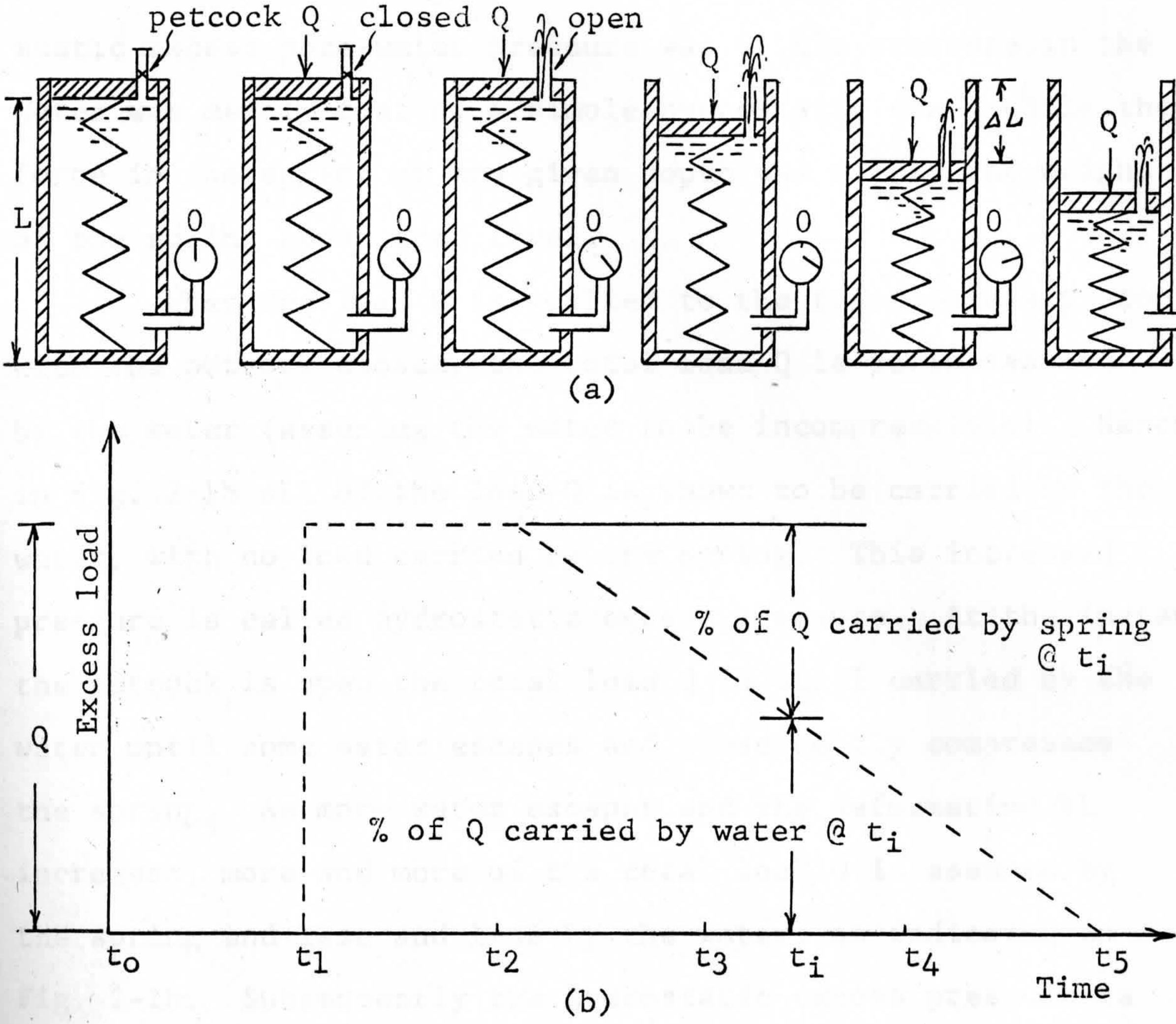
(4) settlement from frost-heaved soils;

(5) other possible factors.

2-3. Consolidation characteristics

One can be aided in understanding the consolidation process by an excellent mechanical analogy schematically shown in Fig. 2-1. Fig. 2-1a is analogous to a saturated clay stratum where the volume of the vessel represents the voids of the soil mass with a porosity of n. All of the voids of the soil mass are fully saturated with water. The spring represents the solid particles, while the petcock, by analogy, represents the seepage paths in the soil through which the water exits. The gauge is to measure only hydrostatic excess pore-water pressure.

Fig. 2-1b shows the relative percentage of the excess load taken by the spring and water at some arbitrary sequence of loading. Before the excess load Q is applied, the hydro-



Corresponding components used in the mechanical analogy model

SOIL MASS	EQUIVALENT	MECHANICAL MODEL
soil	=	spring
saturated voids	=	water
porosity n	=	volume of cylinder
permeability	=	petcock
hydrostatic excess pore-water pressure	=	pressure gauge

Fig. 2-1 Model used to explain by analogy the process of of consolidation of a saturated clay

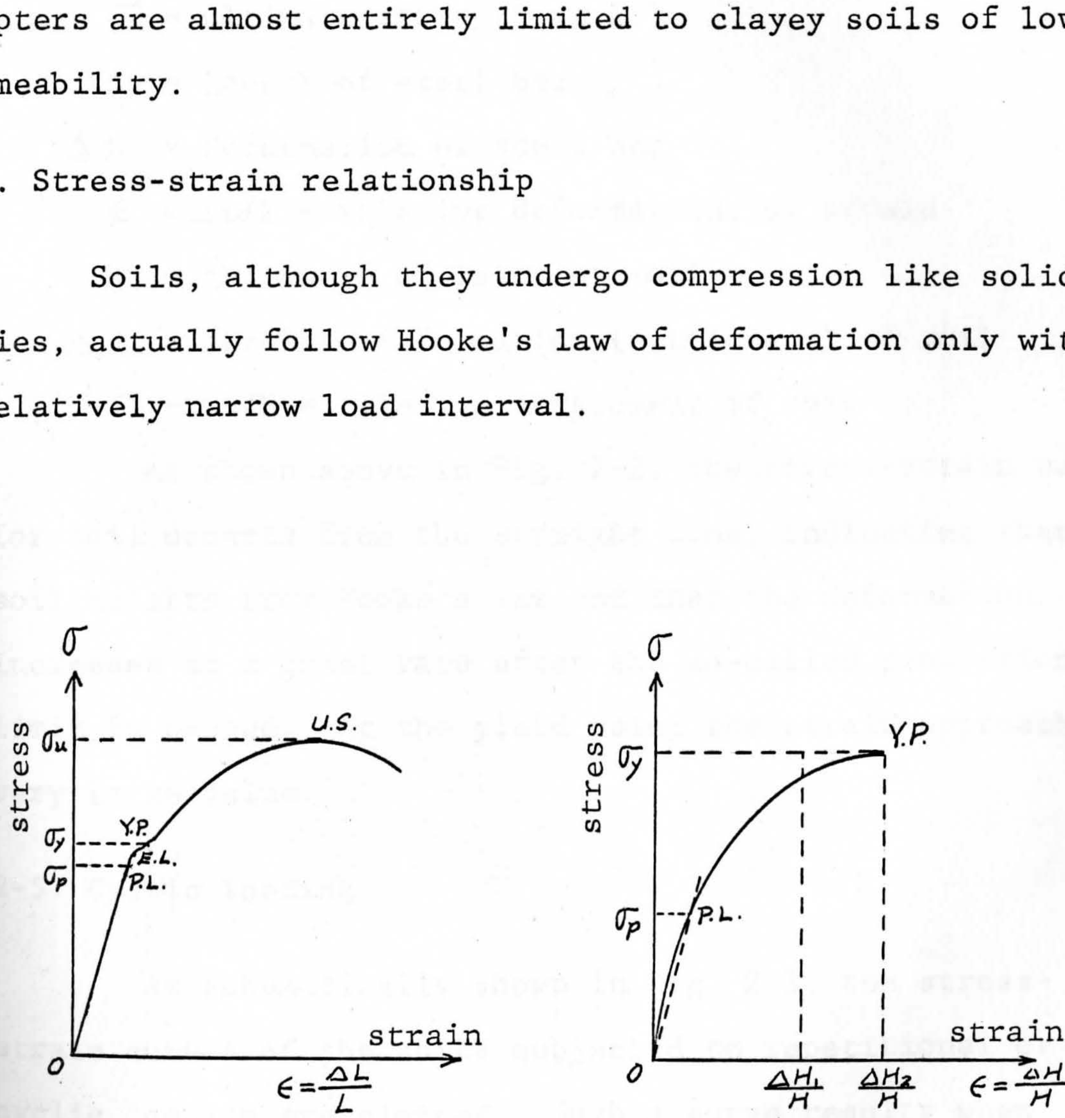
static excess pore-water pressure was 0; the pressure in the water was merely that of a simple hydrostatic case, while the force in the spring at any given depth was merely the weight of the spring above that level.

When the load Q is applied to the frictionless piston with the petcock closed, the total load Q is fully assumed by the water (assuming the water to be incompressible). Hence, in Fig. 2-1b all of the load Q is shown to be carried by the water, with no load carried by the spring. This increased pressure is called hydrostatic excess pressure. At the instant the petcock is open the total load Q is still carried by the water until some water escapes and subsequently compresses the spring. As more water escapes and the deformation ΔL increases, more and more of the total load Q is assumed by the spring and less and less by the water, as indicated in Fig. 2-1b. Subsequently the hydrostatic excess pressure is reduced to 0, with all the applied load carried by the spring. At this point the deformation would cease, analogous to a complete consolidation for a given magnitude of load. Needless to say, increasing the load would constitute a new cycle of the phenomena just described.

"The rate at which the volume change, or consolidation, occurs in a soil is directly related to the permeability of the soil because the permeability controls the speed at which the pore water can escape. The permeability of most sands is so high that the time required for consolidation after a load application can be considered negligible except for cases where a large mass of sand is subjected to a rapid shear or shock. On the other hand, the low permeability of clay makes the rate of volume change after a load application a factor which must be considered."[2] Therefore, the experimental consolidation studies presented in the following chapters are almost entirely limited to clayey soils of low permeability.

2-4. Stress-strain relationship

Soils, although they undergo compression like solid bodies, actually follow Hooke's law of deformation only within a relatively narrow load interval.



(a) stress-strain diagram for steel

(b) stress-strain diagram for a continuously and increasingly loaded soil

Fig. 2-2

where

P.L. = proportional limit

σ_p = stress at proportional limit

E.L. = elastic limit

Y.P. = yield point

σ_y = yield point stress

σ_u = ultimate strength (U.S.)

L = length of steel bar

ΔL = deformation of steel bar

$\epsilon = \Delta L/L$ = relative deformation, or strain

H = thickness of layer of soil

ΔH = absolute deformation (settlement) of soil

$\epsilon = \Delta H/H$ = relative settlement of soil

As shown above in Fig. 2-2, the stress-strain curve for soil departs from the straight line, indicating that soil departs from Hooke's law and that the deformation increases at a great rate after the so-called proportional limit is passed. At the yield point the strain approaches a very large value.

2-5. Cyclic loading

As schematically shown in Fig. 2-3, the stress-strain curves of the soils subjected to repetitional or cyclic loading are plotted. Such a curve results when loading a soil in a lateral confinement, such as a pit surrounded by sheet piling, or in a soil consolidometer.

The branch of the cyclic loading curve, 0-1, is the loading or compression curve, giving a settlement of a soil layer, marked by ΔH_1. Branch 1-2 is the unloading or swelling curve. From the curve shown, we could easily see that after

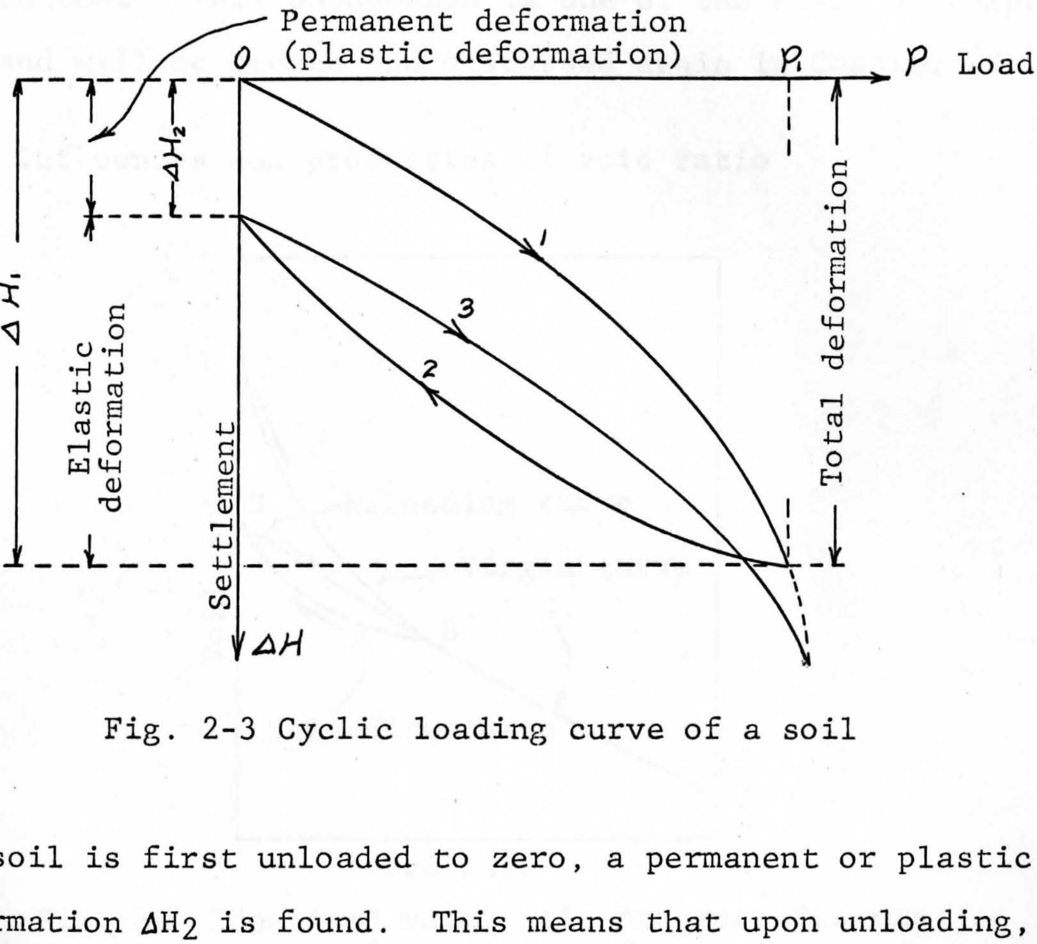


Fig. 2-3 Cyclic loading curve of a soil

the soil is first unloaded to zero, a permanent or plastic deformation ΔH_2 is found. This means that upon unloading, the soil does not swell indefinitely, but swelling ceases at a certain void ratio. Thus, upon unloading, the initial settlement, ΔH_1 does not disappear fully i.e., the load has caused a permanent decrease in the volume of voids. Only the elastic deformations disappear upon unloading the soil. The permanent deformation, ΔH_2, is believed to be caused by soil particle translocation and readjusment. The reloading of the soil, branch 2-3, completes one full loading cycle. The loop of the load-settlement curve is termed the hysteresis loop. Hysteresis loops brought about by cyclic loading are almost parallel among themselves for one soil sample under cyclic test. This phenomenon is one of the bases of comparison and will be proved and discussed again in Chapter IV.

2-6. Influences and properties of void ratio

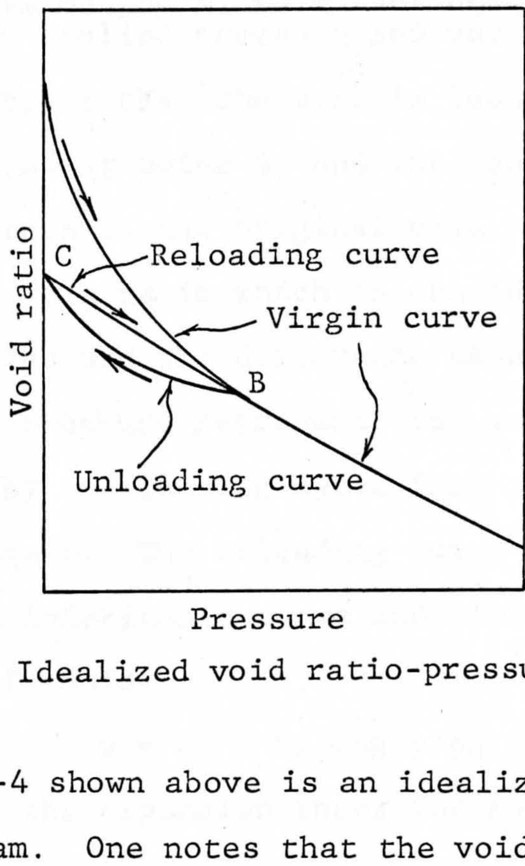


Fig. 2-4 Idealized void ratio-pressure diagram

Fig. 2-4 shown above is an idealized void ratio-pressure diagram. One notes that the void ratio decreases at a relatively rapid rate as early increment of pressure are applied, but the rate of decrease diminishes as the pressure increases. The curve is roughly logarithmic in form; that is, the void ratio decreases in proportion to the logarithm of the pressure. An empirical equation for the curve can be written

$$e = e_o - C_c \log p/p_o \qquad (2\text{-}1)$$

in which

e = void ratio at pressure p;

p = any applied pressure;

e_o = known void ratio at pressure p_o;

p_o = pressure at which void ratio is known; and

C_c = compression index, which defines the relationship between applied pressure and void ratio.

Now suppose that the soil is loaded to some pressure such as indicated at point B, and the load is then removed. Rather than return to the original void ratio, the soil will expand to some void ratio which is considerably less than the initial value and the difference between the two void ratios at zero pressure represents the permanent change in volume caused by the loading cycle from zero to point B and back to zero again. The unloading curve from B to C also approximates a logarithmic curve and may be represented by the following formula:

$$e = e_o - C_E \log p/p_o \qquad (2\text{-}2)$$

in which C_E is the expansion index for soil.

2-7. Compression index

Since the void ratio-pressure relationship for soil is approximately logarithmic in character, when plotted on semilogarithmic graph paper with the pressure on the logarithmic scale, the virgin curve will approximate a straight line. The slope of the curve is negative on this type of plot and is equal numerically to the compression index C_c. From Eq. (2-1), we obtain

$$C_c = \frac{e_o - e}{\log p/p_o} \qquad (2\text{-}3)$$

when $p = 10\ p_o$

$$C_c = e_o - e \qquad (2\text{-}4)$$

That is, the compression index may be quickly determined from a void ratio-pressure diagram by subtracting the void ratio at $10\ p_o$ from the void ratio at p_o.

2-8. Preconsolidation

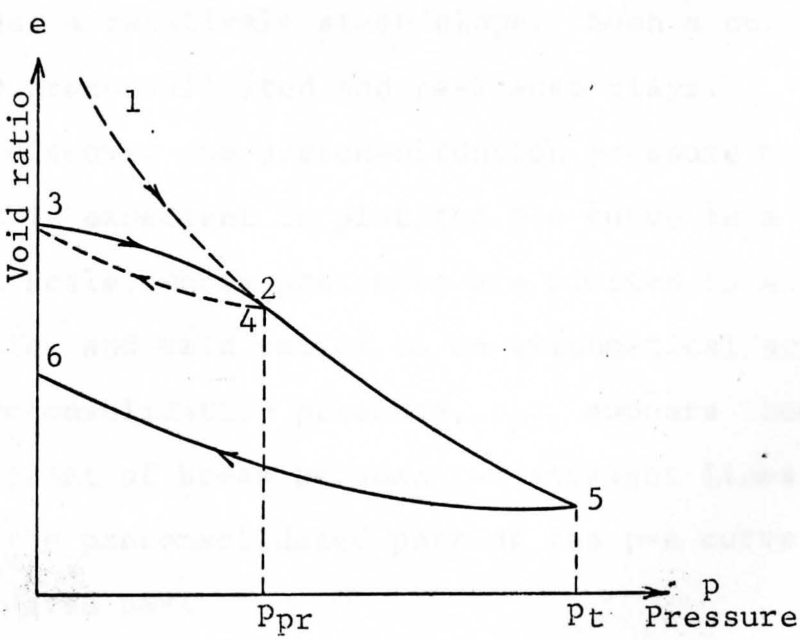


Fig. 2-5 Pressure-void ratio curve of a preconsolidated soil. Arithmetic scale

The preconsolidation load is the probable load to which a soil stratum may have been subjected in the geological past. From Fig. 2-5 shown above, we notice that the soil has been preconsolidated along the theoretical p-e curve from point 1 to point 2 and the preconsolidation pressure p_{pr} is found at point 2. After the consolidation time elapsed the load on the soil was removed to zero, i.e., branch 2-3. Some time after the unloading this sample could possibly be taken and subjected to a consolidation test under loads from $p=0$ to $p=p_t$, i.e., branch 3-4-5, past the previous preconsolidation pressure p_{pr}, that existed on the soil earlier. It can be easily observed that, in Fig.2-5, the pressure-void ratio (p-e curve) has a relatively flat slope from $p=0$ to $p=p_{pr}$, whereas past the preconsolidation pressure the curve has a relatively steep slope. Such a curve is typical for preconsolidated and re-loaded clays.

To discover the preconsolidation pressure most readily, it is expedient to plot the p-e curve to a semi-logarithmic scale, where pressures are plotted to a logarithmic scale, and void ratios to an arithmetical scale. Then the preconsolidation pressure, p_{pr}, appears theoretically as a point of break between two straight lines, separating the preconsolidated part of the p-e curve from the consolidated part.

To determine the preconsolidation pressure, A. Casagrande suggested the following method, see Fig.2-6.

"Locate by eye the point of the greatest curvature (point A on the recompression branch, for example). Through this point draw a horizontal line, A-h, and a tangent, A-t, to the curve. Bisect the angle ∢hAt, thus formed by the by-sector, Ab. Then draw backwards, from point 5, the straight line part of the compression curve, 5-5' (straight dotted line, a-a), until it intersects the bisector line, Ab, to give point p_{pr}. This point then approximately determines the preconsolidation pressure."[3]

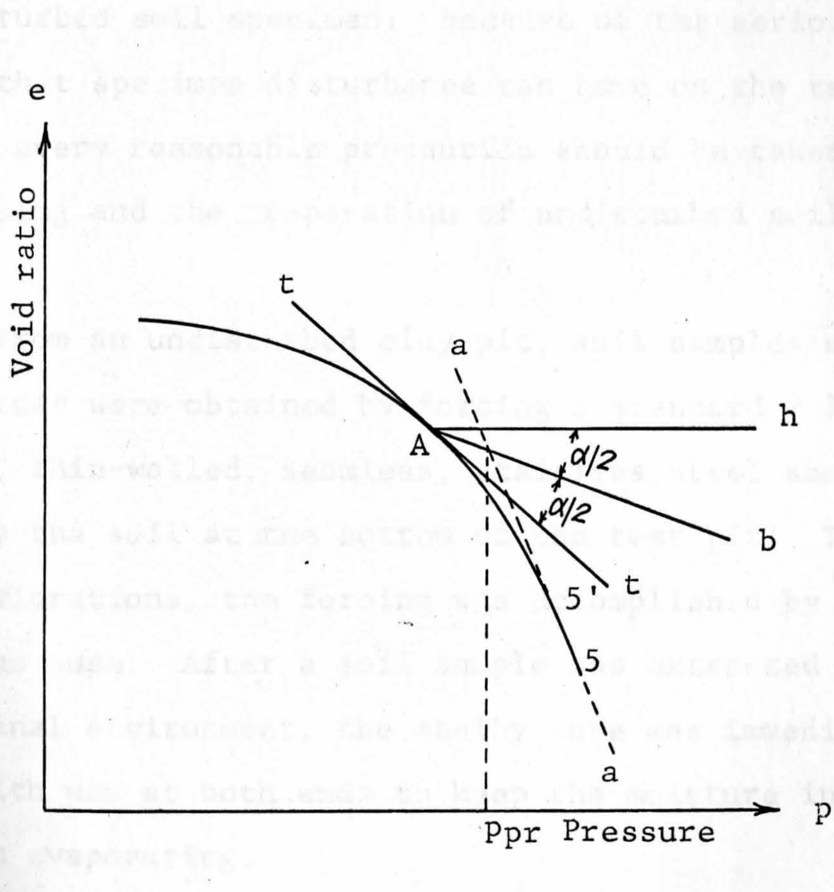


Fig. 2-6 Casagrande method of determining preconsolidation pressure

CHAPTER III

LABORATORY PREPARATIONS AND TESTING PROCEDURES

3-1. Sampling

It has already been mentioned that the success of a consolidation test is heavily dependent upon obtaining an undisturbed soil specimen. Because of the serious effects that specimen disturbance can have on the test results, every reasonable precaution should be taken in the handling and the preparation of undisturbed soil samples.

From an undisturbed clay pit, soil samples used for the test were obtained by forcing a standard 2 7/8 inch diameter, thin-walled, seamless, stainless steel shelby tube into the soil at the bottom of the test pit. To prevent vibrations, the forcing was accomplished by a continuous push. After a soil sample was extracted from its original environment, the shelby tube was immediately sealed with wax at both ends to keep the moisture in the soil from evaporating.

During the specimen preparation, the top 6-inch and the bottom 6-inch sections of the shelby tube were cut off and discarded to reduce any effects of end bearing caused by driving the tube. The remaining middle part was cut into four samples at desired lengths. These were then tested in two sets of adjacent samples to keep the samples as identical as possible.

3-2. Testing apparatus

The following is a description of the testing apparatus, sample preparations and specifications used for both methods:

(1) The consolidometer is used to hold the sample in a ring which is either fixed to the base of the container or floating. Porous stones were placed to each end face of the sample. Both consolidometers also provide a means for submerging the sample, for applying a vertical load, and for measuring the change in thickness of the sample. They are shown schematically in Fig. 3-1 and Fig. 3-2.

(2) The sample diameter used was 2 1/2 inch for the standard method and 2 7/8 inch for the new method; the minimum sample thickness should be 1/2 inch.

(3) The ring is made of a material that is non-corrosive; for the new method, the floating ring is part of the shelby tube.

(4) The porous stones are of silicon carbide, or aluminum oxide. The stone thickness shall be sufficient to prevent breaking.

(5) The porous stones should be saturated so as not to

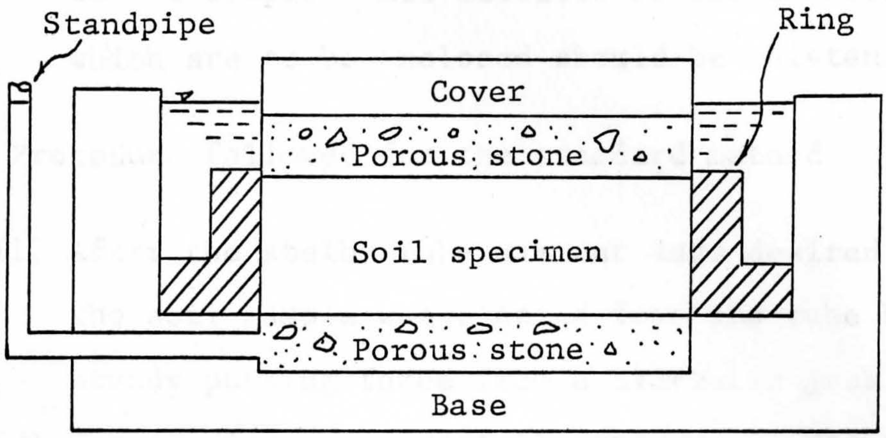


Fig. 3-1 Fixed-ring container for standard consolidation method

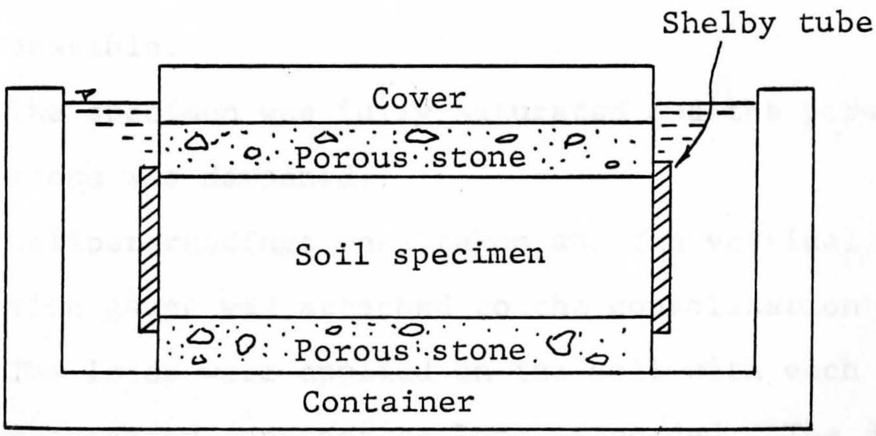


Fig. 3-2 Floating-ring container for new consolidation method

absorb water from the sample or to contribute water to the sample. All surfaces of the consolidometer which are to be enclosed should be moistened.

3-3. Procedure followed for the standard method

(1) After the shelby tube was cut into desired length, the soil sample was ejected from the tube by a steady pushing force from a hydraulic jack.

(2) The specimen was carefully trimmed to the inside diameter of the consolidometer ring, placed in the ring and trimmed flush with the plane surface of the ring. Special caution was taken during this process to insure as near a perfect fit as is possible.

(3) The specimen was fully saturated and the porous stone was dampened.

(4) Caliper readings were taken and the vertical deflection gauge was attached to the consolidation assembly.

(5) The loads were applied on the soil with each weight maintained constant as long as needed. The deflection reading was taken and recorded before each pressure increment was applied. The weight were applied in the following increments: 2.2 lbs., 4.4 lbs., 6.6 lbs., 8.8 lbs., 17.6 lbs., 26.4 lbs., 35.2 lbs., 17.6 lbs. and 2.2 lbs.

(6) At the completion of the test, the entire sample was removed from the consolidometer. The sample was placed in the oven to dry and the dry weight was taken and recorded to obtain the weight of solids.

3-4. Procedure followed for the new method

(1) The shelby tube was cut into a desired length and thickness.

(2) The inside diameter of the shelby tube was measured and recorded. Also, caliper readings of the thickness of the porous stones to be used were taken.

(3) The soil samples were fully saturated and the porous stones were dampened.

(4) The shelby tube with the porous stones at the top and the bottom was placed in a plastic molded container.

(5) The vertical deflection gauge was attached to the consolidation assembly and the weights were applied to the specimen. Each weight was left on as long as needed and a deflection reading was taken and recorded before each pressure increment was applied. The weights were applied in the following increments: 2.2 lbs., 4.4 lbs., 6.6 lbs., 8.8 lbs., 17.6 lbs., 26.4 lbs., 35.2 lbs., 44.0 lbs., 17.6 lbs. and 2.2 lbs.

(6) At the completion of the test, the soil sample was removed from the shelby tube and was placed in the oven to dry. The dry weight was taken and recorded to obtain the weight of solids.

3-5. Calculations

(1) Standard method:

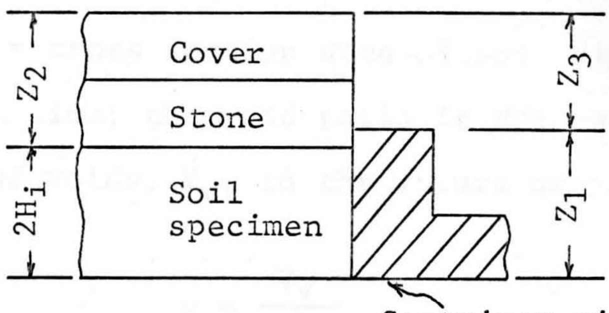


Fig. 3-3 Dimensions of container for standard consolidation test.

As shown above in Fig. 3-3, the sample thickness $2H_i$ is calculated from

$$2H_i = Z_1 - Z_2 + Z_3 \qquad (3\text{-}1)$$

where

$2H_i$ = the height of the soil specimen

Z_1 = the thickness of the consolidometer ring

Z_2 = the combined thickness of the upper porous stone and the cover

Z_3 = the depth from the top of the cover to the top of the ring.

The height of solids, $2H_o$, is found from the Eq.

$$2H_o = \frac{W_s}{G \gamma_w A} \tag{3-2}$$

in which

$2H_o$ = height of solids

W_s = dry weight of soil sample

G = specific gravity of soil

γ_w = unit weight of water

A = cross section area of soil sample.

By definition, the void ratio is the ratio of the volume of voids, V_v, to the volume of solids, V_s, i.e.

$$e = \frac{V_v}{V_s} \tag{3-3}$$

or

$$e = \frac{A\ (2H_i - 2H_o)}{A\ 2H_o} \tag{3-4}$$

$$e = \frac{2H_i - 2H_o}{2H_o} \tag{3-5}$$

(2) New method:

As shown in Fig. 3-4, the sample thickness, $2H_i$, is obtained from

$$2H_i = H_4 - H_5 - H_6 \tag{3-6}$$

where

Z_4 = combined thickness of specimen and two porous stones

Z_5, Z_6 = thickness of top and bottom porous stones.

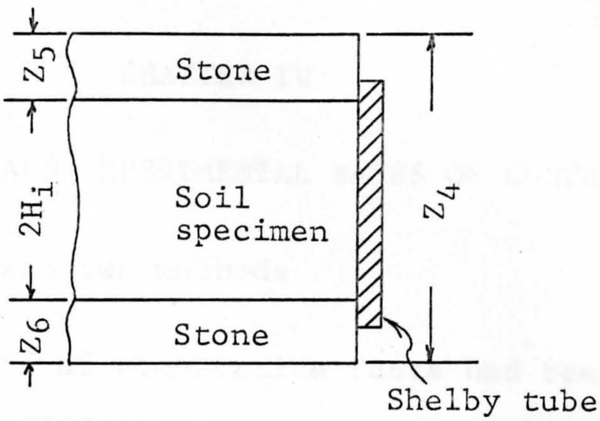


Fig. 3-4. Dimensions of container for new consolidation test.

Using Eq. (3-2), $2H_o = \frac{W_s}{G \gamma_w A}$, the height of solids is obtained. Substitute $2H_o$ and $2H_i$ into Eq. (3-5), $e = \frac{2H_i - 2H_o}{2H_o}$, the void ratio for the new method is also obtained.

CHAPTER IV

THEORETICAL AND EXPERIMENTAL BASES OF COMPARISON

4-1. Test results from two methods

Eighteen sets of comparative tests had been accomplished through two different processes described in the previous chapter. The previously described procedures were carefully followed and data were accurately recorded. Void ratios of soil samples subjected to different applied pressures were subsequently calculated and plotted as void ratio vs log of pressure curves. The records are presented from Table 4-1 to Table 4-18, and the figures are shown from Fig. 4-1 to Fig. 4-18 in the following pages.

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0910	.4608	.7312
428	2.2	.0118	.0118	1.0792	.4490	.7125
856	4.4	.0223	.0105	1.0687	.4385	.6958
1,284	6.6	.0301	.0078	1.0609	.4307	.6834
1,712	8.8	.0374	.0073	1.0536	.4234	.6718
3,424	17.6	.0553	.0179	1.0357	.4055	.6434
5,136	26.4	.0639	.0086	1.0271	.3969	.6298
6,848	35.2	.0690	.0051	1.0220	.3918	.6217
8,560	44.0	.0740	.0050	1.0170	.3868	.6138
3,424	17.6	.0665	-.0075	1.0245	.3943	.6257
428	2.2	.0436	-.0229	1.0474	.4172	.6620

$2H_i$ = 1.0910 in. $2H_o$ = 0.6302 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9980	.4132	.7066
567	2.2	.0289	.0289	.9691	.3843	.6571
1,134	4.4	.0429	.0140	.9551	.3703	.6332
1,701	6.6	.0551	.0122	.9429	.3581	.6123
2,268	8.8	.0603	.0052	.9337	.3529	.6034
4,536	17.6	.0806	.0203	.9174	.3326	.5687
6,804	26.4	.0957	.0151	.9023	.3175	.5429
9,072	35.2	.1046	.0089	.8934	.3085	.5275
4,536	17.6	.1005	-.0041	.8975	.3127	.5347
567	2.2	.0795	-.0210	.9185	.3337	.5706

$2H_i$ = 0.9980 in. $2H_o$ = 0.5848 in.

Table 4-1. Results for test No. 1

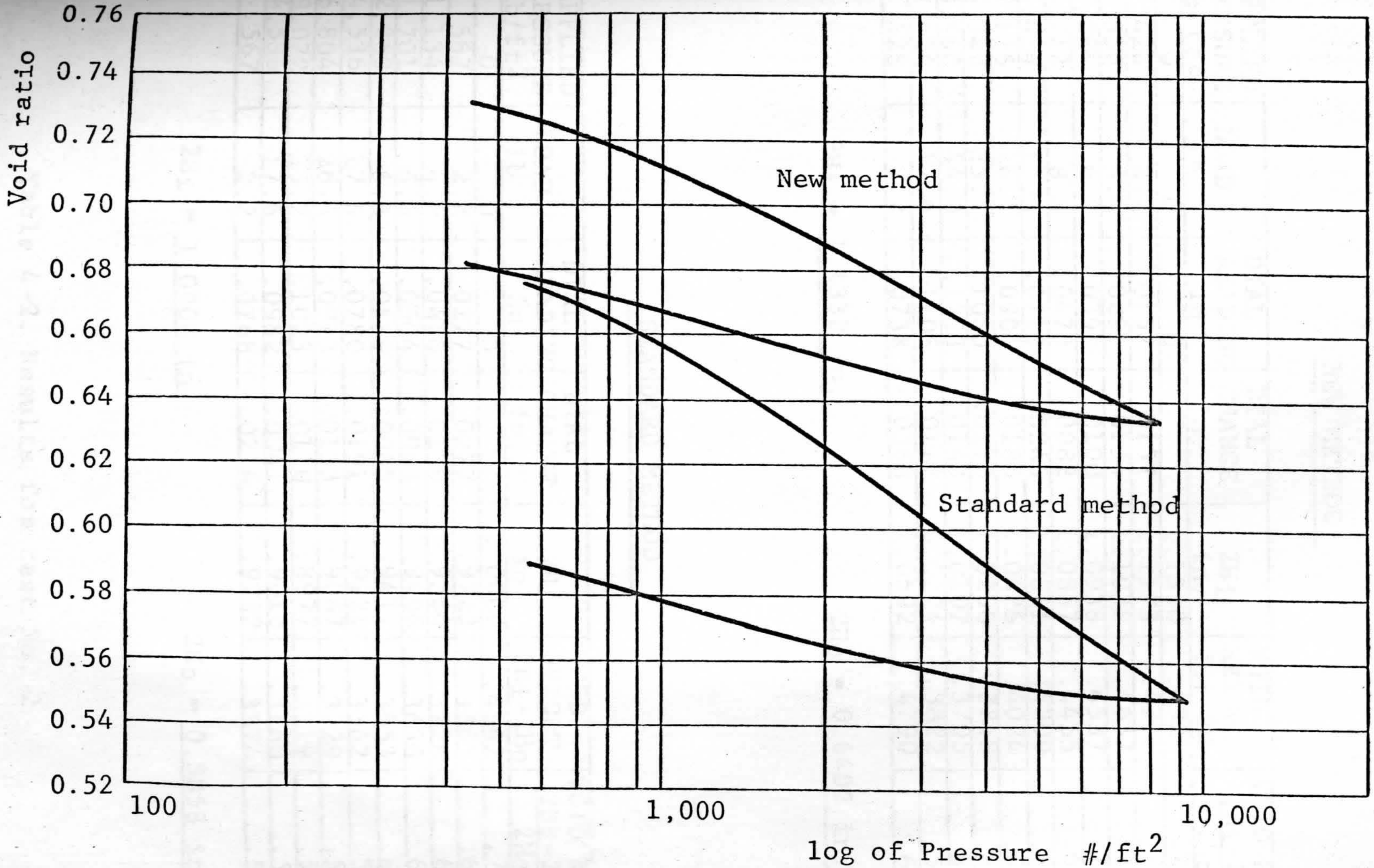


Fig. 4-1. e - log p curves for test No. 1

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.1330	.4928	.7698
428	2.2	.0157	.0157	1.1173	.4771	.7452
856	4.4	.0291	.0134	1.1038	.4637	.7243
1,284	6.6	.0391	.0100	1.0938	.4537	.7087
1,712	8.8	.0473	.0082	1.0857	.4455	.6959
3,424	17.6	.0719	.0246	1.0611	.4209	.6575
5,136	26.4	.0904	.0185	1.0426	.4024	.6286
6,848	35.2	.1080	.0176	1.0250	.3848	.6011
8,560	44.0	.1223	.0143	1.0107	.3705	.5787
3,424	17.6	.1106	-.0117	1.0224	.3822	.5970
428	2.2	.0738	-.0368	1.0592	.4190	.6545

$2H_i$ = 1.1330 in. $2H_o$ = 0.6402 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0000	.4142	.7071
567	2.2	.0277	.0277	.9723	.3865	.6598
1,134	4.4	.0416	.0139	.9584	.3726	.6361
1,701	6.6	.0504	.0088	.9496	.3638	.6210
2,268	8.8	.0589	.0085	.9411	.3553	.6065
4,536	17.6	.0780	.0191	.9221	.3362	.5739
6,804	26.4	.0913	.0133	.9087	.3229	.5512
9,072	35.2	.1023	.0110	.8977	.3119	.5324
4,536	17.6	.0982	-.0041	.9018	.3160	.5394
567	2.2	.0768	-.0214	.9232	.3374	.5760

$2H_i$ = 1.0000 in. $2H_o$ = 0.5858 in.

Table 4-2. Results for test No. 2

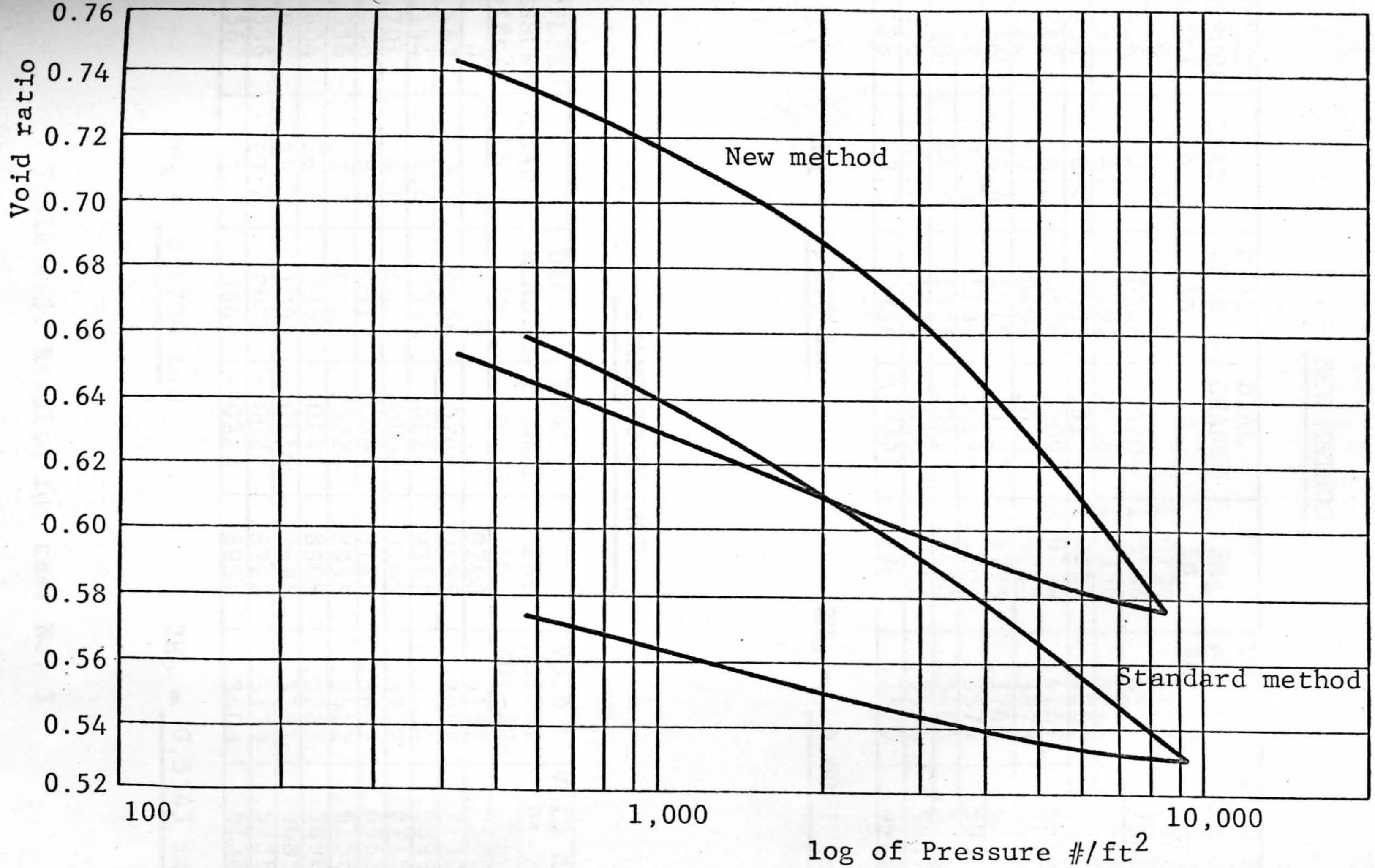


Fig. 4-2. e - log p curves for test No. 2

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0790	.4763	.7903
428	2.2	.0206	.0206	1.0584	.4557	.7561
856	4.4	.0392	.0186	1.0398	.4371	.7252
1,284	6.6	.0503	.0111	1.0287	.4260	.7068
1,712	8.8	.0596	.0093	1.0194	.4167	.6914
3,424	17.6	.0890	.0294	.9900	.3873	.6426
5,136	26.4	.1015	.0125	.9775	.3748	.6219
6,848	35.2	.1189	.0174	.9601	.3574	.5930
8,560	44.0	.1300	.0111	.9490	.3463	.5746
3,424	17.6	.1239	-.0061	.9551	.3524	.5847
428	2.2	.0888	-.0351	.9902	.3875	.6429

$2H_i$ = 1.079 in. $2H_o$ = 0.6027 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9700	.4157	.7500
567	2.2	.0205	.0205	.9495	.3052	.7130
1,134	4.4	.0333	.0128	.9367	.3824	.6899
1,701	6.6	.0435	.0102	.9265	.3722	.6715
2,268	8.8	.0519	.0084	.9181	.3638	.6563
4,536	17.6	.0745	.0226	.8955	.3412	.6156
6,804	26.4	.0892	.0147	.8808	.3265	.5890
9,072	35.2	.1006	.0114	.8694	.3151	.5685
4,536	17.6	.0978	-.0028	.8722	.3179	.5735
567	2.2	.0649	-.0229	.8951	.3408	.6138

$2H_i$ = 0.9700 in. $2H_o$ = 0.5543 in.

Table 4-3. Results for test No. 3

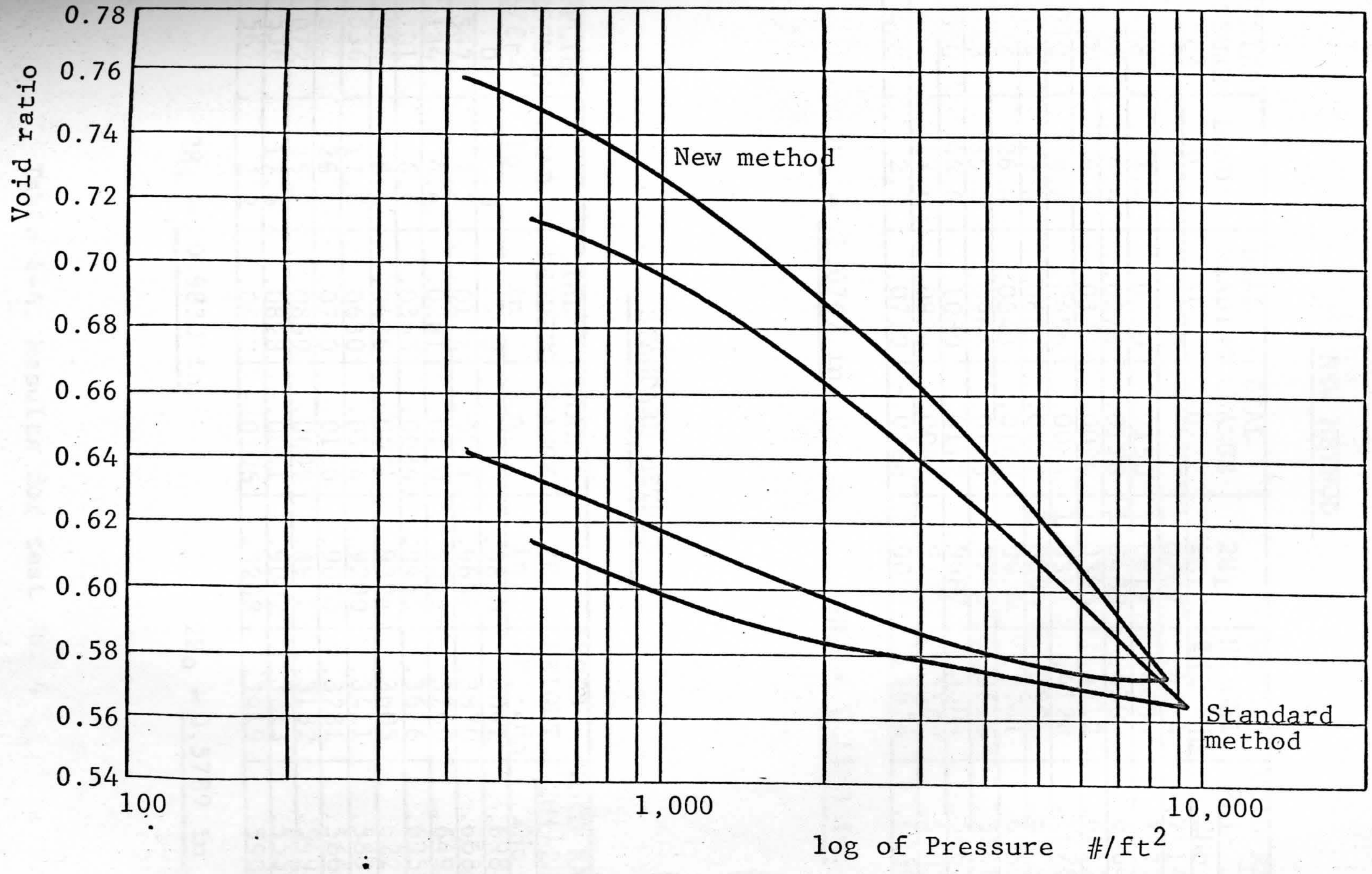


Fig. 4-3. e - log p curves for test No. 3

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0767	.4584	.7414
428	2.2	.0150	.0150	1.0617	.4434	.7171
856	4.4	.0278	.0128	1.0489	.4306	.6964
1,284	6.6	.0372	.0094	1.0395	.4212	.6812
1,712	8.8	.0456	.0084	1.0311	.4128	.6676
3,424	17.6	.0683	.0227	1.0084	.3901	.6309
5,136	26.4	.0843	.0160	.9924	.3741	.6050
6,848	35.2	.0968	.0125	.9799	.3616	.5848
8,560	44.0	.1080	.0112	.9687	.3504	.5667
3,424	17.6	.0996	-.0084	.9771	.3588	.5803
428	2.2	.0710	-.0286	1.0057	.3874	.6266

$2H_i$ = 1.0767 in. $2H_o$ = 0.6183 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9830	.4041	.6980
567	2.2	.0171	.0171	.9659	.3870	.6685
1,134	4.4	.0281	.0110	.9549	.3760	.6495
1,701	6.6	.0365	.0084	.9465	.3676	.6350
2,268	8.8	.0436	.0071	.9394	.3605	.6227
4,536	17.6	.0630	.0194	.9200	.3411	.5892
6,804	26.4	.0760	.0130	.9070	.3281	.5668
9,072	35.2	.0859	.0099	.8971	.3182	.5497
4,536	17.6	.0828	-.0031	.9002	.3213	.5550
567	2.2	.0622	-.0206	.9208	.3419	.5906

$2H_i$ = 0.9830 in. $2H_o$ = 0.5789 in

Table 4-4. Results for test No. 4

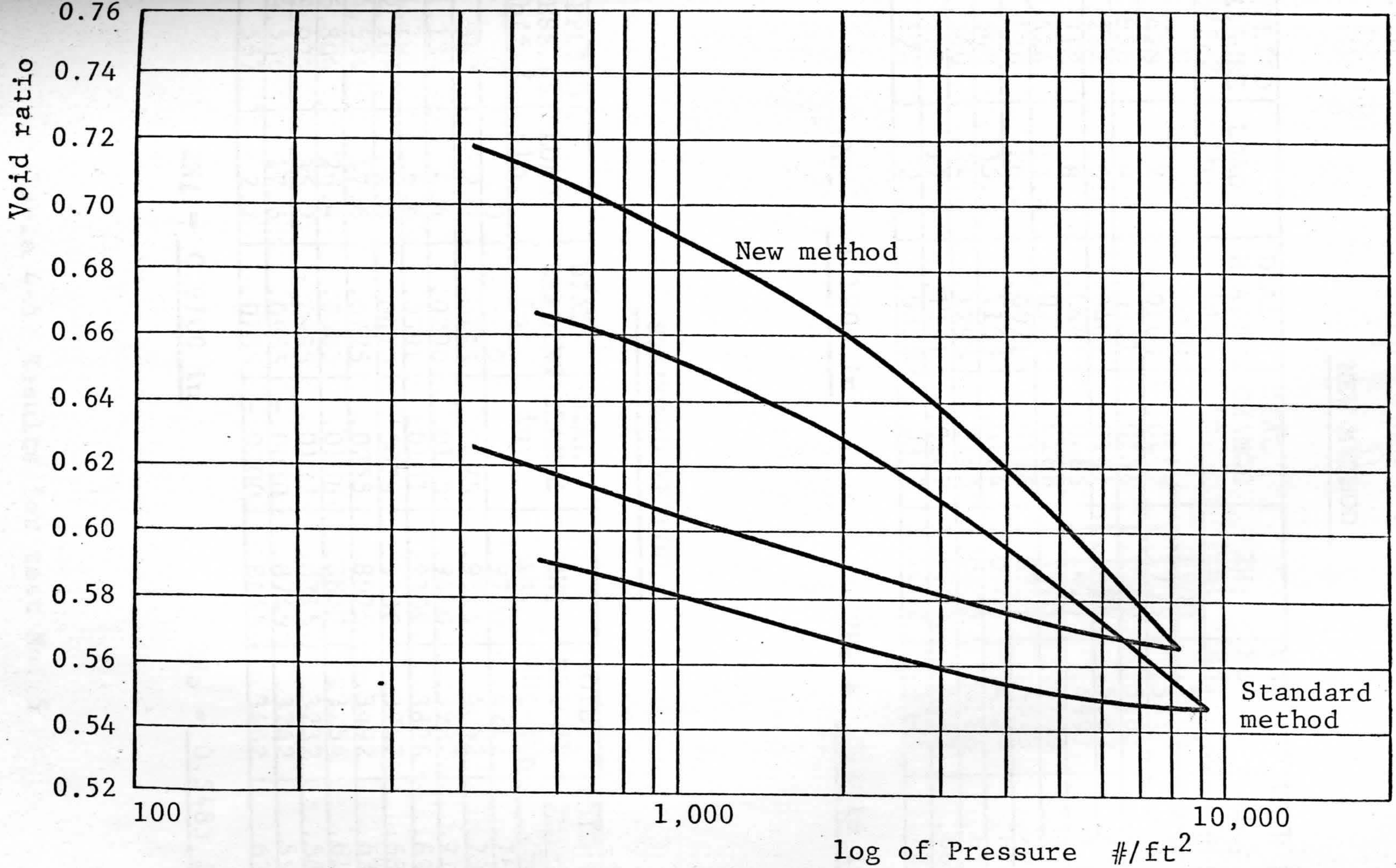


Fig. 4-4. e - log p curves for test No. 4

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.1900	.5484	.8547
428	2.2	.0165	.0165	1.1735	.5319	.8290
856	4.4	.0301	.0136	1.1599	.5183	.8078
1,284	6.6	.0402	.0101	1.1498	.5082	.7921
1,712	8.8	.0487	.0085	1.1413	.4997	.7788
3,424	17.6	.0775	.0288	1.1125	.4709	.7339
5,136	26.4	.0971	.0196	1.0929	.4513	.7034
6,848	35.2	.1155	.0184	1.0745	.4329	.6747
8,560	44.0	.1298	.0143	1.0602	.4186	.6524
3,424	17.6	.1191	-.0107	1.0709	.4293	.6691
428	2.2	.0858	-.0333	1.1042	.4626	.7210

$2H_i$ = 1.1900 in. $2H_o$ = 0.6416 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9700	.4217	.7691
567	2.2	.0175	.0175	.9525	.4042	.7372
1,134	4.4	.0290	.0115	.9410	.3927	.7162
1,701	6.6	.0391	.0101	.9309	.3826	.6978
2,268	8.8	.0470	.0079	.9230	.3747	.6834
4,536	17.6	.0725	.0255	.8975	.3492	.6369
6,804	26.4	.0863	.0138	.8837	.3254	.6117
9,072	35.2	.0985	.0122	.8715	.3232	.5895
4,536	17.6	.0955	-.0030	.8745	.3262	.5949
567	2.2	.0755	-.0200	.8945	.3462	.6314

$2H_i$ = 0.9700 in. $2H_o$ = 0.5483 in.

Table 4-5. Results for test No. 5

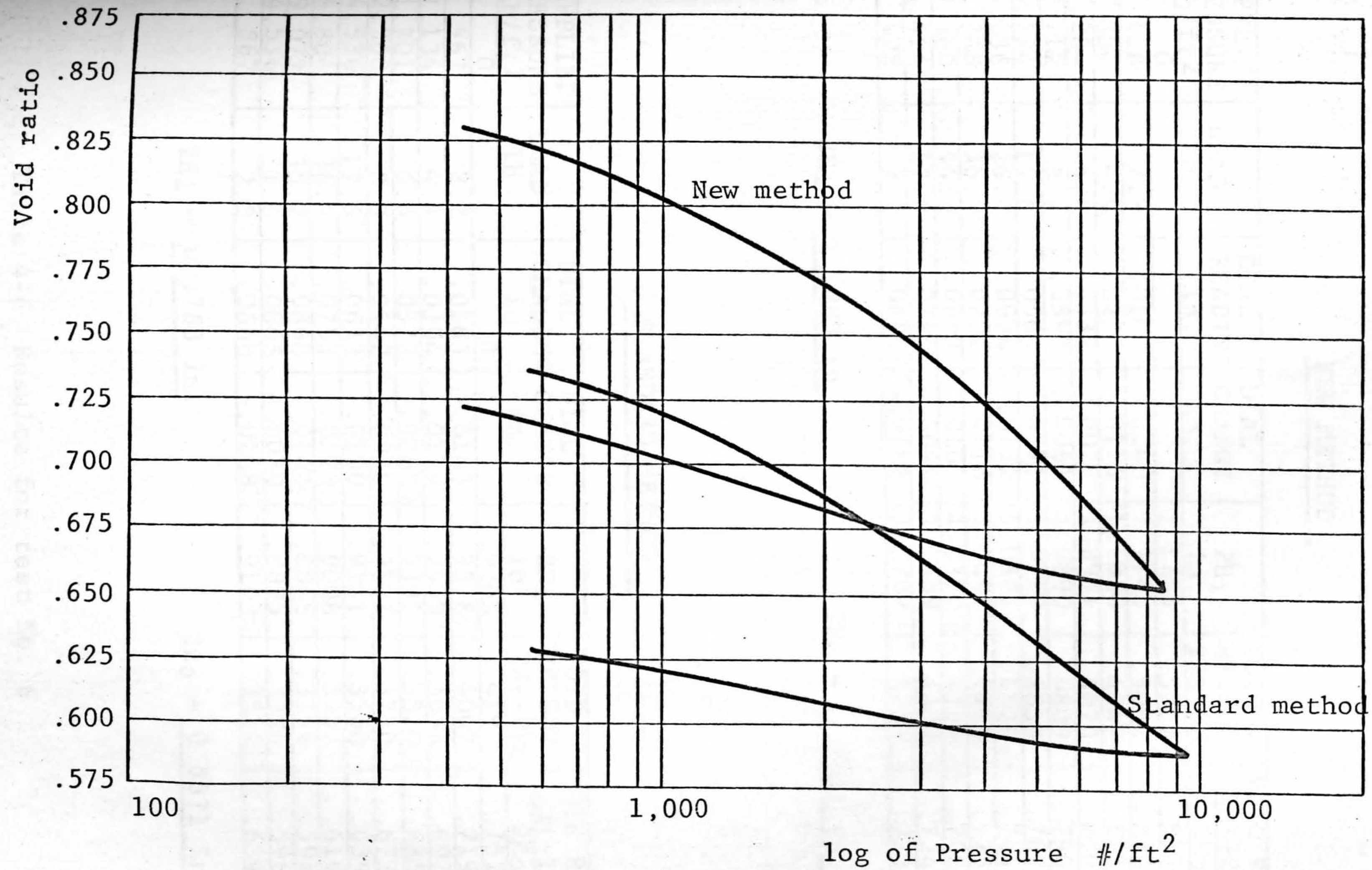


Fig. 4-5. e - log p curves for test No. 5

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i - 2H_o$	VOID RATIO $\frac{2H_i - 2H_o}{2H_o}$
0	0	0		1.1283	.5202	.8555
428	2.2	.0140	.0140	1.1143	.5062	.8324
856	4.4	.0255	.0115	1.1028	.4947	.8135
1,284	6.6	.0332	.0077	1.0951	.4870	.8009
1,712	8.8	.0393	.0061	1.0890	.4809	.7908
3,424	17.6	.0564	.0171	1.0719	.4638	.7627
5,136	26.4	.0676	.0112	1.0607	.4526	.7443
6,848	35.2	.0792	.0116	1.0491	.4410	.7252
8,560	44.0	.0893	.0101	1.0390	.4309	.7086
3,424	17.6	.0794	-.0099	1.0489	.4408	.7249
428	2.2	.0496	-.0298	1.0787	.4706	.7739

$2H_i$ = 1.1283 in. $2H_o$ = 0.6081 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i - 2H_o$	VOID RATIO $\frac{2H_i - 2H_o}{2H_o}$
0	0	0		.9763	.4192	.7525
567	2.2	.0185	.0185	.9578	.4007	.7193
1,134	4.4	.0304	.0119	.9459	.3888	.6979
1,701	6.6	.0372	.0068	.9391	.3820	.6857
2,268	8.8	.0442	.0070	.9321	.3750	.6731
4,536	17.6	.0632	.0190	.9131	.3560	.6390
6,804	26.4	.0755	.0123	.9008	.3437	.6169
9,072	35.2	.0898	.0143	.8865	.3294	.5913
4,536	17.6	.0868	-.0030	.8895	.3324	.5967
567	2.2	.0660	-.0208	.9103	.3532	.6340

$2H_i$ = 0.9763 in. $2H_o$ = 0.5571 in.

Table 4-6. Results for test No. 6

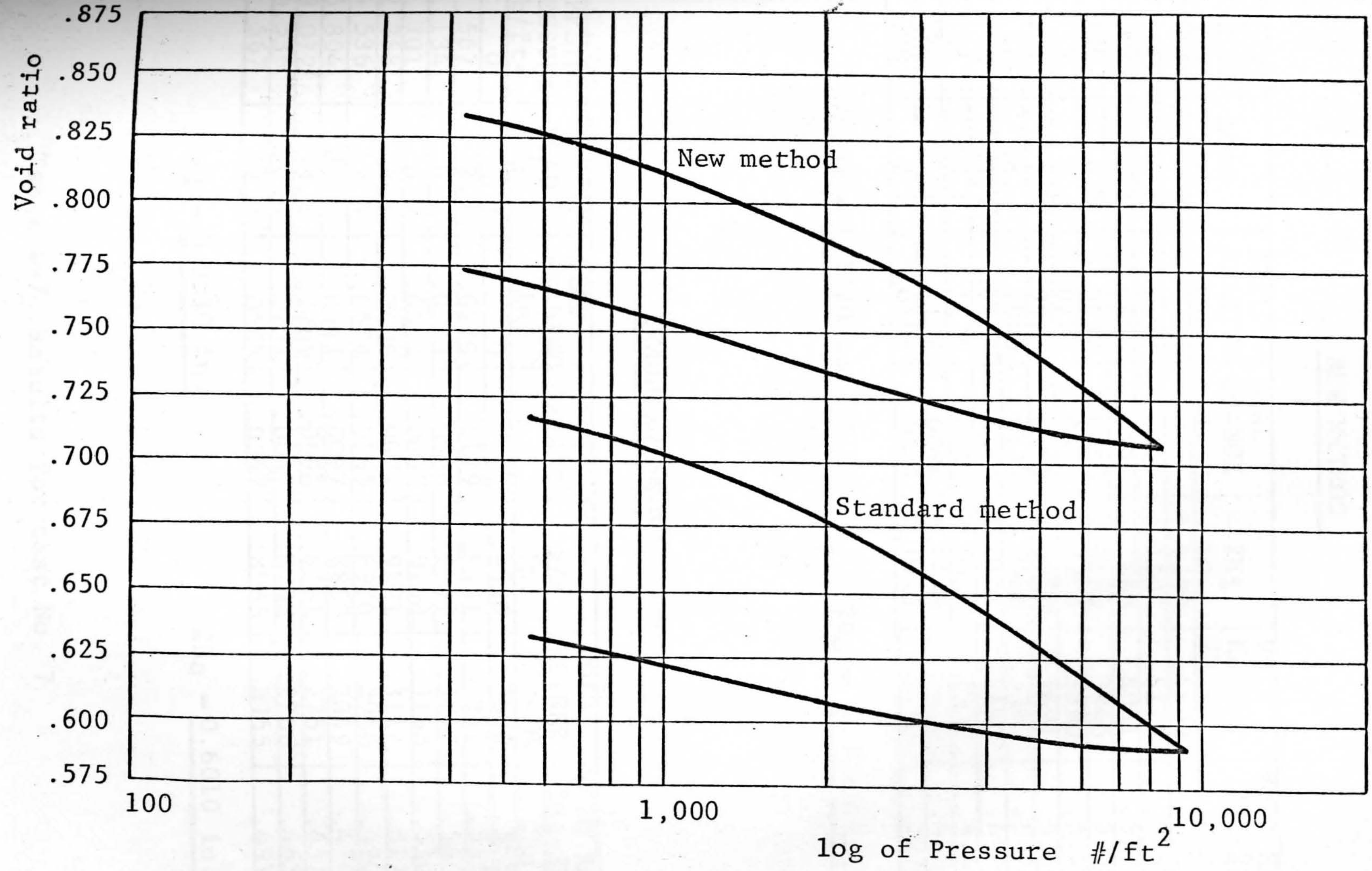


Fig. 4-6. e - log p curves for test No. 6

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.1620	.4602	.6557
428	2.2	.0427	.0427	1.1193	.4175	.5949
856	4.4	.0551	.0124	1.1069	.4051	.5772
1,284	6.6	.0637	.0086	1.0983	.3965	.5650
1,712	8.8	.0712	.0075	1.0908	.3890	.5543
3,424	17.6	.0911	.0199	1.0709	.3691	.5259
5,136	26.4	.1040	.0129	1.0580	.3562	.5076
6,848	35.2	.1159	.0119	1.0461	.3443	.4906
8,560	44.0	.1270	.0111	1.0350	.3332	.4748
3,424	17.6	.1176	-.0094	1.0444	.3426	.4882
428	2.2	.0899	-.0277	1.0721	.3703	.5276

$2H_i$ = 1.1620 in. $2H_o$ = 0.7018 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9530	.3520	.5857
567	2.2	.0179	.0179	.9351	.3341	.5559
1,134	4.4	.0273	.0094	.9257	.3247	.5403
1,701	6.6	.0332	.0059	.9198	.3188	.5304
2,268	8.8	.0409	.0077	.9121	.3111	.5176
4,536	17.6	.0574	.0165	.8956	.2946	.4902
6,804	26.4	.0671	.0097	.8859	.2849	.4740
9,072	35.2	.0769	.0098	.8761	.2751	.4577
4,536	17.6	.0732	-.0037	.8798	.2788	.4639
567	2.2	.0565	-.0167	.8965	.2955	.4917

$2H_i$ = 0.9530 in. $2H_o$ = 0.6010 in.

Table 4-7. Results for test No. 7

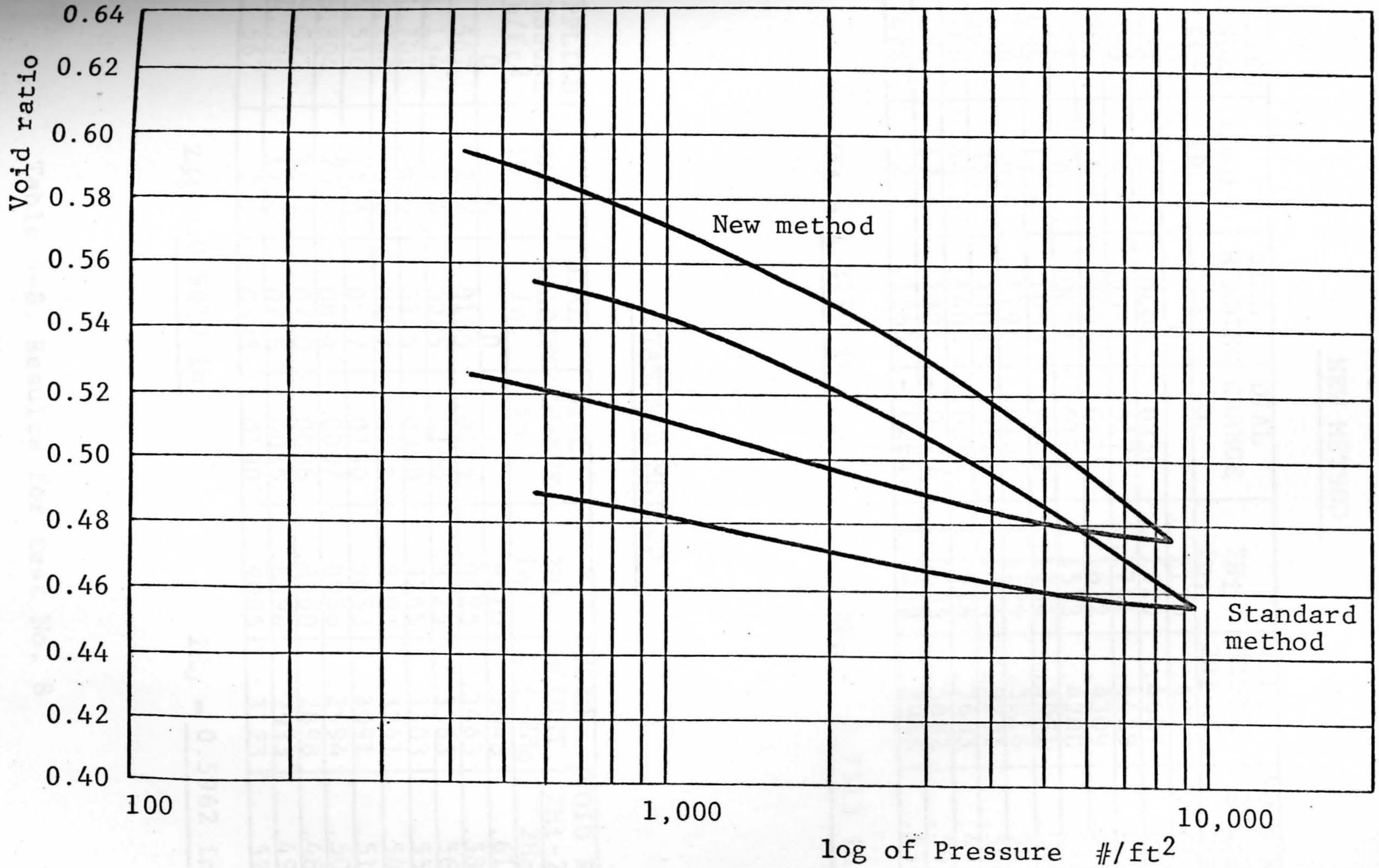


Fig. 4-7. e - log p curves for test No. 7

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.2170	.4857	.6642
428	2.2	.0400	.0250	1.1920	.4607	.6300
856	4.4	.0579	.0179	1.1741	.4428	.6055
1,284	6.6	.0698	.0119	1.1622	.4309	.5892
1,712	8.8	.0807	.0109	1.1513	.4200	.5743
3,424	17.6	.1056	.0249	1.1264	.3951	.5403
5,136	26.4	.1191	.0135	1.1129	.3816	.5218
6,848	35.2	.1306	.0115	1.1014	.3701	.5061
8,560	44.0	.1404	.0098	1.0916	.3603	.4927
3,424	17.6	.1325	-.0079	1.0995	.3682	.5035
428	2.2	.1061	-.0264	1.1259	.3946	.5396

$2H_i$ = 1.2170 in. $2H_o$ = 0.7313 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9610	.3648	.6119
567	2.2	.0185	.0185	.9425	.3463	.5808
1,134	4.4	.0285	.0100	.9325	.3363	.5641
1,701	6.6	.0345	.0060	.9265	.3303	.5540
2,268	8.8	.0407	.0062	.9203	.3241	.5436
4,536	17.6	.0557	.0150	.9053	.3091	.5185
6,804	26.4	.0654	.0097	.8956	.2994	.5022
9,072	35.2	.0750	.0096	.8860	.2898	.4861
4,536	17.6	.0715	-.0035	.8896	.2933	.4919
567	2.2	.0525	-.0190	.9085	.3123	.5238

$2H_i$ = 0.9610 in. $2H_o$ = 0.5962 in.

Table 4-8. Results for test No. 8

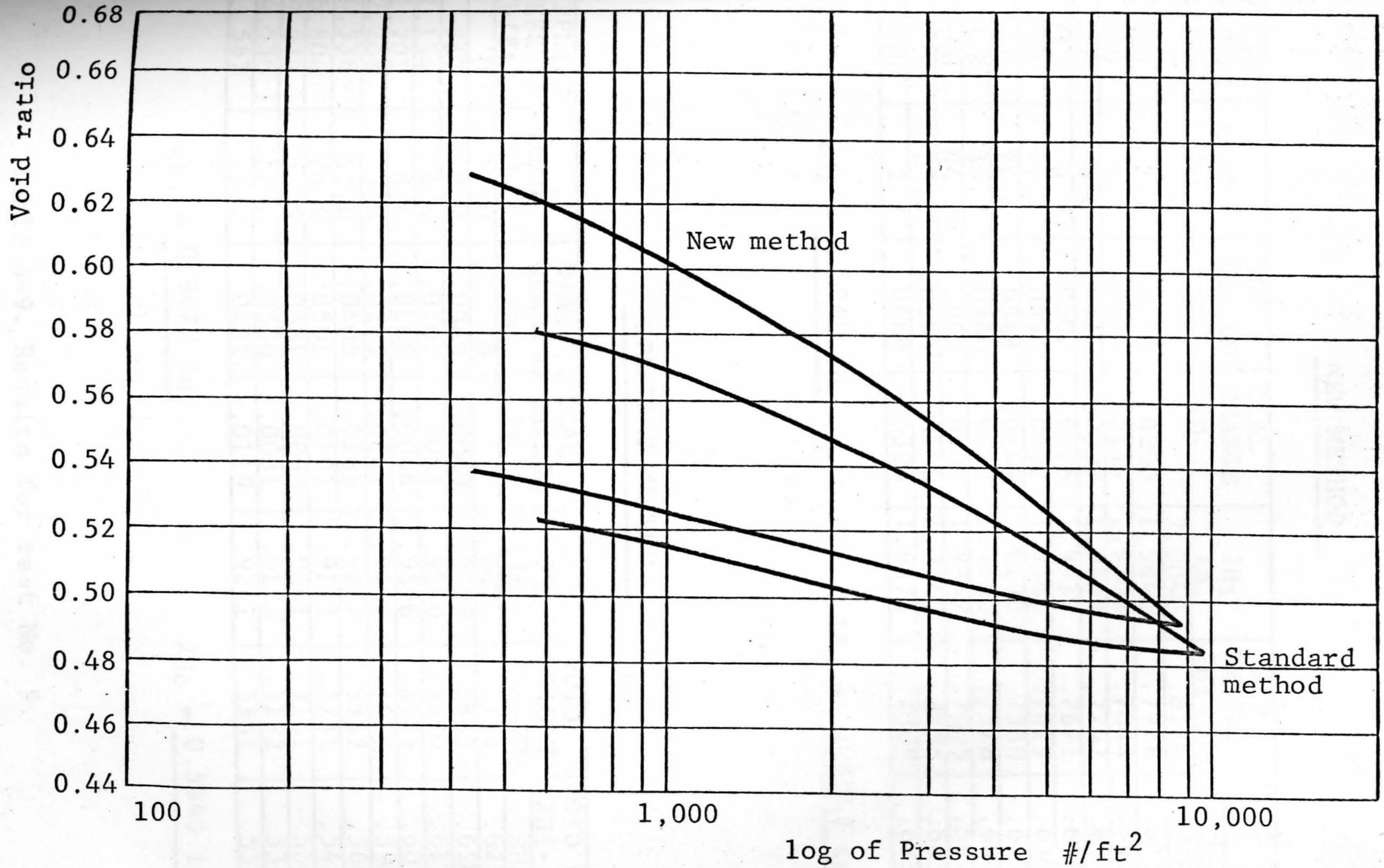


Fig. 4-8. e - log p curves for test No. 8

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		2.0040	.8178	.6894
428	2.2	.0200	.0200	1.9840	.7978	.6726
856	4.4	.0334	.0134	1.9706	.7844	.6613
1,284	6.6	.0431	.0097	1.9609	.7747	.6531
1,712	8.8	.0507	.0076	1.9533	.7671	.6467
3,424	17.6	.0699	.0192	1.9341	.7479	.6305
5,136	26.4	.0848	.0149	1.9192	.7330	.6179
6,848	35.2	.0960	.0112	1.9080	.7218	.6085
8,560	44.0	.1093	.0133	1.8947	.7085	.5973
3,424	17.6	.0993	-.0100	1.9047	.7185	.6057
428	2.2	.0669	-.0324	1.9371	.7509	.6330

$2H_i$ = 2.0040 in. $2H_o$ = 1.1862 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9672	.3732	.6283
567	2.2	.0049	.0049	.9623	.3683	.6200
1,134	4.4	.0102	.0053	.9570	.3630	.6111
1,701	6.6	.0146	.0044	.9526	.3585	.6035
2,268	8.8	.0215	.0069	.9457	.3517	.5921
4,536	17.6	.0400	.0185	.9272	.3332	.5609
6,804	26.4	.0501	.0101	.9171	.3231	.5439
9,072	35.2	.0581	.0080	.9091	.3151	.5305
4,536	17.6	.0550	-.0031	.9122	.3182	.5357
567	2.2	.0431	-.0119	.9241	.3301	.5557

$2H_i$ = 0.9672 in. $2H_o$ = 0.5940 in.

Table 4-9. Results for test No. 9

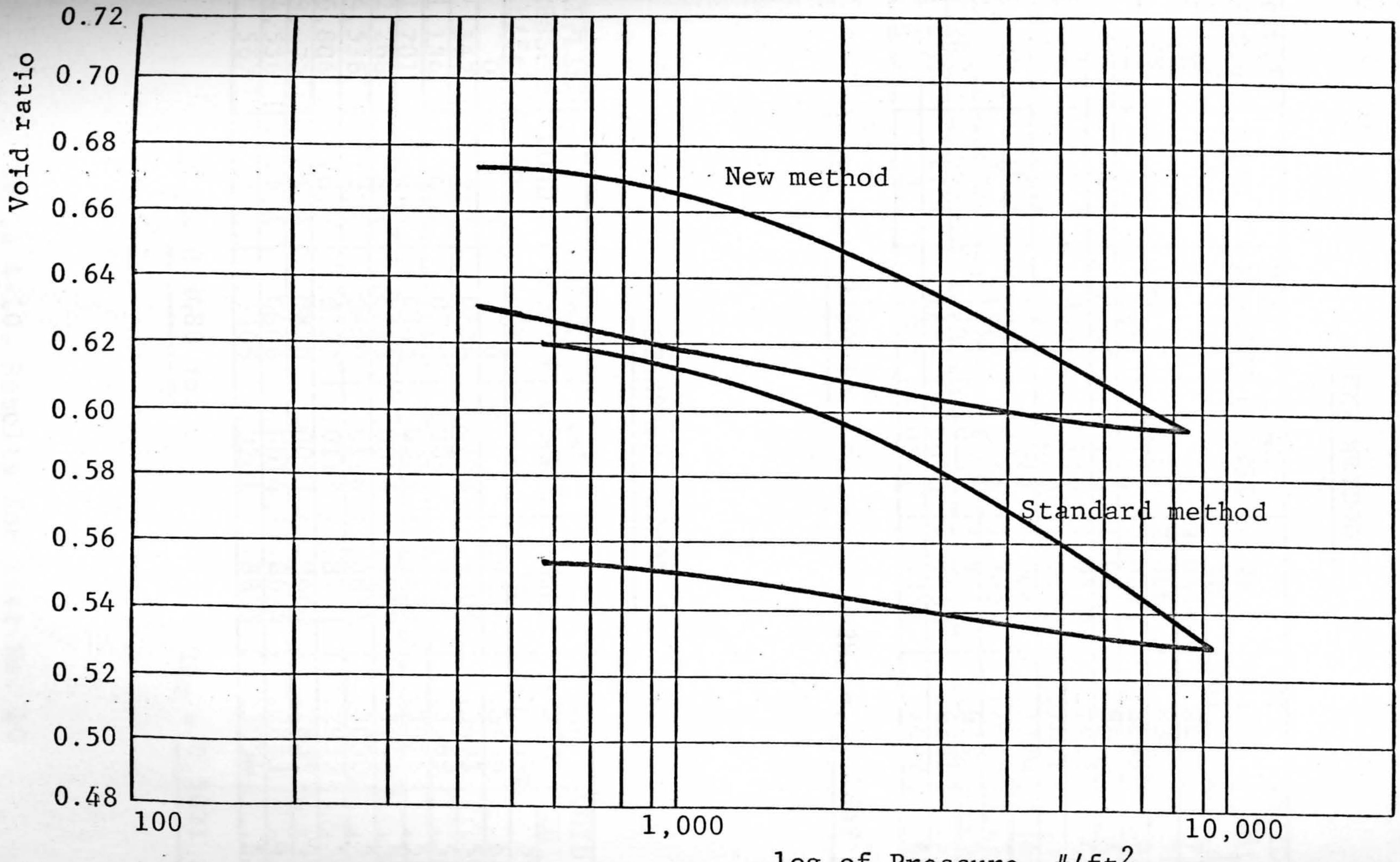


Fig. 4-9. e - log p curves for test No. 9

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		2.0354	.8495	.7163
428	2.2	.0187	.0187	2.0167	.8308	.7006
856	4.4	.0331	.0144	2.0023	.8164	.6884
1,284	6.6	.0466	.0135	1.9888	.8029	.6770
1,712	8.8	.0582	.0116	1.9772	.7913	.6673
3,424	17.6	.1024	.0442	1.9330	.7471	.6300
5,136	26.4	.1271	.0247	1.9083	.7224	.6092
6,848	35.2	.1467	.0196	1.8887	.7028	.5926
8,560	44.0	.1744	.0277	1.8610	.6751	.5693
3,424	17.6	.1694	-.0050	1.8660	.6801	.5735
428	2.2	.1003	-.0691	1.9351	.7492	.6318

$2H_i$ = 2.0354 in. $2H_o$ = 1.1859 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9480	.3659	.6286
567	2.2	.0176	.0176	.9304	.3483	.5984
1,134	4.4	.0270	.0094	.9210	.3389	.5822
1,701	6.6	.0345	.0075	.9135	.3314	.5693
2,268	8.8	.0413	.0068	.9067	.3246	.5576
4,536	17.6	.0609	.0196	.8871	.3050	.5240
6,804	26.4	.0727	.0118	.8753	.2932	.5037
9,072	35.2	.0820	.0093	.8660	.2839	.4877
4,536	17.6	.0806	-.0014	.8674	.2853	.4901
567	2.2	.0605	-.0201	.8875	.3054	.5247

$2H_i$ = 0.9480 in. $2H_o$ = 0.5821 in.

Table 4-10. Results for test No. 10

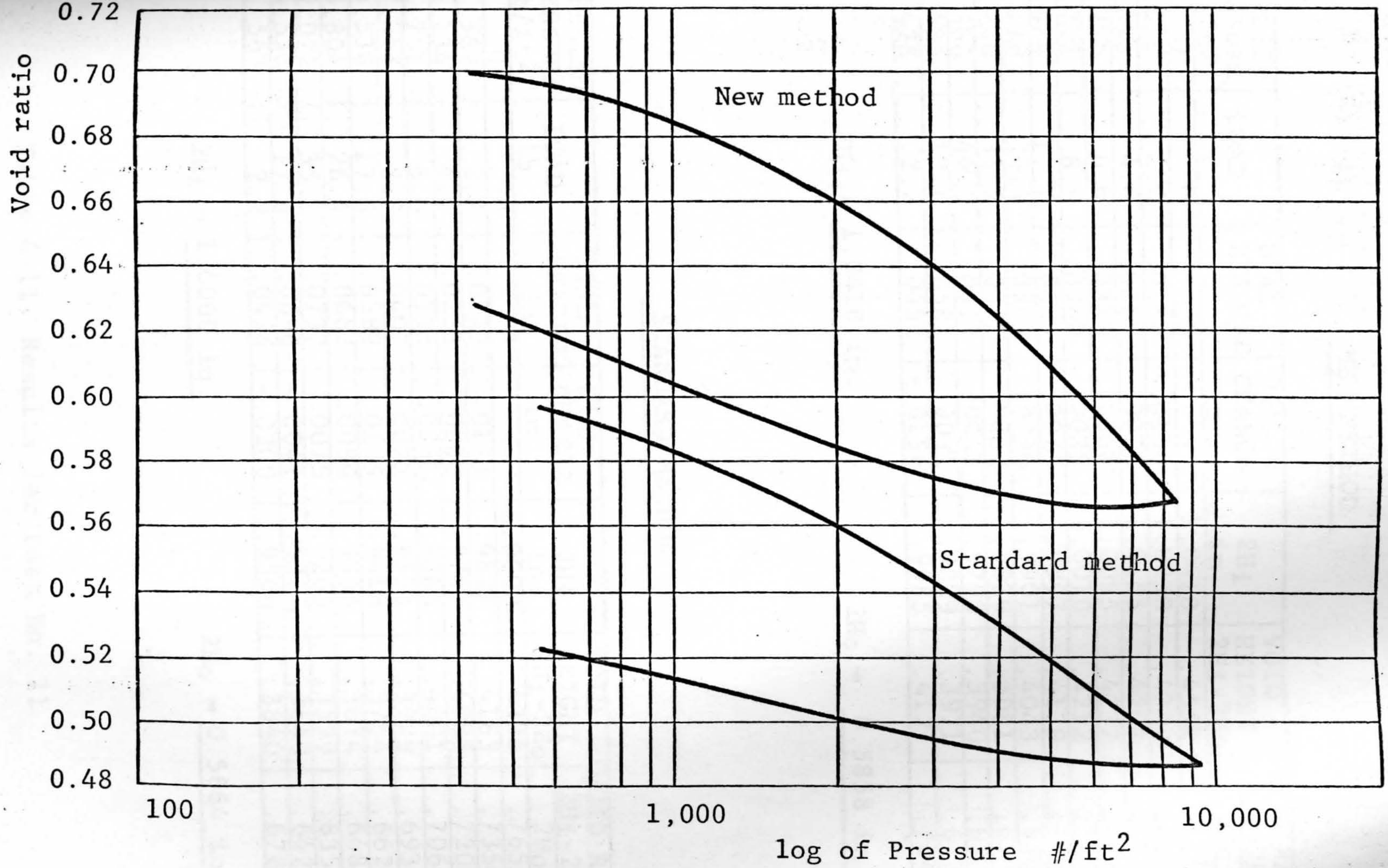


Fig 4-10. e - log p curves for test No. 10

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0410	.4552	.7771
428	2.2	.0180	.0180	1.0230	.4372	.7463
856	4.4	.0255	.0075	1.0155	.4297	.7335
1,284	6.6	.0315	.0060	1.0095	.4237	.7233
1,712	8.8	.0340	.0025	1.0070	.4212	.7190
3,424	17.6	.0413	.0073	.9997	.4139	.7066
5,136	26.4	.0509	.0096	.9901	.4043	.6902
6,848	35.2	.0571	.0062	.9839	.3981	.6796
8,560	44.0	.0612	.0041	.9798	.3940	.6726
3,424	17.6	.0581	-.0031	.9829	.3971	.6779
428	2.2	.0452	-.0156	.9985	.4127	.7045

$2H_i$ = 1.0410 in. $2H_o$ = 0.5858 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0000	.4336	.7655
567	2.2	.0173	.0173	.9827	.4163	.7350
1,134	4.4	.0253	.0080	.9747	.4083	.7209
1,701	6.6	.0335	.0082	.9665	.4001	.7064
2,268	8.8	.0407	.0072	.9593	.3929	.6937
4,536	17.6	.0582	.0175	.9418	.3754	.6628
6,804	26.4	.0662	.0080	.9338	.3674	.6487
9,072	35.2	.0737	.0075	.9263	.3599	.6354
4,536	17.6	.0696	-.0041	.9304	.3640	.6427
567	2.2	.0496	-.0200	.9504	.3840	.6780

$2H_i$ = 1.0000 in. $2H_o$ = 0.5664 in.

Table 4-11. Results for test No. 11

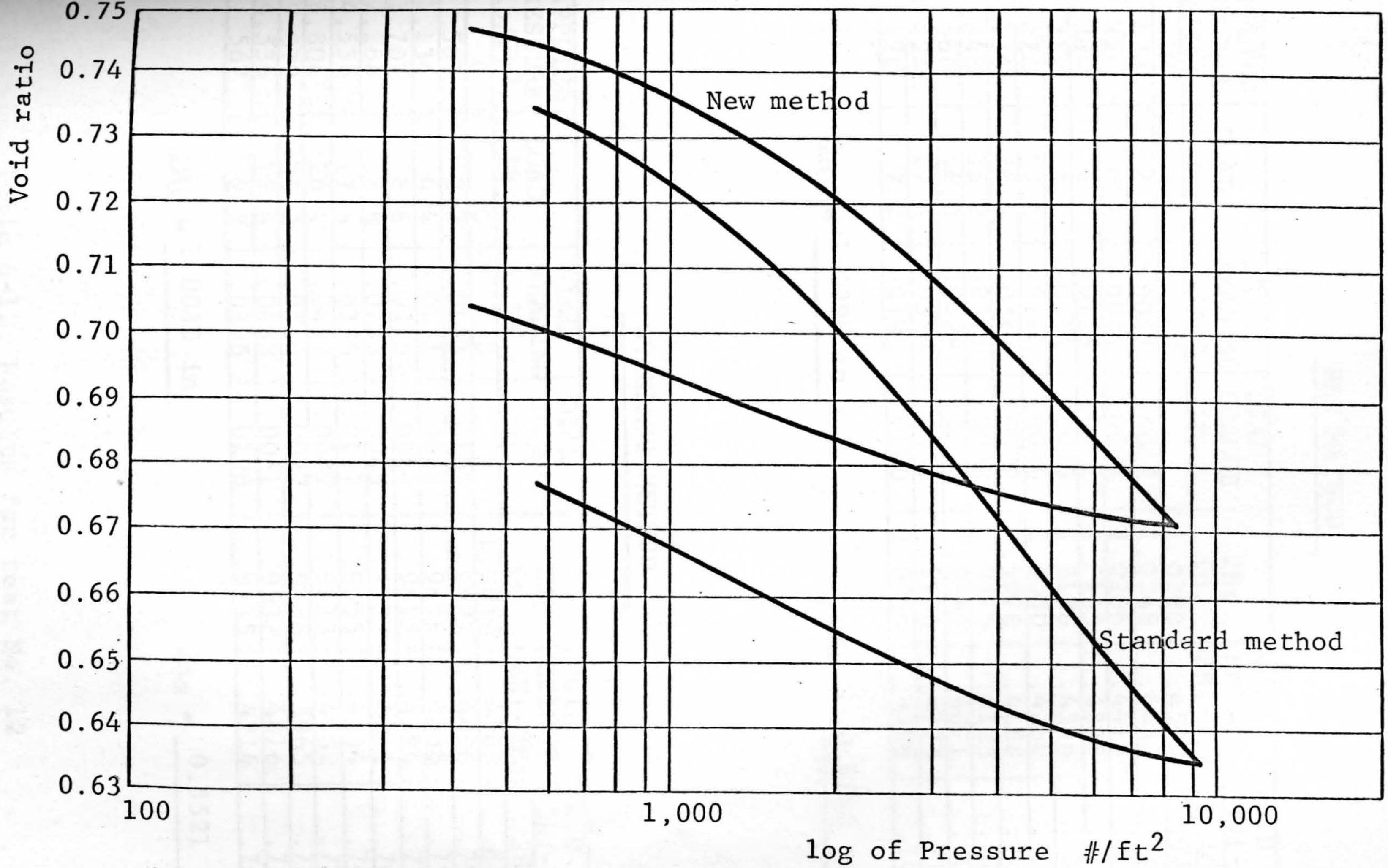


Fig. 4-11. e - log p curves for test No. 11

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0290	.4500	.7772
428	2.2	.0042	.0042	1.0248	.4458	.7699
856	4.4	.0084	.0042	1.0206	.4416	.7627
1,284	6.6	.0125	.0041	1.0165	.4375	.7556
1,712	8.8	.0154	.0029	1.0136	.4346	.7506
3,424	17.6	.0240	.0086	1.0050	.4260	.7358
5,136	26.4	.0312	.0072	.9978	.4188	.7233
6,848	35.2	.0375	.0063	.9915	.4125	.7124
8,560	44.0	.0429	.0054	.9861	.4071	.7031
3,424	17.6	.0374	-.0055	.9916	.4126	.7126
428	2.2	.0194	-.0180	1.0096	.4306	.7437

$2H_i$ = 1.0290 in. $2H_o$ = 0.5790 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0030	.4509	.8167
567	2.2	.0103	.0103	.9927	.4406	.7980
1,134	4.4	.0171	.0068	.9859	.4338	.7957
1,701	6.6	.0215	.0044	.9815	.4294	.7778
2,268	8.8	.0258	.0043	.9772	.4251	.7699
4,536	17.6	.0375	.0117	.9655	.4134	.7488
6,804	26.4	.0457	.0082	.9573	.4052	.7339
9,072	35.2	.0544	.0087	.9486	.3965	.7182
4,536	17.6	.0493	-.0051	.9537	.4016	.7274
567	2.2	.0295	-.0198	.9735	.4214	.7633

$2H_i$ = 1.0030 in. $2H_o$ = 0.5521 in.

Table 4-12. Results for test No. 12

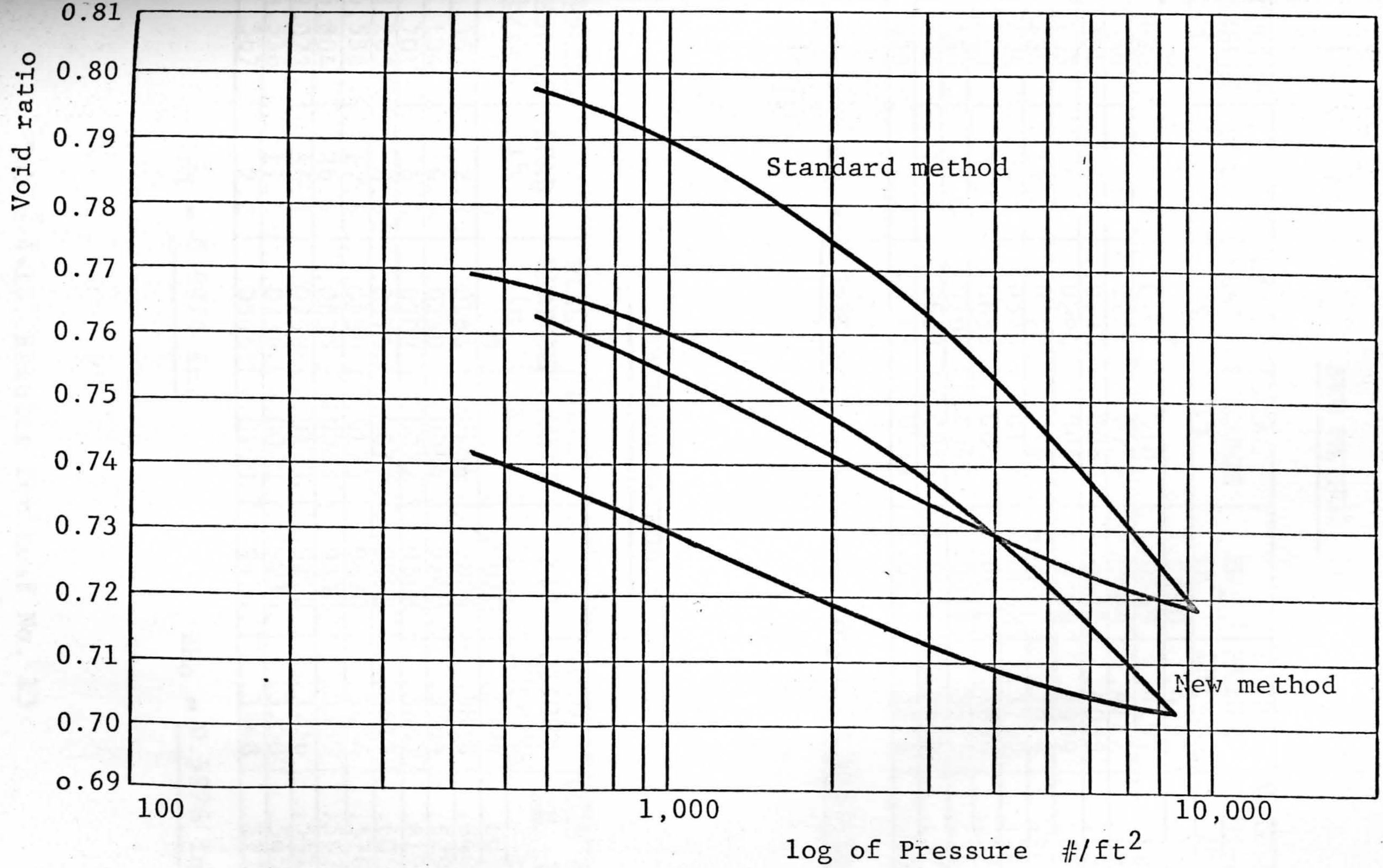


Fig. 4-12. e - log p curves for test No. 12

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i - 2H_o$	VOID RATIO $\frac{2H_i - 2H_o}{2H_o}$
0	0	0		1.0660	.4293	.6743
428	2.2	.0101	.0101	1.0559	.4192	.6584
856	4.4	.0176	.0075	1.0484	.4117	.6466
1,284	6.6	.0238	.0062	1.0422	.4055	.6369
1,712	8.8	.0294	.0056	1.0366	.3999	.6281
3,424	17.6	.0459	.0165	1.0201	.3834	.6022
5,136	26.4	.0573	.0114	1.0087	.3720	.5843
6,848	35.2	.0659	.0086	1.0001	.3634	.5708
8,560	44.0	.0708	.0049	.9952	.3585	.5631
3,424	17.6	.0653	-.0055	1.0007	.3640	.5717
428	2.2	.0419	-.0234	1.0241	.3874	.6084

$2H_i$ = 1.0660 in. $2H_o$ = 0.6367 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i - 2H_o$	VOID RATIO $\frac{2H_i - 2H_o}{2H_o}$
0	0	0		.9970	.3994	.6683
567	2.2	.0166	.0166	.9804	.3828	.6406
1,134	4.4	.0230	.0064	.9740	.3764	.6299
1,701	6.6	.0302	.0072	.9668	.3692	.6178
2,268	8.8	.0375	.0073	.9595	.3619	.6056
4,536	17.6	.0550	.0175	.9420	.3444	.5763
6,804	26.4	.0635	.0085	.9335	.3359	.5621
9,072	35.2	.0758	.0123	.9212	.3236	.5415
4,536	17.6	.0736	-.0022	.9234	.3258	.5452
567	2.2	.0546	-.0190	.9424	.3448	.5770

$2H_i$ = 0.9970 in. $2H_o$ = 0.5976 in.

Table 4-13, Results for test No. 13

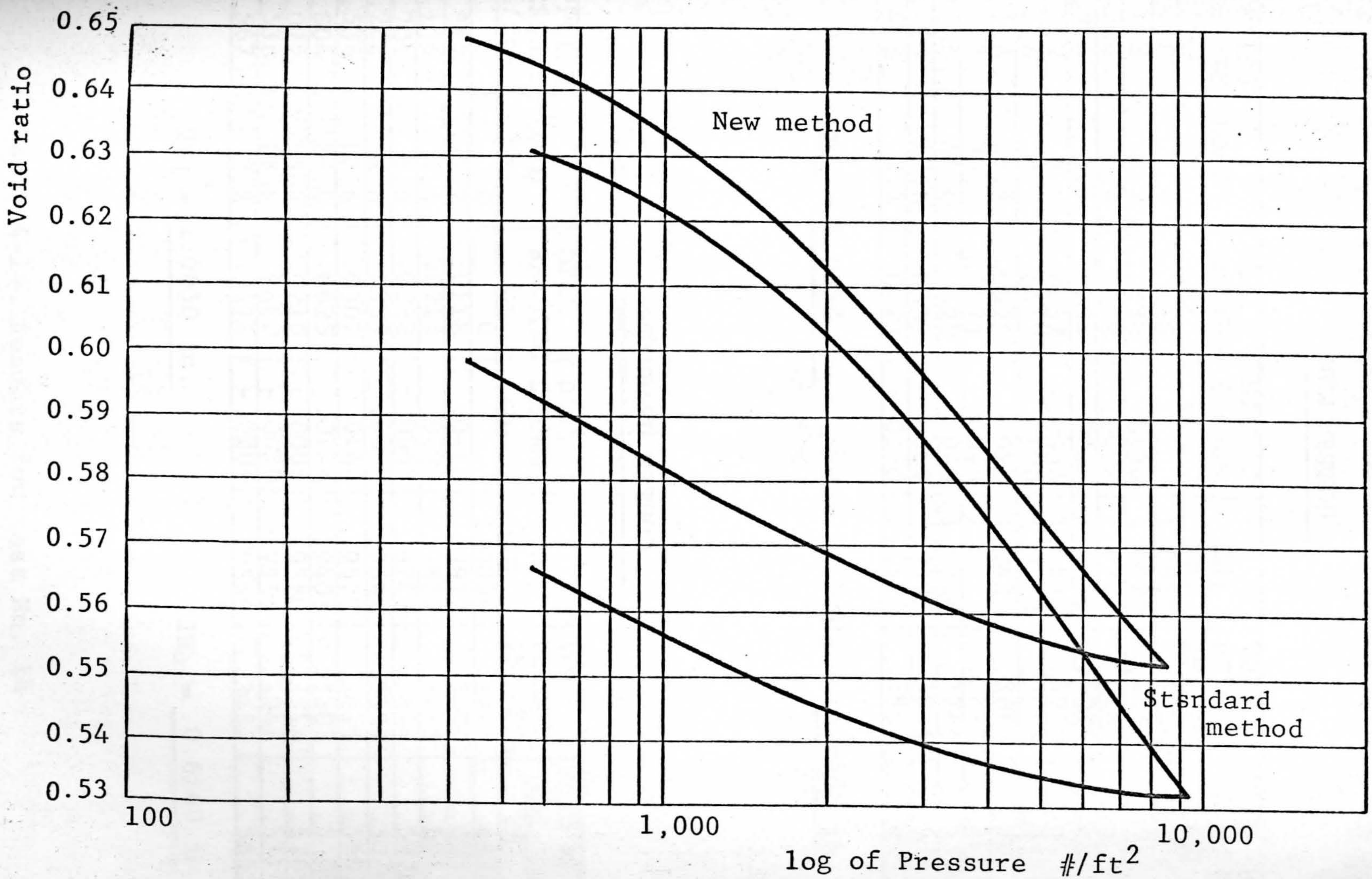


Fig. 4-13. e - log p curves for test No. 13

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0950	.4388	.6687
428	2.2	.0130	.0130	1.0820	.4258	.6530
856	4.4	.0228	.0098	1.0722	.4160	.6340
1,284	6.6	.0294	.0066	1.0656	.4094	.6239
1,712	8.8	.0352	.0058	1.0598	.4036	.6151
3,424	17.6	.0472	.0120	1.0478	.3916	.5968
5,136	26.4	.0525	.0053	1.0425	.3863	.5887
6,848	35.2	.0573	.0048	1.0377	.3815	.5814
8,560	44.0	.0622	.0049	1.0328	.3766	.5739
3,424	17.6	.0553	-.0069	1.0397	.3835	.5844
428	2.2	.0423	-.0130	1.0527	.3965	.6042

$2H_i$ = 1.0950 in. $2H_o$ = 0.6562 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0030	.3926	.6432
567	2.2	.0118	.0118	.9912	.3808	.6239
1,134	4.4	.0213	.0095	.9817	.3713	.6083
1,701	6.6	.0282	.0069	.9748	.3644	.5970
2,268	8.8	.0339	.0057	.9691	.3587	.5876
4,536	17.6	.0507	.0168	.9523	.3419	.5601
6,804	26.4	.0625	.0118	.9405	.3301	.5408
9,072	35.2	.0710	.0085	.9320	.3216	.5269
4,536	17.6	.0680	-.0030	.9350	.3246	.5318
567	2.2	.0474	-.0206	.9556	.3452	.5655

$2H_i$ = 1.0030 in. $2H_o$ = 0.6140 in.

Table 4-14. Results for test No. 14

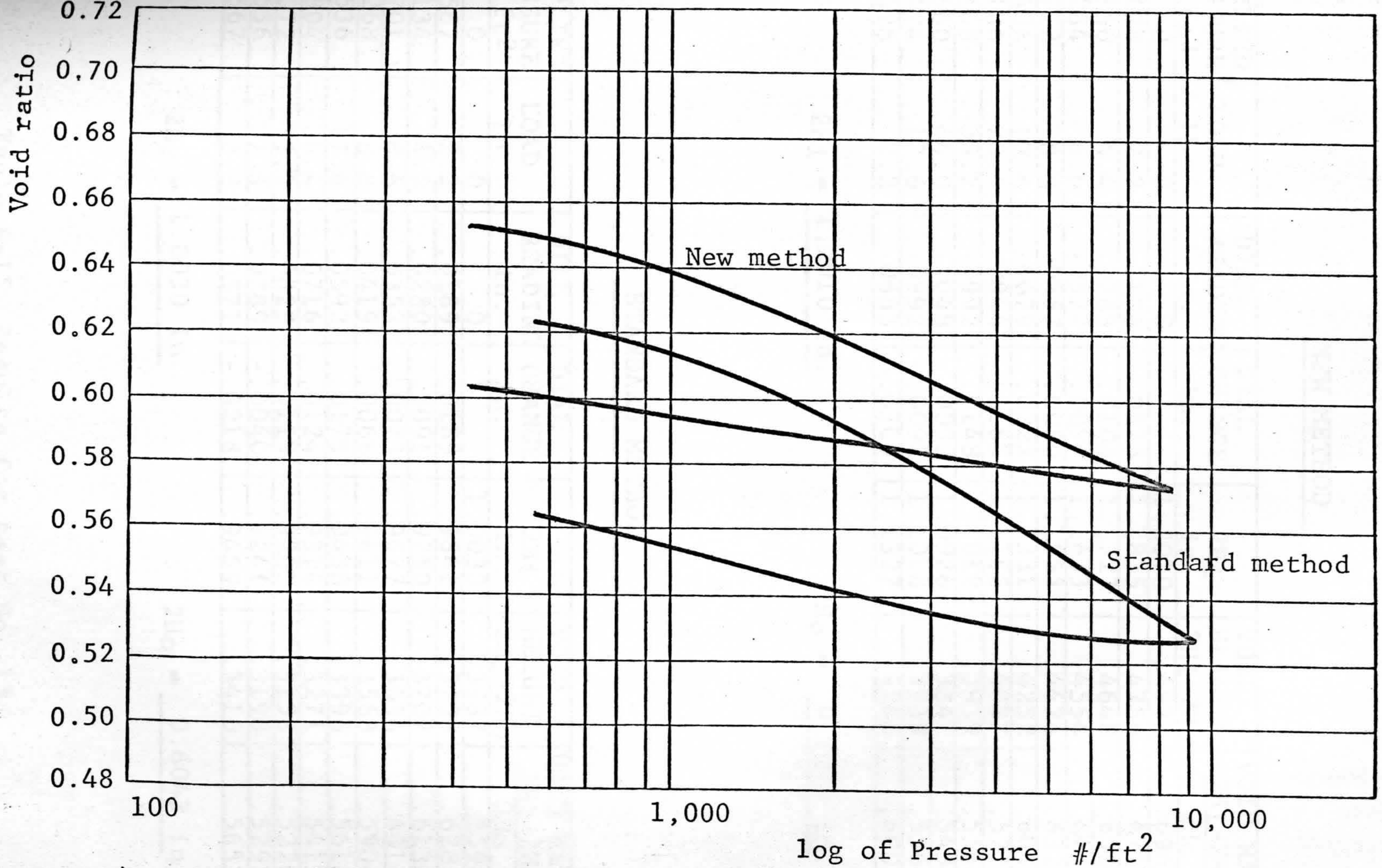


Fig. 4-14. e - log p curves for test No. 14

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.2010	.4931	.6966
428	2.2	.0175	.0175	1.1835	.4756	.6718
856	4.4	.0283	.0108	1.1727	.4648	.6566
1,284	6.6	.0377	.0094	1.1633	.4554	.6433
1,712	8.8	.0457	.0080	1.1553	.4474	.6320
3,424	17.6	.0679	.0222	1.1331	.4252	.6006
5,136	26.4	.0844	.0165	1.1166	.4087	.5773
6,848	35.2	.0992	.0148	1.1018	.3939	.5564
8,560	44.0	.1048	.0092	1.0926	.3847	.5434
3,424	17.6	.0992	-.0092	1.1018	.3939	.5564
428	2.2	.0683	-.0309	1.1327	.4248	.6001

$2H_i$ = 1.2010 in. $2H_o$ = 0.7079 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0030	.3987	.6598
567	2.2	.0189	.0189	.9841	.3798	.6285
1,134	4.4	.0280	.0091	.9750	.3707	.6134
1,701	6.6	.0353	.0073	.9677	.3634	.6014
2,268	8.8	.0419	.0066	.9611	.3568	.5904
4,536	17.6	.0592	.0173	.9438	.3395	.5618
6,804	26.4	.0716	.0124	.9314	.3271	.5413
9,072	35.2	.0829	.0113	.9201	.3158	.5226
4,536	17.6	.0789	-.0040	.9241	.3198	.5292
567	2.2	.0571	-.0218	.9459	.3416	.5653

$2H_i$ = 1.0030 in. $2H_o$ = 0.6043 in.

Table 4-15. Results for test No. 15

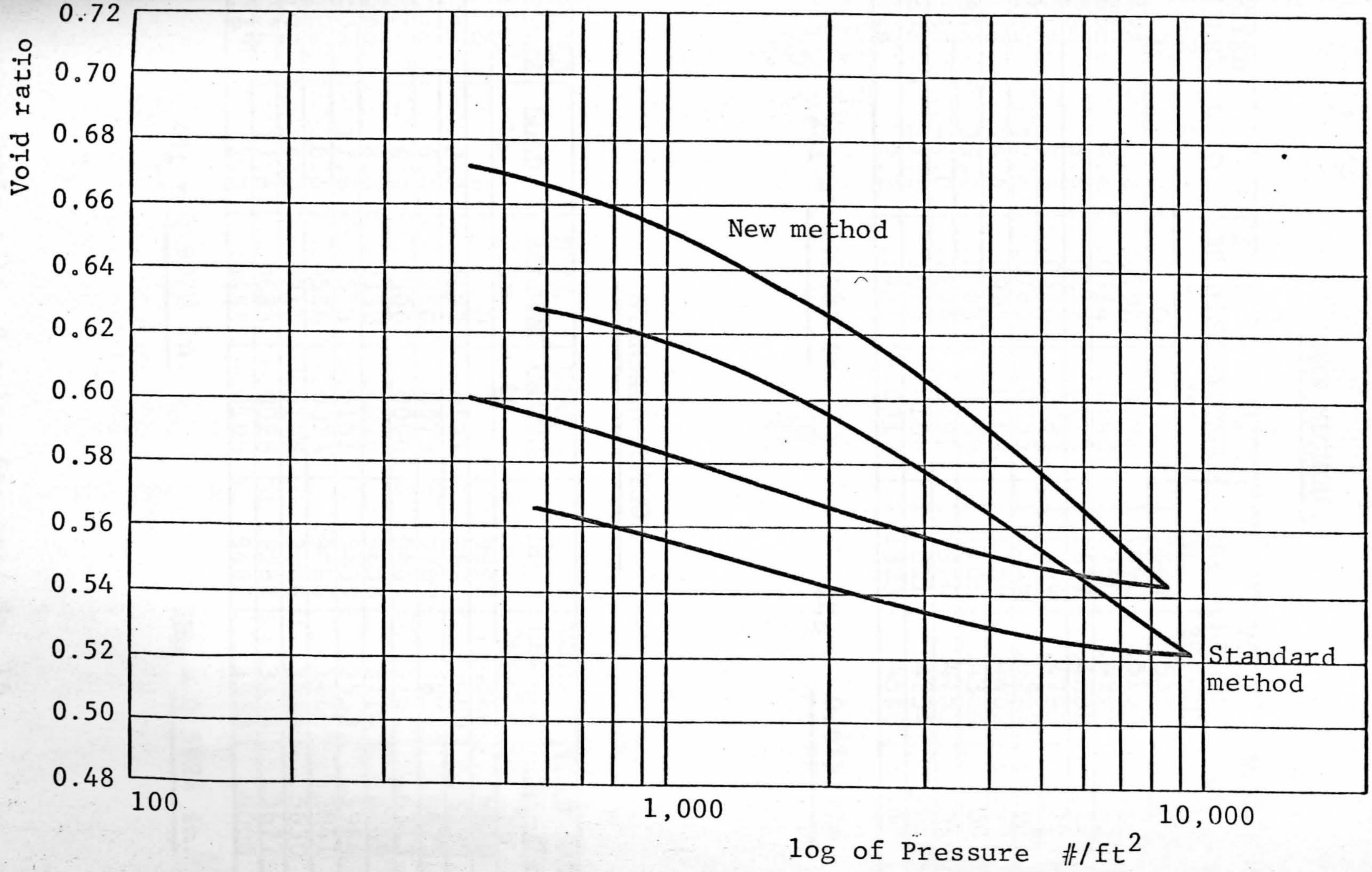


Fig. 4-15. e - log p curves for test No. 15

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.2084	.5327	.7884
428	2.2	.0192	.0192	1.1892	.5135	.7599
856	4.4	.0334	.0142	1.1750	.4993	.7389
1,284	6.6	.0447	.0113	1.1637	.4880	.7222
1,712	8.8	.0530	.0083	1.1554	.4797	.7099
3,424	17.6	.0765	.0235	1.1319	.4562	.6752
5,136	26.4	.0925	.0160	1.1159	.4402	.6515
6,848	35.2	.0983	.0058	1.1101	.4344	.6429
8,560	44.0	.1174	.0191	1.0910	.4153	.6146
3,424	17.6	.1073	-.0101	1.1011	.4254	.6296
428	2.2	.0761	-.0312	1.1323	.4566	.6757

$2H_i$ = 1.2084 in. $2H_o$ = 0.6757 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9970	.4046	.6830
567	2.2	.0137	.0137	.9833	.3909	.6599
1,134	4.4	.0210	.0073	.9760	.3836	.6475
1,701	6.6	.0306	.0096	.9664	.3740	.6313
2,268	8.8	.0332	.0026	.9638	.3714	.6269
4,536	17.6	.0513	.0181	.9457	.3533	.5964
6,804	26.4	.0586	.0073	.9384	.3460	.5841
9,072	35.2	.0711	.0125	.9259	.3335	.5630
4,536	17.6	.0683	-.0028	.9287	.3363	.5677
567	2.2	.0488	-.0195	.9482	.3558	.6006

$2H_i$ = 0.9970 in. $2H_o$ = 0.5924 in.

Table 4-16. Results for test No. 16

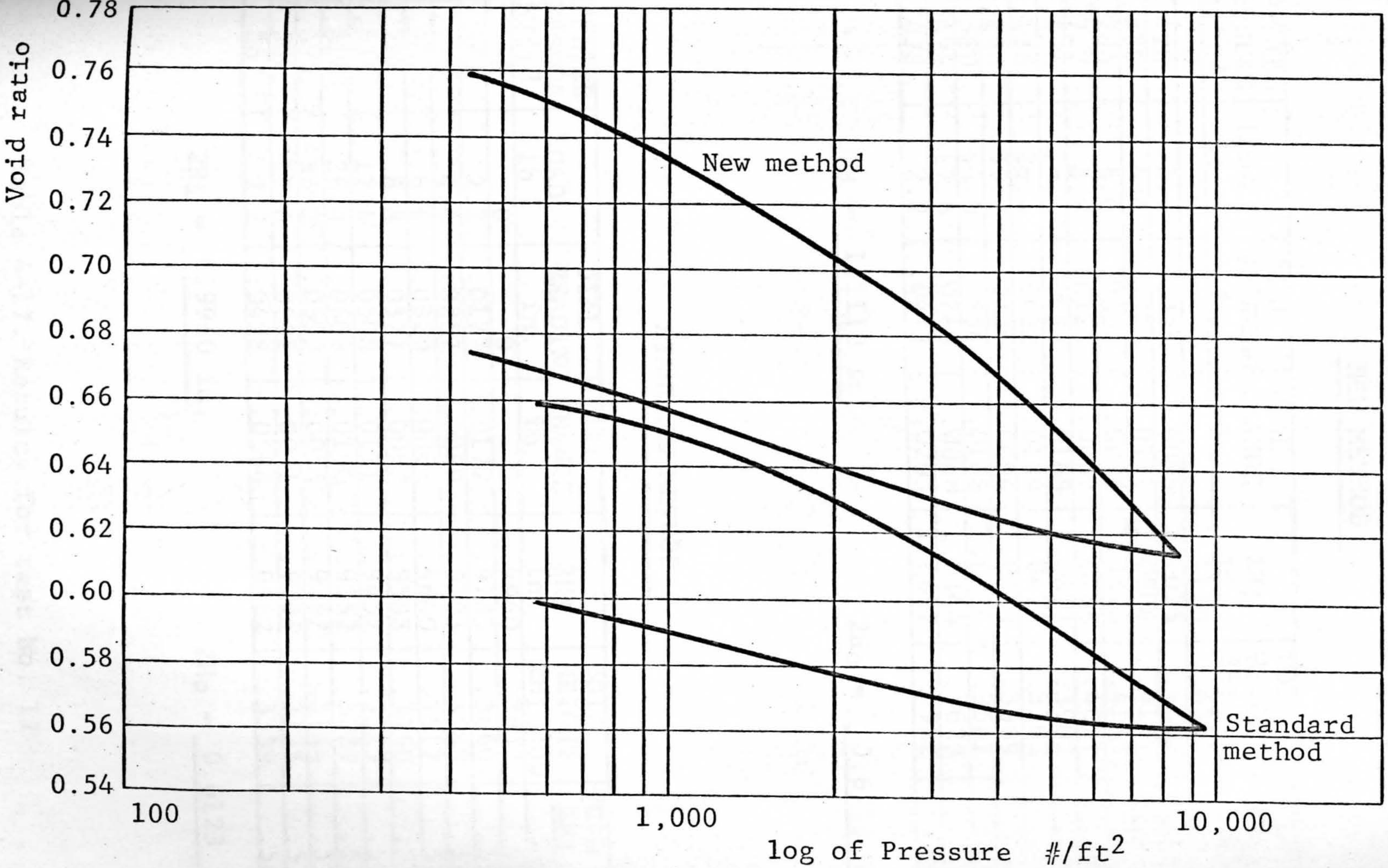


Fig. 4-16. e - log p curves for test No. 16

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.1110	.4395	.6545
428	2.2	.0095	.0095	1.1015	.4300	.6404
856	4.4	.0226	.0131	1.0884	.4169	.6208
1,284	6.6	.0262	.0036	1.0848	.4133	.6155
1,712	8.8	.0324	.0062	1.0786	.4071	.6063
3,424	17.6	.0493	.0169	1.0617	.3902	.5811
5,136	26.4	.0632	.0139	1.0478	.3763	.5604
6,848	35.2	.0726	.0094	1.0384	.3669	.5464
8,560	44.0	.0834	.0108	1.0276	.3561	.5303
3,424	17.6	.0766	-.0068	1.0344	.3629	.5404
428	2.2	.0473	-.0293	1.0637	.3922	.5841

$2H_i$ = 1.1110 in. $2H_o$ = 0.6715 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		.9960	.3837	.6267
567	2.2	.0177	.0177	.9783	.3660	.5977
1,134	4.4	.0257	.0080	.9703	.3580	.5847
1,701	6.6	.0320	.0063	.9640	.3517	.5744
2,268	8.8	.0377	.0057	.9583	.3460	.5563
4,536	17.6	.0508	.0131	.9452	.3329	.5437
6,804	26.4	.0608	.0100	.9352	.3229	.5274
9,072	35.2	.0693	.0085	.9267	.3144	.5135
4,536	17.6	.0662	-.0031	.9298	.3175	.5185
567	2.2	.0498	-.0164	.9462	.3339	.5453

$2H_i$ = 0.9960 in. $2H_o$ = 0.6123 in.

Table 4-17. Results for test No. 17

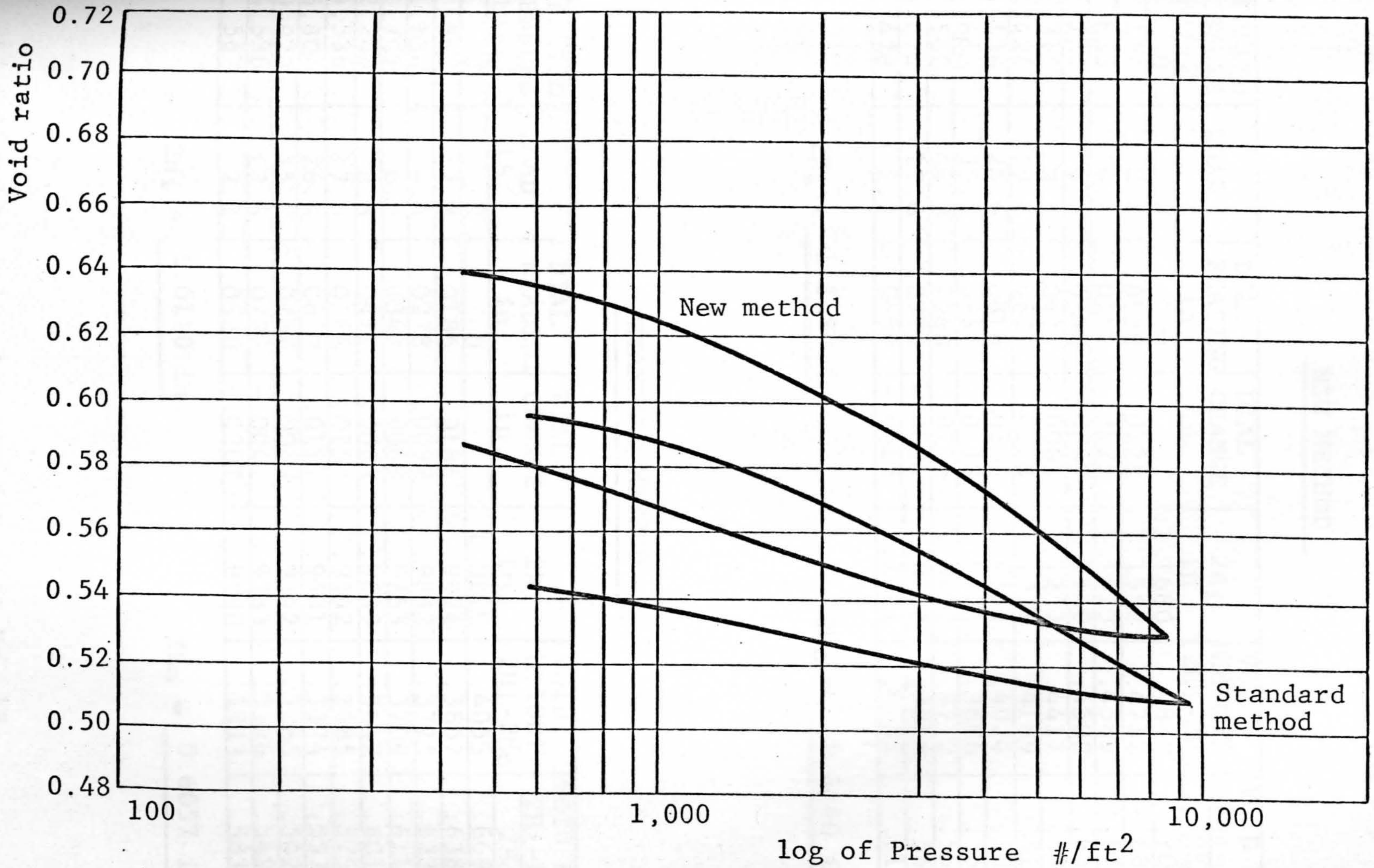


Fig. 4-17. e - log p curves for test No. 17

NEW METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.1680	.4840	.7076
428	2.2	.0128	.0128	1.1552	.4712	.6889
856	4.4	.0251	.0123	1.1429	.4589	.6709
1,284	6.6	.0340	.0089	1.1340	.4500	.6579
1,712	8.8	.0423	.0083	1.1257	.4417	.6458
3,424	17.6	.0651	.0228	1.1029	.4189	.6124
5,136	26.4	.0816	.0165	1.0864	.4024	.5883
6,848	35.2	.0954	.0138	1.0726	.3886	.5681
8,560	44.0	.1068	.0114	1.0612	.3772	.5515
3,424	17.6	.0974	-.0094	1.0706	.3866	.5652
428	2.2	.0659	-.0315	1.1021	.4181	.6113

$2H_i$ = 1.1680 in. $2H_o$ = 0.6840 in.

STANDARD METHOD

APPLIED PRESSURE lb/ft^2	LOAD lb	DIAL READING in.	DIAL CHANGE in.	$2H_i$ in.	VOID HEIGHT $2H_i-2H_o$	VOID RATIO $\frac{2H_i-2H_o}{2H_o}$
0	0	0		1.0110	.4053	.6691
567	2.2	.0186	.0186	.9924	.3867	.6384
1,134	4.4	.0278	.0092	.9832	.3775	.6232
1,701	6.6	.0347	.0069	.9763	.3706	.6119
2,268	8.8	.0411	.0064	.9699	.3642	.6013
4,536	17.6	.0568	.0157	.9542	.3485	.5754
6,804	26.4	.0679	.0111	.9431	.3374	.5570
9,072	35.2	.0768	.0089	.9342	.3285	.5423
4,536	17.6	.0727	-.0041	.9383	.3326	.5491
567	2.2	.0520	-.0207	.9590	.3533	.5833

$2H_i$ = 1.0110 in. $2H_o$ = 0.6057 in.

Table 4-18. Results for test No. 18

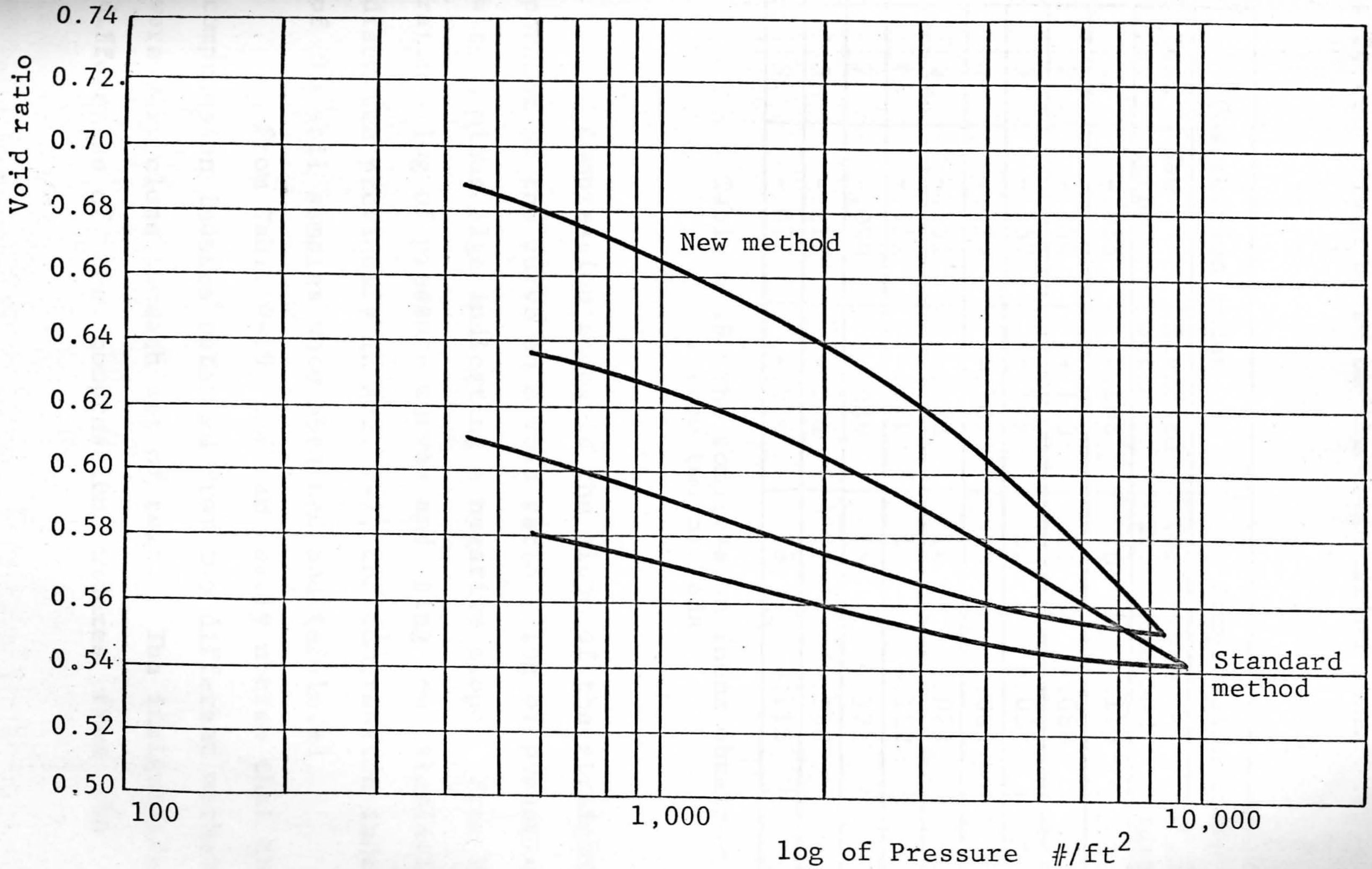


Fig. 4-18. e - log p curves for test No. 18

4-2. Comparison based on the compression index

Compression index			Compression index		
Test No.	New method	Standard method	Test No.	New method	Standard method
1	-.085	-.108	10	-.119	-.116
2	-.144	-.110	11	-.068	-.088
3	-.157	-.129	12	-.059	-.069
4	-.129	-.102	13	-.085	-.083
5	-.151	-.128	14	-.073	-.080
6	-.111	-.111	15	-.112	-.091
7	-.099	-.089	16	-.121	-.091
8	-.112	-.084	17	-.096	-.078
9	-.077	-.076	18	-.113	-.077

Table 4-19 The compression index obtained from two methods.

Compression index is the slope of the straight line portion of the curve on a void ratio - log of pressure plot, with a minus sign indicating a negative slope. From void ratio - log of pressure curves and using the calculations discussed previously in Art. 2-7, the compression indexes of the soil samples were obtained and tabulated.

From Table 4-19, one can easily notice that the compression indexes obtained from two different methods were very close in each set of test. The insignificant differences of the compression indexes from the two methods not only means the void ratio - log of pressure curves have the same slope but also explains why these curves are parallel to each other in every set of test. Most importantly, one can also note that from 18 sets of e - log p curves, excluding test No. 12, all of the curves obtained by the new method fell above those from the standard method.

During the cyclic loading, "if the pressure is removed, the soil will tend to rebound or swell back somewhat, but not to the original shape; a permanent deformation will have resulted. Then, if the load is re-applied, the cycle will form a hysteresis loop, with initial segment of the new curve being somewhat parallel to the first."[4]

From the discussions presented above, it can be logically concluded that the void ratio - log of pressure curves obtained by the standard method are actually one cycle lower than those obtained by the new method.

4-3. Comparison based on the preconsolidation pressure

As mentioned in Art. 2-8, the preconsolidation pressure is the pressure to which a soil has been subjected at some time previously, before the compression test. The preconsolidation pressure could have been induced in a number of ways, for example, previous building loads, soil over-burdens, or glacial over-burdens.

Using Casagrande's method discussed previously, the preconsolidation pressures of 18 sets of comparative

Preconsolidation pressure #/ft^2			Preconsolidation pressure #/ft^2		
Test No.	New method	Standard method	Test No.	New method	Standard method
1	2,400	2,280	10	2,800	2,600
2	2,700	2,450	11	2,600	2,300
3	2,550	2,300	12	2,500	2,700
4	2,750	2,700	13	2,300	2,270
5	2,650	2,260	14	2,490	2,450
6	2,600	2,500	15	2,700	2,500
7	2,550	2,500	16	2,700	2,400
8	2,750	2,570	17	2,720	2,400
9	2,530	2,480	18	2,700	2,450

Table 4-20 The preconsolidation pressure obtained from two methods

tests were obtained from the corresponding e - log p curves. As shown above in Table 4-20, one notes that for each case the preconsolidation pressure obtained from the two methods were very close. Also, the new method consistently yielded higher values for the preconsolidation pressures, except test No. 12, than did the standard method with the differences ranging from 30 psf for test No. 13 to 390 psf for test No. 5.

Since "remolding of the clay specimen - even if only partial, for instance that due to a slight disturbance during sampling - is liable to lower the preconsolidation pressure because of the resulting decreased curvature of the void ratio-pressure curve."[5] The higher values for the preconsolidation pressures obtained by the new method did indicate that the new method caused less disturbance than did the standard method.

One notes that even under the best of sampling procedures, the loads on the sample would be removed, thereby permitting the specimen to rebound. Hence, theoretically speaking, if a soil sample could be maintained "undisturbed" during the handling and the preparation, the first cycle of loading in the laboratory actually corresponds to a re-loading cycle for a soil which has been subjected to a precompression or preconsolidation load. That is, the first cycle of laboratory loading, in fact, represents the equivalent of a re-compression curve (i.e. second cycle) of an in-situ situation. Also, from the previous observation, it has already been shown that the void ratio - log of pressure curves obtained by the standard method are actually one cycle lower than those obtained by the new method.

Therefore, as concluded previously in Art. 4-2 and discussed above, another improtant result is obtained: The standard consolidation method actually measures the third consolidation cycle while the new method measures the second consolidation cycle.

4-4. Comparison based on the void ratio

Void ratio under pressure of 1,000 psf			Void ratio under pressure of 1,000 psf		
Test No.	New method	Standard method	Test No.	New method	Standard method
1	.6935	.6375	10	.6875	.5850
2	.7200	.6425	11	.7370	.7235
3	.7255	.6965	12	.7600	.7895
4	.6920	.6535	13	.6435	.6315
5	.8045	.7200	14	.6390	.6150
6	.8125	.7050	15	.6545	.6175
7	.5725	.5150	16	.7350	.6525
8	.6015	.5700	17	.6260	.5900
9	.6675	.6150	18	.6650	.6275

Table 4-21 The void ratio obtained from two methods. under the pressure of 1,000 psf

The values of the void ratios of two methods under a constant pressure of 1,000 psf were determined directly from the void ratio - log of pressure diagrams and the results were tabulated above. From Table 4-21, it is noted that for each comparative test set, except No. 12, the void ratio obtained from the new method was higher than that from the standard method under a constant pressure. As discussed previously in Art. 2-5, under cyclic loading and for a constant pressure (e.g. 1,000 psf), the more frequently a soil sample is reloaded the lower its void ratio will be. Therefore, once again it is proved that the first consolidation cycle obtained by the standard method is actually the re-loading cycle of the first void ratio - log of pressure curve obtained by the new method.

4-5. Comparison based on the settlement

As soil settlement is discussed, the relative amount of deformation of the solids is negligible; hence, it is the change in the void volume that is assumed to be the cause of the settlement. In Eq.

$$S = \Delta H = \frac{H}{1 + e_i} C_c \log \frac{p}{p_o} \qquad (4\text{-}1)^4$$

where

S = Settlement

H = Height of the soil sample

e_i = Initial void ratio

C_c = Compression index

p = Any applied pressure

p_o = Pressure at which void ratio is known.

Since for both consolidation methods, H, p, p_o obviously are constant under any given situation; the Eq. (4-1) can be rearranged as

$$S = \frac{C_c}{1 + e_i} K \qquad (4\text{-}2)$$

where

$$K = H \log \frac{p}{p_o}$$

The values of $\frac{C_c}{1 + e_i}$ from two methods were calculated and shown in Table 4-22. Once again the results obtained

C_c / $(1+e_i)$			C_c / $(1+e_i)$		
Test No.	New method	Standard method	Test No.	New method	Standard method
1	.049	.063	10	.069	.071
2	.081	.065	11	.038	.049
3	.087	.074	12	.033	.037
4	.074	.060	13	.051	.050
5	.081	.081	14	.044	.048
6	.060	.063	15	.066	.055
7	.059	.056	16	.067	.054
8	.067	.052	17	.058	.048
9	.046	.046	18	.066	.046

Table 4-22 The value of C_c / $(1+e_i)$ for both methods.

by the two methods were noted to be nearly identical.

From previous discussion, one notes that both values of C_c and e_i for the new method are higher than those for the standard method and the differences were cancelled out in the calculation of C_c / $(1+e_i)$ value. Therefore, it can be logically concluded that the close values of the soil settlements obtained by the two methods can be attributed to the consistency of the differences in the C_c and e_i values.

CHAPTER V

CONCLUSION

From the results obtained and presented in the previous chapters, it can be concluded that:

(1) The compression index from both methods is nearly identical.

(2) The new method consistently yields higher values for the preconsolidation pressure and the void ratio than does the standard method.

(3) The values for the estimate of total settlement are nearly identical for both methods.

(4) The new method offers lateral restriction more effectively.

(5) The new method permits a very accurate measurement of the sample thickness.

(6) The mew method eliminates the necessity for extracting, trimming and fitting, thus saving a considerable amount of time, thereby making the preparation of the soil sample more expedient.

(7) Since the sample is tested within the tube, the new method permits the consolidation of coarser silts, which frequently fall apart during the extraction and trimming process.

(8) The new method permits the testing of thicker (longer) samples.

For the new consolidation method, some precautionary measures should be adhered to during the processes of sampling and handling to guarantee accuracy of the test results. These include:

(1) A continuous and steady pushing force is required during the sampling to ensure good adhesion between the specimen and the wall of the tube.

(2) Special care should be taken during the transportation to avoid any impact or jarring forces which could result in disturbing the sample.

(3) During the process of specimen preparation, a suitable sharp cutting tool (i.e. a band saw with a vise in good working order) is required to prevent unnecessary vibrations.

In view of the advantages of accuracy, efficiency and expediency that have been demonstrated through the tests in this study, there is strong evidence that the New Consolidation Method developed by Dr. John N. Cernica is superior to the Standard Consolidation Method.

REFERENCES

[1]A. R. Jumikis, Soil Mechanics. New Jersey: D. Van Nostrand Company, Inc., 1962, p. 359

[2]T. William Lambe, Soil Testing for Engineers. New York: John Wiley & Sons, Inc., 1951, p. 74

[3]A. R. Jumikis, Soil Mechanics. New Jersey: D. Van Nostrand Company, Inc., 1962, p. 373

[4]J. N. Cernica, Elements of Soil Mechanics and Foundations. Submitted for Publication, pp. 244-245

[5]G. P. Tschebotarioff, Soil Mechanics, Foundations, and Earth Structures. New York: McGraw-Hill Book Company, Inc. 1951, p. 104