ANALYSIS OF FOUNDATION STRESSES AND SETTLEMENTS AT THE HAYDEN LIBRARY

by

HARL PRESLAR ALDRICH, Jr.

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Signature of Author . . . .
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Certified by . . . .
Thesis Supervisor

. . . .
Chairman, Departmental Committee on Graduate Students

Abstract
of a thesis entitled

ANALYSIS OF FOUNDATION STRESSES AND
SETTLEMENTS AT THE HAYDEN LIBRARY

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Submitted for the degree of Sc.D. in the
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While the Charles Hayden Memorial Library was under construction in 1948, thirty two settlement observation points were placed throughout the basement, and ten piezometers were installed in the foundation clay stratum. The primary objective of this investigation is to correlate observed settlement and pore water pressure data with results of laboratory consolidation tests and existing theories for computing settlements and foundation stresses.

Since the piezometer installation is the first of its type in the foundation soil of a building, emphasis is given to the use of pore pressure data for determining the stress-strain time properties of the clay stratum during loading. A viscous flow consolidation analogy model has been used to study the speed at which the settlement and pore pressure dissipation have occurred. Results of this investigation show that the compressibility of the Library foundation clay is one fifth to one tenth that which would be estimated from consolidation tests on small samples. Consolidation of the clay stratum and hence settlement of the building have progressed fifteen to eighteen times as fast as laboratory tests would indicate. These discrepancies are largely due to the Library's small net load, to radial flow of water in the clay stratum, and to misinterpretation of consolidation test data. The importance of the net building load and a comparison between the Library settlement and that of the main M.I.T. buildings are discussed in some detail.

A second part of this investigation describes the results of a unique group of laboratory consolidation tests on large and small clay samples with pore water pressures measured at the center of the specimen during the test. Results of these tests on undisturbed Boston blue clay, have thrown new light on present consolidation theories. In addition they have shown that the true coefficient of consolidation generally lies midway between values given by the log time and square root of time fitting methods. A part of the discrepancy between the two fitting methods has been found to be a result of sidewall friction.

It was concluded that the settlement and pore pressure predictions based on theory are very undependable when the net pressure increase is as small as occurred in the case studied; a heavier building might have shown an action in better agreement with theory and permitted more definite conclusions.

Cambridge, Massachusetts
May 10, 1951

Professor Joseph S. Newell
Secretary of the Faculty
Massachusetts Institute of Technology
Cambridge, Massachusetts

Dear Sir:

In partial fulfillment of the requirements for the degree of Doctor of Science in Civil Engineering, this thesis entitled "ANALYSIS OF FOUNDATION STRESSES AND SETTLEMENTS AT THE HAYDEN LIBRARY" is submitted.

Respectfully yours,

Harl P. Aldrich, Jr.

TO LOIS

for her untiring patience
and assistance during the course
of this investigation.

ACKNOWLEDGEMENTS

The writer wishes to acknowledge the help and guidance given by Professor D. W. Taylor throughout this investigation.

He is also indebted to Mr. C. M. Stahle and Mr. F. F. Flanders for their valuable assistance in constructing and assembling apparatus for this research. Mr. T. Jordan of the Department of Buildings and Power at M.I.T. has kindly given the writer free access to Library construction data, which has been greatly appreciated.

Finally, the writer wishes to thank other members of the Soil Mechanics staff, especially Mr. W. R. Sutherland, who have assisted in taking settlement observations, piezometer readings, and data during the laboratory testing program.

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I

INTRODUCTION

An important phase of the foundation investigation for any large building is a settlement analysis, which is based on building loads and characteristics of the foundation soil. Before the final selection of a substructure is made the amount and rate of settlement for preliminary designs should be estimated. In order to estimate the settlement of a building founded above a thick stratum of clay, for example, data are required from laboratory tests on undisturbed samples of the clay. A common procedure is to place a small sample in a cylindrical container and apply increments of load, recording for each increment the amount of compression and the time rate at which it occurs. The degree to which this test approximates the mechanical properties of the clay in the field is probably the most important single consideration in any settlement analysis.

To ascertain the validity of laboratory data in representing field performance and to check existing methods for computing settlements, observation points are occasionally placed throughout the building for periodic settlement readings. Although such observations have given valuable data, a great many factors still cannot be explained by present theories and hypotheses.

When construction was started at the Charles Hayden Memorial Library at M.I.T., 32 settlement observation points were placed in the basement of the building. In addition, 10 water pressure measuring devices (piezometers) were installed in the 90-foot-thick layer of Boston blue clay which underlies the structure. The purpose of this installation, the first of its type in the foundation soil of a building, is to record the water pressure variation in the clay during and following the construction of the building.

According to a commonly accepted theory developed by K. Terzaghi[(16)*], the water in the voids of the foundation clay initially carries the total weight of the building. Piezometers in the clay should show this phenomenon by recording an increase in water pressures as the building is constructed. This excess pressure gradually dissipates as water flows from the voids of the clay, allowing the building to settle. According to the Terzaghi theory this dissipation and settlement follow certain time laws which depend on the physical and mechanical properties of the clay. These properties, in turn, can be predicted from consolidation tests.

A more recent theory[(12)], developed by D. W. Taylor and called Theory B, accounts for certain variables not

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* Numbers in parentheses refer to the Bibliography in Appendix I.

considered in the Terzaghi theory. The possibility of a part of the building load being transferred directly to the intergranular structure of the soil mass without causing an increase in the pore water pressure is recognized. In addition, the theory accounts for plastic resistances during consolidation.

The primary purpose of this investigation is to correlate the piezometer readings and the settlement observations at the Hayden Library with data obtained from consolidation tests and with existing theories and hypotheses for predicting settlements and pore water pressures.

In the past the Terzaghi consolidation theory has been used almost exclusively to interpret and apply results of laboratory consolidation tests. When the piezometer installation at the Hayden Library was planned it was hoped that the record of pore water pressures in the clay during and after construction would give an indication of the validity of Theory B. Unfortunately the installation was not as ideal as expected, for the following reasons:

1. It was not possible to install the piezometers until the foundation for the building had been completed. Therefore, initial water pressures in the clay and the variation in pressure throughout the excavation and early loading period are not known. Since the stress release due to excavation occurred rapidly and its magnitude is easily

estimated, this period would have been ideal to estimate the proportion of the stress release initially registered by the pore water in the foundation clay.

2. Because of changes in the original Library design and the small magnitude of the present book load, the net load (building load minus load release due to excavation) is very nearly zero throughout the clay. For this reason, at least in part, a majority of the settlement and pore pressure dissipation occurred during the relatively slow construction of the building. If any of the load was initially carried by the intergranular structure of the clay, it was masked by the speed at which consolidation took place. Finally, after the piezometers were installed no load of appreciable magnitude was applied suddenly enough to give a measurable sudden increase in pore water pressure.

Even though there may be bonds and plastic resistances not considered in the Terzaghi theory, it is impossible to evaluate their magnitude from piezometer and settlement data at the Library. Thus, Theory B will not be presented in detail in this investigation but qualitative effects of the factors which it recognizes will be mentioned.

The availability of pore pressure data makes this investigation unique among those correlating laboratory data and field data for a building foundation. Because of this, considerable weight will be given to the value and use of piezometer readings in estimating field compressibility and consolidation characteristics. These characteristics will be compared with results of laboratory consolidation tests on samples of the undisturbed foundation clay.

The four distinct types of data which have been gathered in this investigation are discussed in Chapters II through V. In Chapter II the physical and mechanical characteristics of the clay are presented. These are determined from exploratory borings and from laboratory consolidation tests on undisturbed samples. Data on the sequence of construction activities and records of the magnitude of building loads and their distribution are given in Chapter III. Chapter IV is a resumé of the procedure for taking settlement observations and a presentation of the Library settlement data. The piezometer readings and installation procedure are discussed in Chapter V. Considerable emphasis is given to this chapter since the piezometer installation is of primary importance in this investigation. VI and VII are devoted to a review of the standard settlement analysis procedure and a fairly thorough presentation of the many important variables affecting the settlement analysis, in particular as they may affect the Library correlations which follow in VIII and IX.

Chapter VIII is an analysis of the magnitude of the Library settlement and a determination of the compressibility characteristics of the clay. Chapter IX deals specifically with a correlation of the time-settlement characteristics with other Library data and a determination of the consolidation properties of the foundation soil using piezometer data.

These chapters comprise Part I of this thesis.

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During the course of this investigation there was considerable question regarding the small magnitude of the pore pressures at the Hayden Library. The piezometers recorded a maximum excess pressure head of about 2 feet while the average building load applied to the top of the clay during the same period was about 0.35 tons per square foot which corresponds to over 11 feet of head.

For some time it was thought that intergranular bonds recognized in Theory B were accounting for a large portion of the building load. Because of this, the writer undertook a program of laboratory consolidation tests with attending pore pressure measurements in order to develop a further understanding of bond phenomena in undisturbed soil. Up until this time pore pressure research reported by D. W. Taylor(12) and others(13)(14) had been conducted primarily on remolded soil.

It became apparent, however, that the small piezometer readings at the Library were caused by rapid consolidation rather than by bond phenomena. Therefore the objectives of this special research were altered somewhat to a study of the coefficient of consolidation utilizing pore pressure data.

Part II of this thesis is a presentation in some detail of the apparatus and techniques developed for obtaining pore pressures during consolidation. (They differ from those used in previous research.) In addition, preliminary results and observations are given for a group of 6 tests with pore pressure measurements taken at the middle of the sample. Three of these tests were run on samples 9.55 inches in diameter and three are on standard size specimens 4.25 inches in diameter. These preliminary tests did not show the pronounced structural plastic resistance to compression that Professor Taylor found in remolded clay. Nevertheless, some bond did exist in certain loading increments.

Although the results of these tests are not used quantitatively in the analysis of Part I, they are presented with the hope that they will stimulate further interest in pore pressure research, directed especially toward a study of bond and plastic phenomena in undisturbed clay.

PART I

ANALYSIS OF PIEZOMETER READINGS AND SETTLEMENTS

II

FOUNDATION EXPLORATION, SAMPLING AND TESTING PROGRAM

The foundation exploration, sampling and testing investigation for the Hayden Library was carried out during the spring and summer of 1946. A summary of the results of this program is given in a report by W. Enkeboll[1].

A. EXPLORATION AND SAMPLING: Numerous "dry sample" borings were made by B. F. Smith & Company, Inc. of Boston and by the Gow Division of the Raymond Concrete Pile Company of Boston. The boring logs and geologic sections are given on Hayden Library blueprints 1203-F-1 to 1203-F-4 inclusive.

In addition to the "dry sample" borings, one three-inch diameter undisturbed boring, called boring No. 11, was made by the Gow Company in May, 1946 using the fixed piston type sampler designed by M. J. Hvorslev and described in Reference (2). Samples from this boring were used for the laboratory testing program. The foundation plan of Figure 1* indicates the location of boring No. 11 in relation to the library proper and the piezometers.

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* All figures and tables are presented consecutively at the end of this thesis.

The geologic section in Figure 2 has been prepared to show the average conditions revealed by the boring data. Borings indicated fairly similar soil profiles above the clay but there was considerable variation below the clay. Most of the borings showed soft clay to a depth of about 125 feet, Figure 2, with either hard clay sand and gravel, hardpan, or rock indicated at this depth. However, the undisturbed boring log, Figure 3, shows soft clay to a depth of 108 feet. It is important to note at this time that the deepest piezometers (depth approximately 125 feet) do not indicate a completely free drainage surface at this elevation. The piezometer curves described in V show a small response at this level to applied building loads. Nevertheless, in this investigation the blue clay will be considered to have free drainage at its top and bottom surfaces and its depth will be assumed equal to 90 feet.

B. TESTS ON UNDISTURBED SAMPLES:

General: W. Enkeboll[1] described the quality of the undisturbed samples as good with the exception of one sample which was disturbed, when the sampler struck a rock. It is the writer's impression that the soil tests were carried out with unusual care and that the results are for the most part very good.

A summary of the clay properties as determined by Enkeboll from boring No. 11 is reproduced in Figure 3.

Consolidation Tests: Twelve consolidation tests were run on samples from boring No. 11. These tests were performed on specimens 2.75 inches in diameter and 0.85 inches thick using standard loading procedure, Reference (2), of doubling the previous load at 24-hour intervals. Figures 4a, 4b and 4c are reproductions of Enkeboll's void ratio vs pressure (log scale) curves computed from the consolidation test data. The results of test 11-5-46.6 will not be used since the sample was considerably disturbed.

Table I has been prepared to show a summary of the most important consolidation data which include the initial void ratio, maximum past pressure, coefficient of consolidation and compression index. Notes at the bottom of the table explain the notation used. Further explanation is given in VI.

A study of the summary table shows that the clay stratum may be conveniently divided into two layers called the "upper foundation clay" and the "lower foundation clay." The shaded areas of Figure 5 indicate the range of the coefficient of consolidation and compression index values as a function of pressure for these two layers. These values are typical for Boston blue clay.

Curve 1 of Figure 6 shows an estimate of the present overburden intergranular pressure as a function of depth. Curve 2 is the maximum past pressure determined from consolidation tests using the Casagrande graphical construction[3] and curve 3 is an estimate of the actual preconsolidation pressure. The latter curve is based on the following reasoning. If curve 2 were extrapolated to the bottom of the clay it would indicate a preconsolidation pressure nearly 1 ton per sq ft lower than the overburden pressure, curve 1. The most probable explanation for this phenomenon would be the existence of artesian pressures at the bottom of the clay. This hypothesis has, as a matter of fact, been proposed in several cases where similar phenomena have been found. However, in this case, piezometers at the bottom of the clay show that the pressure there is actually somewhat less than hydrostatic. J. P. Gould[4] found similar conditions at the Logan airport. In this case then the only explanation is that the graphical construction has given values of maximum past pressure which are too small. A minimum value at the bottom of the clay then is represented by the overburden pressure at that point. From this point curve 3 has been drawn whose abscissae are approximately 1 ton per sq ft greater than those of curve 2.

The stiff upper foundation clay is generally believed to be a result of surface drying thousands of years ago when the clay was above water. I. B. Crosby[5] states that "most, if not all, of the clay (Boston Basin) was deposited at the close of this glaciation (the last one)." "At the time of the ice retreat the sea stood a little higher than at present, and then the land rose until sea level was at least 70 feet lower than at present. During this period of lower sea level the clay was eroded and weathered and peat was formed in places. Then the sea level rose, and silt was deposited upon the clay and peat."

Evidence to support the erosion and surface drying as described by Crosby are the gullies, some of which are 50 feet deep, and the fact that the stiff upper clay is often yellow. "The lowest known elevation of the yellow clay, 62 feet below low tide, is in Everett (Mass.) and here the yellow clay has been reported as 50 feet thick."

Accompanying the capillary pressures at the surface of the clay during drying is a consolidation pressure which may be many tons per sq ft. The adjusted maximum past pressure line of Figure 6 then is further evidence of the effect of surface drying in the Boston Basin area. It appears that the effects of surface drying following the glacial retreat extend nearly to the bottom of the clay in the M.I.T. area.

An important contribution of the consolidation test data then is to point out that the clay stratum is in a state of varying precompression in which case the field consolidation follows a recompression curve which involves considerably smaller settlements than if no precompression existed.

III

LIBRARY CONSTRUCTION AND LOADING DATA

A. GENERAL DESCRIPTION OF LIBRARY: The Charles Hayden Memorial Library is a steel frame building supported for the most part by concrete caissons (see Figure 2) which transmit the load to the coarse sand and gravel stratum overlying the Boston blue clay. Creosoted wood piles, driven to the coarse sand, support the exterior stairways, terrace and passageway to Building 2.

Except for a heavy reinforced concrete slab, 3 1/2 feet thick, 26 1/2 feet wide and extending the length of the building as shown in Figure 1, the basement floor consists of a 9 or 10-inch reinforced slab which transfers its loads to the caissons by reinforced concrete beams. Superstructure loads are carried by steel columns directly to the tops of the caissons except in the heavy slab area where each column load is distributed to several caissons.

The Library above the basement level may be conveniently divided into 5 areas as shown in Figure 8. As far as load is concerned the building is essentially U-shaped, the heaviest areas being the north, east and south wings which have 3 or 4 floors.

Voorhees, Walker, Foley and Smith of New York, architects and engineers for the Library, designed its superstructure as lightweight as possible. Most of the steel framing is covered with vermiculite plaster while lightweight concrete (105 pounds per cu.ft.) was used for the floor slabs above the basement. A majority of the interior partitions were built of cinder block.

Moran, Proctor, Freeman and Mueser of New York acted as consulting foundations engineers for the Library, and Thompson-Starrett Company, Inc. was awarded the construction contract.

B. CONSTRUCTION SCHEDULE: Excavation for the library started April 7, 1948. This date will be referred to as the start of construction and most of the time curves in this investigation will use the abscissa "days since the start of construction."

Before the caissons were constructed the excavation was surrounded by well points which lowered the water table 26 feet to the coarse sand and gravel. The well points were functioning for about 4 months. Construction progressed rapidly from the time steel was erected in November and December 1948 to the summer of 1949 when offices in the north wing were occupied. The English and History Library moved in during November 1949 while books in the basement and Dewey Library were brought in the latter part of January and the first of February, 1950.

Figure 7 has been prepared to show a summary of the construction schedule of the Hayden Library. The majority of the information used for this plot has been taken from H. de R. Gibbons'[6] "The Loads Effective in Causing Consolidation at the Hayden Library." For the ordinate the writer has used the average net intensity of load over the area of the Library basement projected to the elevation of the top of the sand. This elevation was selected since all building loads are applied at this level by the caissons. The pressure release due to excavation is the average release over the building area projected to the top of the sand. For this release and for the increase in pressure intensity causing consolidation due to the drawdown, there are effective loads <u>outside</u> the projected area which must be considered when estimating the stress change at a given point in the clay stratum. Figure 7 shows that the average net intensity of load at the top of the clay for the present building is slightly above zero.

The progress photographs in Appendix II are included so the reader may more readily visualize the building layout and construction progress.

C. <u>LOADS</u>: Careful records of the building loads were kept by Gibbons[6] through April, 1949. From that date to the present, T. Jordan of the Department of Buildings and Power at M.I.T. has kindly supplied the writer with data he has not kept himself.

Figure 8 shows the caisson plan and numbering system. Beside each caisson are two numbers - the first is the caisson design load used by the foundation engineers, Moran, Proctor, Freeman and Mueser (from blueprint 1203-F-5, revision 7). The second is an estimate of the existing total dead and live load as of April 1, 1951. This estimate is based on design dead column loads used by Seelye, Stevenson and Value, consulting structural engineers (blueprint 1203-32, revision 12) except in the court area where actual dead loads were computed. Design loads call for future expansion in the court area. To the dead column load the weight of basement slabs, basement walls and other details not included in the Seelye, Stevenson and Value figures has been added. To obtain total loads, existing live loads (books primarily) were added to the dead loads. A more detailed discussion of the load analysis is presented by C. H. Spaulding[7] who worked independently of the writer but used essentially the same procedure.

Appendix III gives all pertinent data for each caisson including bell diameter, thickness of sand, and design and present loads.

IV

SETTLEMENT OBSERVATIONS

A. LOCATION AND DESCRIPTION OF OBSERVATION POINTS: During the early stages of construction on the Charles Hayden Memorial Library, 32 settlement observation points were placed throughout the basement of the building. Figure 9 shows the location plan of the observation points and a detail of their construction. Points on the interior columns are 3/4" d bolts which have been welded to the flange of the steel column. On the exterior walls or pilasters the bolts were cast into the reinforced concrete. In general the face of the bolt is flush with the face of the finished column or wall.

A 1/8" d hole has been drilled in the center of the bolt into which a brass pin is inserted. All elevations of settlement observation points refer to the elevation of the top of the brass pin at a point along the pin either on line with the face of the column or wall, or at the face of the bolt, whichever sticks out further.

Two bolts were welded to each of the 9 interior columns selected for observation points (Hayden Library blueprint 1203-F-14). Settlement readings were taken on both bolts until construction of partitions rendered many bolts inaccessible. At the present time readings are taken only on the more accessible point of each column. The table of Figure 9 indicates which of these two points has been used for the settlement data described later.

In order to obtain settlement readings as soon as possible, temporary reference points, in the form of indentations made with a center punch near the edge of the bolt face, were established before the 1/8" holes were drilled. After the holes were drilled the elevations of the temporary points were tied in with those of the tops of their corresponding pins. Elevations of the tops of the pins were then backfigured and recorded as if the pins had been used from the start.

B. MEASUREMENT OF SETTLEMENTS:

General: The first set of readings on the temporary settlement points was made on December 4, 1948. At this time most of the structural steel was at the site and about half of it had been erected. An engineer's level was used, first to carry elevations into the basement of the building and then to establish elevations of the center-punch indentations. Beginning with the March 17, 1950 readings the aqualevel, which is described in a later section, has been used to determine the difference in elevation of the tops of the pins once the correct elevation of one pin has been established with the level.

<u>Methods and Techniques</u>: USCGS elevations are carried from the M.I.T. Benchmark (El. +9.551 ft.) between Building 1 and Memorial Drive to a temporary benchmark which has been established near the Library at Building 2. From this temporary benchmark, elevations are carried into the basement of the Library using the doorway and staircase near the exhibition room in the west wing. Backsights and foresights were estimated to the nearest 0.001 foot on a rod graduated to 0.01 foot and reruns were made until elevations were checked to within 0.002 or 0.003 feet.

To obtain the elevation of the center-punch indentation on the bolt face a special rod was built as shown in Figure 10a. When taking a reading the rod was plumbed by eye with the point of the pin in the center-punch hole. This rod was not satisfactory, however, because extremely large errors resulted when the rod was tilted only slightly. As a result it was necessary to take several readings in order to obtain a satisfactory elevation.

Another special rod, Figure 10b, was built early in 1949 to be used with the engineer's level to obtain the elevation of the brass pin. This rod gave more satisfactory readings than did the previous one since it was easier to read and could be plumbed easily against the face of the column or wall.

The present procedure used to obtain a set of settlement readings involves first rechecking the elevation of the temporary benchmark at Building 2 using an engineer's level and a Boston rod with target and level bubble. After this elevation has been established to within 0.002 feet, elevations are carried to settlement point 12 using the level and Boston rod. For the final foresight to point 12 the triangular rod of Figure 10b is used. The difference in elevation between this point and the remaining settlement points in the basement of the Library is determined by means of the aqualevel.

<u>The Aqualevel</u>: In the latter stages of construction it became exceptionally tedious to obtain settlement readings using the engineer's level because of obstructions in the Library basement. In addition, it became evident that more precise observations were desirable. For these reasons, a precision water level device, which the writer calls an aqualevel, was built in February, 1950. The aqualevel, built by C. M. Stahle of the M.I.T. Soils Laboratory, is based on a design originally presented by K. Terzaghi[8]. It is a differential leveling device based on the fundamental principle of the tendency of connected bodies of water to seek the same elevation.

One unit of the aqualevel, Figure 11, is hung on the pin at one observation point; the other, which is connected to the first by a length of heavy rubber tubing, is hung on an adjacent point. After the two valves shown in the figure are opened, each unit is leveled by means of a small circular level bubble and several minutes are allowed for adjustment to be reached. A pointed brass rack which is attached to a vernier is brought into contact with the water surface. The rack is raised and lowered by a small pinion gear which is turned by a thumbscrew. Contact with the water surface is noted easily when a meniscus forms suddenly on the point of the brass rack. Readings at both units are taken to the nearest 0.01 inches and the difference computed. The units are then reversed and a second difference is determined. An average of the two differences indicates the true difference in elevation between the settlement points. The writer has had no difficulty in closing a circuit of 6 or 8 settlement points to within 0.02 inches.

A noted earlier, each unit of the aqualevel is provided with a circular level bubble to assure the operator that the unit is truly vertical. A thumbscrew immediately above the level bubble is used to level the instrument in a vertical plane perpendicular to the column or wall. At several observation points the leveling device cannot be hung from the pin because of obstructions in which case the aqualevel may be mounted above the pin by using the knife-edge and thumbscrew provided at the bottom of the unit.

C. <u>SETTLEMENT READINGS</u>: Settlement observations were taken frequently during the early stages of construction in order to obtain enough points to define the settlement curves which were changing rapidly. In addition, the accuracy of the initial readings was low for reasons already discussed.

A summary of the settlement readings which are based on the USCGS elevation of the M.I.T. Benchmark of +9.551 feet is given in Appendix IV. Contours of the Library settlement to date are shown in Figure 12. These contours are based on settlements determined by drawing a smooth line through the time-settlement curve of each observation point. The initial elevation of the points has been estimated by extrapolating the smooth curve back to December 1, 1948.

V

PIEZOMETER INSTALLATION AND READINGS

A. INTRODUCTION:

General Background: It is common for settlement observation points to be placed in a building for the purpose of checking predicted settlements. Many papers have been written on such installations and vast amounts of valuable data have been collected and analyzed. On the other hand, the writer believes that the piezometer (water pressure measuring device) installation at the Hayden Library is the first of its type in a buried clay stratum below a building.

The value of water pressure measurements in earth materials has been recognized for many years and installations in earth dams and below masonry dams are not uncommon. A. Casagrande[9] and J. P. Gould[4] write about an installation at the Logan International Airport in Boston where a wide area of clay fill was involved.

Knowing surface load intensities, which may be caused by building loads, and using any of several load distribution theories the soils engineer is able to estimate the vertical stress intensity at any point in the foundation soil below a building. On the basis of the commonly accepted Terzaghi theory of consolidation for fine grained cohesive soils, the water in the voids of the clay initially carries the stress increment caused by the building loads. In other words, the pressure in the pore water at a given point in the clay is expected to increase an amount equal to the stress increment for that point. As consolidation of the clay takes place and the building settles, this excess water pressure dissipates as water flows both vertically to drainage surfaces, and horizontally to cause a swell in the clay surrounding that immediately below the building. The rate at which this excess pressure dissipates depends on properties of the soil, notably the thickness and its coefficient of consolidation.

To date no check has been made on the magnitude of this excess pressure caused by building loads and the rate at which it dissipates. For this purpose then, ten piezometers in two groups of five were installed at the Charles Hayden Memorial Library in October of 1948.

Description of Piezometer: The piezometer used for this installation is a non-metallic type developed by A. Casagrande[9]. It consists of a 1 1/2 inch diameter Norton porous stone cylinder, 2 feet long with a 1/4 inch wall, surrounded by a pocket of sand and connected to a 1/2 inch diameter Saran tube standpipe which extends to the basement of the Library. Figure 13 shows a section through the piezometer assembly drawn to a greatly exaggerated horizontal scale. As shown in the figure the porous stone cylinder, called the porous point, is effectively sealed off from water inside the steel casing by layers of bentonite which have been compacted above the porous point between the Saran tube and casing.

Readings of the elevation of the water surface in the Saran tube standpipe are taken by means of an electrical sounding device. The water pressure at the piezometer location is given then by the vertical distance between the water surface in the standpipe and the mean elevation of the sand pocket.

Location of Piezometers: Figure 1 shows the location in plan of both groups of piezometers with respect to the Library as a whole. Figure 14 gives the numbering system adopted and a more detailed location of each piezometer, and Table II shows a summary table of elevations of the important components of each piezometer. Piezometers A-1 and B-1 are located in the coarse sand above the clay while A-5 and B-5 were placed as deep as possible, presumably at the bottom of the clay. Figure 14 and Table II do not agree with Hayden Library blueprint 1203-F-14 since the drawing was not revised to show the existing location and numbering of the piezometers.

B. PIEZOMETER INSTALLATION:

General Information: Ten piezometers were installed at the Hayden Library during October of 1948 by the Boston Gow Division of the Raymond Concrete Pile Company of New York. Funds for the installation were made available by M.I.T. at the request of D. W. Taylor and with the cooperation of W. H. Mueser of the firm, Moran, Proctor, Freeman and Mueser, Consulting Engineers. The writer supervised the installation and personally assisted the placing of each of the 10 porous points.

Installation Procedure: A complete description of the piezometer and installation procedure is given in Appendix V. This appendix, based primarily on the installation procedure presented by A. Casagrande[9], incorporates the writer's experience gained from installing piezometers at the Hayden Library and helping plan an installation at the Union Falls Dam in Maine.

Hayden Library blueprint 1203-F-14 gives an abstract of the original installation specifications. As noted earlier the drawing does not show the correct location and numbering of the piezometers. Figure 14 and Table II should be referred to for these data.

The deepest piezometer, number 5 of each group, was installed first, followed by the next to deepest and so on up to number 1, which is located in the coarse sand above the clay. Since the piezometers in each group are fairly close together this sequence was followed in order that porous points already installed would not be damaged by subsequent borings. In addition to this precaution each casing was driven with a slight batter away from the group as a whole in an effort to avoid striking casings already in place. Furthermore this scheme tends to remove the porous point assembly from the vicinity of soil disturbed by casings passing near it.

In general the installation procedure outlined in Appendix V was followed with these exceptions. According to the boring foreman, the casing for piezometer B-2 struck one of the other casings during driving. Since the casing had not been driven as deep as planned, a 3 1/2-foot sand pocket instead of 5 feet was used to keep the mean elevation of the piezometer as deep as possible. This piezometer is somewhat less sensitive to pore pressure changes than the average (see Figure 18b) but its normal functioning should not be impaired. One bentonite seal of at least 6 inches in length was placed alternately with sand between each coupling of the casing from the piezometer location to approximately elevation -10. This continuous seal was thought to be desirable because of the possibility of water leaking from the casing and affecting readings of a piezometer nearby.

Shortly after Group B piezometers were installed a temporary shelter was constructed around the group to protect it from inquisitive workmen, moving equipment, falling rivets and other hazards of construction. Figure 15 shows a photograph of Group B piezometers and the shelter before the door and roof were constructed. This temporary shelter served until a permanent enclosure of cinder block was built. No shelter was provided for Group A piezometers since they are more or less isolated.

Cost of the Installation: The following statement is a breakdown of the total cost of the piezometer installation. It is based on the statements in the Bursar's Office for Special Account No. 1007.2, and cost records at the Department of Buildings and Power at M.I.T.

I. Expenses incurred by the Soil Mechanics Laboratory for the construction of the piezometers and installation equipment:

COMPANY	ITEM	COST
KAUFMAN HARDWARE CO.	18 ft.#00 N.P.Chain, 2 1/8" Galvanized Thimbles 1 roll #19 Soft Iron Wire	$ 1.43
NORTON CO.	10 Norton Porous Tubes 1" x 24", 1/4" wall	32.97
SIMPSON, INC.	2 bags Pea Gravel	.90
BROWN WALES CO.	856 ft.0.5" O.D.x0.062" Saran Tubing	138.12

COMPANY	ITEM	COST
CENTRAL SCIENTIFIC CO.	1 #90250 Pump 10 ft. 18202E Rubber Tubing 5 ft. 18202D " "	$ 11.70
AUSTIN-HASTINGS CO.	1 pc. 1 5/8" O.D. x1/2" wall x 36" long Steel Tubing	7.30
JOHN A. ROEBLING SONS	150 ft. 1/8" diam. 7x19 Preformed Galvanized Aircord	15.68
WESTON ELECTRICAL INSTRUMENT CORP.	1 Model 564 Type 3C Volt-Ohmeter	29.89
NATIONAL LEAD CO.	13 lbs. 12" x 2 1/2 Sheet Lead	3.27
OTTAWA SILICA CO.	3 bags Ottawa Sand 20-30, C-190	15.00
BOSTON INSULATED WIRE & CABLE CO.	300 ft. #22 B&S 7/30 Tinned Copper Neoprene Wire	3.99
M. I. T. Lab Supplies	Misc. Supplies and Equipment	7.67
	Express Charge	2.85
	Labor for constructing Tamping Hammer	12.38
	Student labor for filling Piezometer Casings with Sand and Bentonite	51.00
	TOTAL	$334.15

II. Expenses incurred by Thompson Starrett Construction Company in connection with the piezometer installation:

A. Subcontract to Gow Division of the Raymond Concrete Pile Company for installing piezometers:

Labor for Installation	$1162.50	
Insurance, Taxes, etc.	16.72	
Moving tools and equipment to and from the site	30.72	
2" I.D. Strong Pipe Casing	748.71	
Gas and Oil	13.13	
Photostats	2.00	$1973.78

B. Labor and material for installing 4" pipe sleeves through basement floor, constructing temporary shelter for Group B, constructing permanent enclosure for Group B and miscellaneous other labor and material $\underline{\$\ 458.22}$

TOTAL	$2432.00
GRAND TOTAL COST	$2766.15

The writer believes that if the cost were to include the numerous miscellaneous items absorbed in the regular Soil Mechanics Laboratory Account and in the general Thompson Starrett construction costs, the total would be close to $3000. However, the cost of duplicating such an installation may be two or more times this figure since it does not include labor for construction of the porous points and engineering supervision during installation.

C. PIEZOMETER READINGS:

Method of Taking Readings: First piezometer readings were taken near the end of October, about 220 days after the start of construction. At this time nearly all of the basement slab and walls had been poured (see Appendix II). An electrical sounding device which is described in Appendix V under "Equipment for Measuring the Water Level" has been used for nearly all observations made to date. B. D. Zimmerman[10] mounted mercury manometers at Group B in the spring of 1950 but difficulties in the form of air bubbles as air came out of solution (the water being in tension) discouraged concentrated efforts to perfect the manometers. It is possible to take readings with an accuracy of 0.02 feet with the sounding device but normally readings were taken and recorded to the nearest 0.1 of a foot.

Figures 16a and 16b represent a summary of all piezometer elevations plotted against time in days since the start of construction. The writer's present impression of the steady-state readings of the piezometers in the clay are shown by dotted lines. Piezometric curves as a function of depth, sometimes called isochrones, are given in Figure 17 for 100-day intervals. These curves are plotted from data taken from smooth curves drawn through the piezometer readings of Figure 16.

General Discussion of Readings: A quick glance at piezometer curves A-5 and B-5 reveal two important items. Even though the two piezometer readings have never been more than two or three tenths of a foot apart at any one time, it may be seen that there is not complete free drainage at these locations since the piezometers register a pattern of water pressure change similar to that of piezometers in the clay. A possible explanation is that the piezometers are located slightly above the true drainage surface at the bottom of the clay. On the other hand there may actually be restricted drainage at the bottom of the clay. Casings for both A-5 and B-5 were driven to refusal before the porous point was placed. In the case of B-5 fine sand was recovered from the bottom of the casing indicating at least a pocket of sand at this location. By filling the piezometers with water it was found that while B-5 came back to adjustment within 5 minutes it required nearly 12 hours for the excess water in A-5 to flow out.

A second item of importance in connection with curves A-5 and B-5 is that they indicate that the total head (pressure head plus elevation head) at the bottom of the clay is 1.5 to 2.0 feet <u>less</u> than the total head at the drainage surface at the top of the clay. This condition, which may be thought of as a negative artesian condition, indicates that under normal conditions there is a steady seepage of water from the top of the clay to the bottom. If the coefficient of permeability of the clay was constant with depth, the equilibrium readings of the piezometers would lie on a sloping straight line connecting the reading at the top with that at the bottom in Figure 17. Actually, however, the equilibrium curve appears to be located to the right of this line indicating lower permeability at the bottom of the clay.

The sudden irregularity in the readings of A-1 and B-1 at about 920 days (see Figure 16) is caused by pumping from well points at the site of the new Sloan Laboratory at the corner of Vassar Street and Massachusetts Avenue. This building is approximately 1300 feet from the Library. A second, more pronounced and prolonged dip at 970 days is due to pumping from well points at the new John Thompson Dorrance Laboratory located just 500 feet from the Library. Studies relating to this change in boundary condition are presented in IX.

Sensitivity of Piezometers: A measure of the sensitivity of the piezometers, or the speed at which they will record a sudden change in water pressure, may be obtained by filling the Saran tube standpipes and noting the time required for this imposed excess head to be dissipated. M. J. Hvorslev[11] gives the following formula for flow toward a cylindrical shaped well point in an isotropic homogeneous soil mass of infinite dimensions:

$$q = \frac{2\pi L k h}{\ln\left[\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]} \quad \text{-------- (A)}$$

where

q = rate of flow (cm^3/sec.)

L = length of cylinder (cm)

k = coefficient of permeability (cm/sec)

h = head (cm)

D = diameter of porous point (cm)

Hvorslev also gives the following expression for the case when a well point is located through a thin permeable layer between impervious strata:

$$q = \frac{2\pi L k_h h}{\ln \frac{R_o}{R}} \text{ - - - - - - - - - - - - - - - (B)}$$

where

R = radius of the well point

R_o = effective radius to source of supply

Because of continuity the rate of flow, q, as given by the above expressions must be equal to $\left(- a \frac{dh}{dt}\right)$ where "a" is the area of the Saran tube standpipe and $\left(- \frac{dh}{dt}\right)$ is the velocity of fall at any time (the negative sign signifies a decreasing head with increasing time).

Then for expression (A):

$$- a \frac{dh}{dt} = \frac{2\pi L k h}{\ln \left[\frac{L}{D} + \sqrt{1 + (\frac{L}{D})^2}\right]}$$

which has a solution:

$$t_1 = \frac{a}{2\pi L k} \ln \frac{h_o}{h_1} \ln \left[\frac{L}{D} + \sqrt{1 + (\frac{L}{D})^2}\right] \text{ - - - - (A')}$$

where t_1 is the time required for the head to fall from h_o to h_1.

Formula (B) has a solution:

$$t_1 = \frac{a}{2\pi Lk_h} \ln \frac{h_o}{h_1} \ln \frac{R_o}{R_1} \quad \text{- - - - - - - - - - - -} \quad (B')$$

In both cases it may be seen that time t_1 is proportional to $\ln \frac{h_o}{h_1}$ for any given piezometer. A plot of t vs $\frac{h_o}{h_1}$ (log scale) should therefore be a straight line the slope of which is a measure of the speed at which the piezometer would record a sudden change in water pressure.

Figures 18a and 18b show plots of this type for the 6 piezometers in the clay. In all cases, with the exception of piezometer A-4, 95 per cent adjustment to an imposed head which averaged 7 feet was reached within 48 hours. Even though A-4 required a somewhat longer time to return to its original level, the lags are very small compared to the speed at which pore pressure changes take place in the clay. It is felt therefore that time lags need not be studied further insofar as they affect the dependability of the piezometer readings.

Several other points in connection with the piezometer tests are of interest, however. First it may be seen from Figure 18 that the curves are not straight lines as formula (A') and (B') would lead one to expect. Even though these expressions are admittedly approximate the straight-line relationship should hold regardless of the formula used _if_ the coefficient of permeability remains constant during the test. Perhaps the reason why the curves are steeper initially is because of partial saturation in the clay surrounding the porous point. As the head decreases during the test, any air present will increase in volume, tend to decrease the void area available for flow and hence decrease the permeability. In addition, the increase in water pressure at the piezometer location caused by filling the standpipe will immediately cause the clay near the porous point to swell somewhat. As the head dissipates, swell takes place progressively outward and reconsolidation starts first near the porous space, then extends outward. The formulae do not recognize any swelling or effects of partial saturation.

The piezometer tests show that longer times were required for adjustment to be reached in the 1950 tests than in 1948. This may be due to a small amount of fine material plugging the porous space as flow takes place back and forth across the sand-clay interface. Piezometer B-3 showed the opposite trend, however, for which the writer has no logical explanation.

The only case where the piezometer tests indicated that the piezometers were not reading properly occurred at B-3 on January 31, 1950. The level of the water after the test was completed was 0.4 feet higher than the initial level. This prompted the writer to draw the water down (approximately 2 feet) in B-2, B-3, and B-4 to check the elevation to which the water would rise. Results of these tests are shown by dotted curves in Figure 18b. B-3 rose to the level it had fallen to in the piezometer test a few days before.

Since the piezometers, with the exception of B-2, have essentially the same length and diameter of sand at the porous point, the slopes of the piezometer test curves give an indication of the relative permeability of the clay at the piezometer locations. Both piezometers near the center of the clay show unusually rapid adjustment indicating relatively high permeability. The curves of Figure 17 tend to substantiate this fact since the total head in most cases is fairly constant over a considerable depth at the center of the clay. This indicates small gradients or high permeabilities.

An evaluation of the coefficient of permeability by means of expressions (A') or (B') is indeed very crude. (A') neglects the very important fact that the horizontal coefficient of permeability may be 10 to 100 times the vertical. (B') assumes that the vertical coefficient of permeability is zero, which may be reasonable, but the uncertainty involved in selecting R_o makes results questionable. J. P. Gould[4] used equation (A') assuming the k is the horizontal coefficient of permeability, k_h, of an anisotropic soil. If we make that assumption here and use data from the February 1, 1950 tests

$$k_h = 57 \times 10^{-9} \text{ cm/sec for A-2}$$
$$k_h = 167 \times 10^{-9} \text{ cm/sec for A-3}$$

The mean of these values is about one-eighth the permeability that Gould obtained at the Logan International Airport where preconsolidation pressures and overburden pressures were much lower than at the Library.

Effect of Atmospheric Pressure: The only variable, other than actual pore pressure change, which was found to have appreciable effect on the piezometer readings was atmospheric pressure. Between October 16 and October 21, 1950, piezometer readings at Group B were taken every few hours to 0.01 of a foot and compared with corresponding barometric pressures. Figure 19 shows the results of this special study. During the period, no precipitation occurred and no construction was in progress.

Atmospheric pressure change had little or no effect on the piezometers in the clay but B-1 showed a definite response and B-5 showed a moderate trend. The dashed curve in the figure shows what piezometer B-1 would have read had there been 100 per cent response to atmospheric pressure change (13.6 inch drop per 1 inch rise in barometric pressure). The curve is fitted at point A when the barometer was fairly steady. Actually, piezometer B-1 responds to about 60 per cent of the pressure change. This may be seen from a plot of barometric pressure vs B-1 piezometer reading in Figure 20.

The fact that the piezometer reacts to atmospheric pressure change is due, at least in part, to partial saturation in the sand and silt overlying the blue clay. Furthermore the hysteresis loop shown in Figure 20 is probably a form of time lag caused by the relatively impervious silt.

To explain the piezometer variation with atmospheric pressure change, it will be convenient to use the following relationship:

$$\sigma_a = \overline{\sigma} + u + p_{atm}$$

or, the total absolute pressure, σ_a, normal to a horizontal plane at a point in the soil mass is equal to the sum of the intergranular pressure, water pressure, and atmospheric pressure. The water pressure, u, is represented by the height to which a column of water will rise in a piezometer located at the point. If no seepage is occurring, the piezometer level will indicate the water table elevation.

Consider a volume element of sand, represented by point A in Figure 21, which has initial pressures given by

$$\sigma_{ai} = \bar{\sigma}_1 + u_1 + p_{atm\ i}$$

We will now analyze the effect of a sudden increase in atmospheric pressure, Δp_{atm}, on the water pressure and intergranular pressure at point A, by first considering several simplified cases:

Case I. Assume that the soil is saturated below the water table. If water and soil solids are considered incompressible, the water table will not change when the atmospheric pressure is increased. Therefore, Δp_{atm} has no effect on the water pressure and intergranular pressure at point A, or

$$u = u_i \quad , \quad \bar{\sigma} = \bar{\sigma}_1 \quad , \quad p_{amt} = p_{atm\ i} + \Delta p_{atm}$$

and

$$\sigma_a = \sigma_{ai} + \Delta p_{atm}$$

which must hold for all cases.

If this were the case then, piezometer B-1 would not respond to atmospheric pressure changes.

Case II. Assume that the soil is partially saturated below the water table. Furthermore, assume for this case that the silt and sand have an infinite permeability. If the latter assumption were valid, $\Delta\sigma_{atm}$ would cause an immediate compression of the gas in the sand and silt voids. This compression would be accompanied by a sudden drop in the water table level and piezometer reading.

According to Boyle's Law, the volume of a gas varies inversely as the absolute pressure if the temperature remains constant. The magnitude of the piezometer response will depend, then, on the degree of saturation of the soil and the change in atmospheric pressure. Under the conditions of Figure 21 and assuming a degree of saturation of 90 per cent, it is easy to show that the piezometer will fall a few hundredths of a foot only, when the atmospheric pressure increases by 1 inch of mercury. Therefore, we may write

$$u = u_1(-) \quad , \quad \bar{\sigma} = \bar{\sigma}_1(+)$$

for this case.

Case III. Assume now, that the sand and silt are partially saturated and that the silt has a very small permeability. These conditions represent the actual case at the Library.

A sudden atmospheric pressure change, $\Delta\sigma_{atm}$, will be carried partly by intergranular pressure at point A and partly by water (or gas) pressure. (The silt may be thought of as an impervious membrane which transmits the load to the sand.) The proportion of load initially carried by the intergranular structure depends on the relative compressibilities of the sand and gas, assuming water is incompressible. Using the notation shown in Figure 21, the following relationship for the volume of gas in element A, after the application of load, may be derived from Boyle's Law

$$H_g - \Delta H_g = H_g \left(\frac{u_i + p_{atm\ i}}{u_i + p_{atm\ i} + k\,\Delta p_{atm}} \right) \text{ - - - - } (A)$$

where k is the percentage (expressed as a decimal) of the load, Δp_{atm}, initially carried by the pore water or gas.

The coefficient of compressibility of sand may be defined as

$$a = \frac{\Delta(\text{void ratio})}{\Delta(\text{pressure})} = \frac{\Delta H_g}{(1-k)\ \Delta p_{atm}}$$

$$\Delta H_g = a(1-k)\ \Delta p_{atm} \text{ - - - - - - - - - - - - - } (B)$$

Combining (A) and (B) we may write:

$$a(1-k)\ \Delta p_{atm} = H_g \left(1 - \frac{u_i + p_{atm\ i}}{u_i + p_{atm\ i} + k\Delta p_{atm}} \right) \text{- } (C)$$

which can be solved by trial for k.

Immediately after the application of load then, the intergranular pressure at point A becomes

$$\overline{\sigma} = \overline{\sigma}_i + (1-k)\ \Delta p_{atm}$$

in which case

$$u = u_1 - (1-k)\ \Delta p_{atm}$$

and the piezometer falls a distance equivalent to $(1-k)\ \Delta p_{atm}$. (Assuming a degree of saturation of 90 per cent, a void ratio equal to 0.4, and a value of a of 0.001 ft² per ton, k is less than 0.1 for an atmospheric pressure change of 1 inch of mercury.)

Since the water table elevation has not changed appreciably a gradient is immediately established and water begins to flow from the silt to the sand. If sufficient time were allowed after the application of $\Delta\sigma_{atm}$, equilibrium would be reached and Case II stress conditions would prevail. A detailed study of the effects of time and a gradually changing atmospheric pressure has not been undertaken in this investigation.

Piezometers in the clay show no noticeable response to atmospheric pressure change. This must be due primarily to a high degree of saturation and very small permeabilities.

It is interesting to note that changes in atmospheric pressure in this area are equivalent to placing a uniform load of perhaps 30 or 40 pounds per sq ft on the clay, then removing it. The period of the cycle is equal to that of the atmospheric pressure which may be a week more or less.

Since atmospheric pressure changes affect only those piezometers at the top and bottom of the clay, no corrections have been made to piezometer readings given in Figures 16a and 16b.

VI

INTRODUCTION TO SETTLEMENT ANALYSIS PROCEDURE

A. TERZAGHI CONSOLIDATION THEORY: Soil mechanics, which has been defined as the scientific approach to the understanding of soil action is generally considered to have been founded by K. Terzaghi in the 1920's. His presentation of a classical theory on the consolidation of fine grained soils in 1925[16] is still considered one of the most outstanding contributions to soil mechanics to date. This theory forms the basis of nearly all settlement analyses of buildings founded on or above compressible clay soils. In 1942 D. W. Taylor[12] published a consolidation theory, called Theory B, accounting for plastic resistance to compression and another theory accounting for secondary compression has been presented by D. W. Taylor and W. Merchant[20]. Although both theories recognize phenomena not considered in the Terzaghi theory and explain some of its inconsistencies, neither has gained more than mathematical recognition. This is largely due to their complexity and the lack of correlation based on consolidation tests on undisturbed clay samples. Some of the important concepts involved in Theory B are discussed in VII-F and XII.

The writer does not intend to present a detailed discussion of the Terzaghi consolidation theory since it is available in many current publications[15][2][12]. However, in order to acquaint the reader with notations used in this research a summary will be given at this time.

Consolidation of an element of soil involves a gradual transfer of hydrostatic excess pressure, u, to intergranular pressure, p. This stress transfer is accompanied by a decrease in volume of the element as water flows from its voids. The basic differential equation governing consolidation at a point in the soil mass is given by

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \quad \text{- - - - - - - - - - - - - - (A)}$$

where

c_v = coefficient of consolidation, a soil property

$\frac{\partial^2 u}{\partial z^2}$ = space rate of change of the excess pressure gradient, $\frac{\partial u}{\partial z}$, in the vertical direction

$\frac{\partial u}{\partial t}$ = time rate of change of the excess water pressure at the point.

A Fourier series solution of the above equation for a soil sample which has been subjected to a constant initial excess pressure may be plotted as a family of curves(2,p.235). These curves show the progress of consolidation at any point throughout the depth of the sample as a function of a composite variable known as the time factor, T.

$$T = \frac{c_v t}{H^2} \quad \text{- - - - - - - - - - - - - - (B)}$$

where

t = time since the application of the load

H = one half the height of the sample (sample drained at top and bottom).

The state of the consolidation process at a point in the soil mass at some time t is defined as the consolidation ratio, U_z.

$$U_z = \frac{e_1 - e}{e_1 - e_2} \quad \text{- - - - - - - - - - - - - - (C)}$$

where

e = void ratio at any time t

e_1 = initial void ratio

e_2 = final void ratio

One of the important assumptions in the Terzaghi theory is that the void ratio varies linearly with intergranular pressure, p, during the consolidation process in which case

$$U_z = \frac{p - p_1}{p_2 - p_1} = \frac{u_i - u}{u_i}$$

where

u_i = initial hydrostatic excess pressure.

When speaking of the average consolidation ratio (that for the sample as a whole) the subscript z is omitted. A unique relationship between T and U for the case of linear hydrostatic excess pressure is given by curve[2,p. 237], and by Figure 10.11b and Figure 10.12b of the same reference. This relationship will be used to compute the time-rate of settlement.

B. <u>EVALUATION OF ULTIMATE SETTLEMENTS</u>: The following expression for ultimate settlement, ρ_u, as a result of void ratio change may be easily derived

$$\rho_u = \frac{\text{thickness}}{1-e_1} \Delta e \text{ - - - - - - - - - - - - - } (D)$$

where

e_1 = initial void ratio

Δe = change in void ratio during consolidation.

If we define a coefficient of consolidation, a_v, equal to $\frac{\Delta e}{\Delta p}$ (where Δp equals the load increment causing consolidation), then

$$\rho_u = \frac{\text{thickness}}{1-e_1} a_v \Delta_p \text{ - - - - - - - - - - - - } (E)$$

The coefficient of consolidation is generally determined from laboratory tests on undisturbed samples of the soil. Standard procedure in the M.I.T. Soils Laboratory is to load the clay by increments, doubling the previous load every 24 hours.

Since the results of a consolidation test are generally plotted as e vs p (log scale), it is convenient to express ρ_u in terms of the slope of this curve, $\frac{\Delta e}{\Delta \log_{10} p}$, at the average pressure, $\frac{1}{2}(p_1+p_2)$, which applies to the given problem. This slope is known as the compression index, C_c. Making appropriate mathematical conversion, equation (E) becomes

$$\rho_u = \frac{\text{thickness}}{1+e_1} \frac{0.435\ C_c}{\frac{1}{2}(p_1+p_2)} \Delta_p \quad - - - - - - - - \text{(F)}$$

Since p_1 and p_2 vary with depth and Δp, C_c, and e_1 are likely to vary, it is often desirable to compute the total settlement by summing the settlements due to small increments of depth.

C. <u>TIME-SETTLEMENT CURVES</u>: The derivation of equation (F) is independent of the Terzaghi consolidation theory but the time distribution of the ultimate settlement is generally based on equation (B):

$$t = \frac{TH^2}{c_v}$$

The coefficient of consolidation c_v is determined by fitting laboratory time-compression curves to the theoretical T vs U curve mentioned in a previous paragraph. These fitting methods are described in Reference 2, page 238. Once the value of c_v is determined, time t may be expressed as a function of T. For any settlement ρ, associated with a given percentage of ρ_u and hence U, a unique value of T exists which may be used to determine the time required for ρ to occur.

VII

EVALUATION OF FACTORS AFFECTING THE SETTLEMENT ANALYSIS

A. GENERAL: In order to determine the ultimate settlement and time settlement curves for a building, data from three sources are required:

1. Exploratory Borings: Used to evaluate the thickness of the compressible stratum, its drainage conditions, and to determine its depth relative to ground surface.

2. Consolidation Tests on Undisturbed Samples: For determining the compression characteristics of the clay, the initial void ratio, e_1, and the coefficient of consolidation, c_v. In addition, the consolidation test results may be used to estimate the maximum pressure to which the clay has been consolidated in its past history.

3. Building Loads: To be used with a stress transmission theory for evaluating the pressure increment, Δ_p, causing consolidation. The distribution of the building loads may be affected by the rigidity of the structure which should therefore be studied.

Before the settlement characteristics can be predicted accurately from the above data, the investigator must have a thorough knowledge of the assumptions made and of the factors which affect these assumptions.

Assumptions involved in the Terzaghi consolidation theory are well known and have been discussed elsewhere(2). Only the most important of these as they affect this investigation will be discussed here. By far the most critical consideration is the interpretation of data obtained from consolidation tests. If factors such as sample disturbance, load increment ratio, load increment duration and consolidation history are not well understood, the consolidation test may be completely worthless.

Thus, the purpose of this chapter is to discuss the most important factors which affect the settlement prediction, in particular as they affect correlations between field and laboratory data at the Hayden Library.

B. <u>THICKNESS OF CLAY AND DRAINAGE CONDITIONS</u>: Exploratory borings at the Hayden Library have shown that an average blue clay thickness of 90 feet may be assumed (see Figure 2). Since ultimate settlement varies directly with thickness of the clay stratum, all other things being equal, a small variation in thickness is relatively unimportant.

According to the Terzaghi theory the time required to reach a given percentage of the ultimate settlement is directly proportional to the square of the length of the longest drainage path, H:

$$t = \frac{TH^2}{c_v}$$

The number of free drainage surfaces within the clay, then, has a pronounced effect on the time rate of settlement. Borings have indicated and piezometer readings have verified that there is complete drainage at the top of the clay stratum below the Library and that no free drainage surfaces exist within the clay. Even though there appears to be restricted drainage at the bottom of the clay, Figures 16 and 17, it will be assumed that there is free drainage at this level. Therefore, in computing time-settlement curves, one half the thickness, or 45 feet is a reasonable assumption for H. Actually, a 10 or even a 20 per cent variation in H from the true value, is small compared to the effects of radial flow and the questionable value of the coefficient of consolidation determined from laboratory tests.

C. <u>INITIAL VOID RATIO, e_1</u>: The validity of consolidation test data for ascertaining the void ratio in the field has always been a controversial issue. Many soils engineers believe that with careful undisturbed sampling little or no swell occurs, in which case the void ratio at zero load in the consolidation test is equal to that in nature. K. Terzaghi has repeatedly stated that with carefully controlled sampling in highly cohesive soils, the water content remains practically unchanged. A. Casagrande[3] and M. J. Hvorslev[17] have also expressed this belief. Expansive soils, however, especially those which are organic and have absorbed gas in the pore water, undoubtedly swell considerably after removal from the ground. G. P. Tschebotarioff[18] has attributed settlement predictions which are two or three times too large to swell which occurs after sampling. R. F. Dawson[19,discussion] has reported that samples of the Southwest clays have expanded sufficiently in the laboratory humid room to break the paraffin covering.

Regardless of the school of thought, the value of e_1 used has relatively little effect on ultimate settlement computations. However, the subject warrants further discussion insofar as it affects the determination of maximum past pressure and consequently the value of compression index to be used.

Volume changes, which are required to effect a change in the sample void ratio, may occur before the actual sampling operation, during sampling and during sample storage and preparation for testing.

Let us analyze the effect of careful sampling of a saturated clay similar to Boston blue clay, by means of a fixed piston type of sampler. We will start with a consideration of the actual sampling operation itself. According to

M. J. Hvorslev[17], the bore hole, by relieving the overburden pressures in the zone to be samples, may cause some swell at the top of the sample but it generally does not affect the lower part of the sample. Furthermore, Hvorslev states that it is unlikely "that fully saturated soils of low permeability will be subject to significant volume changes during the actual sampling when the sampler is forced rapidly into the soil."

It is difficult if not impossible to estimate the amount of volume change the sample will undergo while it is stored in the laboratory. If carefully sealed, the over-all void ratio will probably remain essentially unchanged even though some water migration may occur. In the opinion of the writer then, the problem becomes a matter of determining what happens to the sample after it is extruded from the sample tube and during the preparation preceding testing. Whatever volume change takes place during this period, whether it be a decrease or increase, will depend on the volume change tendencies of the soil sample which in turn may depend to a large extent on the disturbance which the sample has experienced.

M. J. Hvorslev[17] has presented a comprehensive discussion of the types of disturbances which a sample may undergo during sampling. Two of these which undoubtedly apply to Boston blue clay are the change in stress conditions during sampling and disturbance of the soil structure.

After a saturated sample of clay has been taken from the ground and exposed to atmospheric pressure, the original external forces applied to the sample will be replaced somewhat by capillary pressures developed by surface menisci. The magnitude of these pressures is not known, but it certainly depends on the magnitude of the original in-situ pressures, the type of soil - whether it has a tendency toward expansion or not, the pressure history of the soil and the structural disturbance. Ideally, the capillary pressures are likely to be nearly equal to the effective stresses in situ.

The second type of disturbance, structural disturbance, can have two effects according to Hvorslev, depending on the type of clay. If the soil is highly preconsolidated the void ratio would tend to increase as a result of disturbance.

A normally consolidated clay, however, may tend to decrease in volume much like a loose sand does during shear or vibration. P. C. Rutledge(19) states that the sampling operation removes the tendency of the mineral grain structure to expand. As a result there may be little or no capillary pressure acting on the soil.

The writer visualizes the following conditions prior to and after the sample extrusion. Before the sample is extruded, little or no over-all volume change has occurred. However, there may have been water migration to the center of the sample from a thin remolded zone around its circumference. The possibility of this occurring was pointed out as early as 1936 by A. Casagrande(3).

The general structure of the entire sample may be somewhat disturbed but it still has a moderate tendency to expand, this expansion having been prevented thus far by the walls of the sample tube. The extrusion process further remolds the sample surface and relieves the confining pressure. Swell tends to occur and immediately surface capillary pressures develop to balance the swelling tendency. These pressures are, in all probability, small compared to the original intergranular pressure in situ.

Since the surface is remolded it consolidates readily even under small capillary pressures. However, the capillary pressure required to prevent over-all swell cannot develop until sufficient consolidation has taken place. As a result, the interior "undisturbed" zone will further swell while the exterior disturbed zone consolidates. This internal swell may be appreciable even though the volume of the remolded material is only one hundredth of that which is essentially undisturbed. This follows since the compression index of the former is a great many times the swelling index of the latter.

The writer believes, then, that by the time the sample has been prepared for testing there will be little or no negative pore water pressure and the sample will have undergone a measurable amount of swell. In this case the void ratio represented by the value at zero load in the consolidation test will be somewhat higher than the in-situ void ratio.

While running consolidation tests on Boston blue clay, on one occasion only has the writer observed the sample to swell under a small initial load. During the course of triaxial shear research with pore pressure measurements at M.I.T., little evidence has been found which shows that an initial negative water pressure exists in the unsheared specimen. In this case, however, enough water may have been added to the sample during insertion of the pore pressure measuring device to relieve some of the negative water pressure.

Even though the writer believes that the sample does swell prior to testing, the values of e_1 at the beginning of the consolidation test have been assumed equal to the void ratio in the field, simply for the reason that it makes little difference in the ultimate settlement computation. These values of e_1 are tabulated in Table I.

D. COMPRESSION INDEX, C_c: In the usual settlement analysis procedure the slope of the laboratory e - log p curve at the average pressure $\frac{1}{2}(p_1 + p_2)$ is taken as the compression index. If it is positively known that the clay in nature is normally consolidated (not precompressed), then the slope of the virgin straight-line portion of the curve is used for C_c. K. Terzaghi[19, discussion] states that consolidation tests give reasonable results for the compression index if the clay is normally consolidated and if the net load increment is large. However, it is generally believed that the value of C_c determined from consolidation tests for a precompressed clay gives values of settlement which may be several times too large. Since the clay stratum beneath the Hayden Library may be considered precompressed (Figure 6), the following discussion will concentrate on determining a rational value of C_c for this case.

In Figure 22 the compression curve for sample 11-13-72.6 has been plotted with a solid line. Lines representing the initial void ratio, e_1, the overburden pressure, p_1, and the adjusted maximum past pressure, see Table I and Figure 6, have also been indicated. These solid lines then represent conditions which are known. Our object now is to estimate the void ratio-pressure condition in nature and the probable field compression curve.

If no sample swell had occurred between the time the sample was taken in the field and the time data were obtained from the consolidation test to determine e_1, point A would represent the void ratio-pressure condition in nature. However, some sample swell undoubtedly occurs and the true void ratio-pressure condition in situ, therefore, lies below point A*. To define the lower boundary of the void ratio in situ, we will make use of the fact that a soil sample after rebound from p_1 will generally recompress to p_1 at a lower void ratio. This void ratio difference depends to a large extent on the amount of remolding which the sample has undergone and the magnitude of the pressure release from p_1. Point B, therefore, represents a lower limit to the void ratio-pressure condition in nature. Point C has been located to represent the probable condition in situ. Swell which has occurred subsequent to sampling is represented by the small void ratio change CA. No rebound curve from C to D has been drawn since the sample may have reached e_1 by a combination of effective pressure decrease at no void ratio change (due to disturbance of the structure) and internal swell prior to testing.

Point F, which represents the probable maximum past pressure and void ratio at this pressure, has been located

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* It is possible that some highly precompressed expansive soils will swell sufficiently after sampling so that the laboratory compression curve will pass above point A.

at a slightly smaller void ratio than point; C some small swell would have taken place when the soil rebounded from F to C. Line FG represents the virgin compression curve in nature. It is shown slightly steeper than the virgin laboratory curve since it is believed that these straight lines tend to converge.

The slope of the heavy dotted curve to the right of point C represents the approximate compression index in nature. It is easily seen that the slope of laboratory curve near point B is ten to twenty times as steep as the curve in nature. Perhaps the example drawn shows an extreme variation between the actual and the laboratory values of C_c. For larger differences between the maximum past pressure and the overburden pressure the two values will give a better check. Similarly, if there is no precompression the values check closely since the slopes of the virgin curves are approximately the same. K. Terzaghi and R. B. Peck[21] state that, for the best of samples, the slope of the laboratory curve is two to five times the field compression curve if Δp is smaller than about one half the amount the clay is precompressed.

Probably the most important factor affecting the recompression portion of the laboratory e - log p curve is sample disturbance. The effect is shown in Figure 22 by a dashed curve which represents a slightly disturbed clay. T. Van Zelst[22] shows that the process of planing the top and bottom surfaces of the sample during preparation for testing has a very important effect on the e - log p curve in the recompression range - the greater this disturbance, the steeper the slope. P. C. Rutledge[19] states that maximum past pressures determined from laboratory tests are smaller than the actual values because of sample disturbance. This effect is shown in sample 11-13-72.6 by the difference between points H and F.

A method of ironing out part of this major discrepancy between the field and laboratory compression curves has been used by some investigators. The sample is first loaded to its maximum past pressure then rebounded to the present overburden before reloading. Figure 22b shows the effect of this cycle. In many cases this method will give considerably better results for C_c than the standard test. However, the problem of determining the maximum past pressure still exists.

Such factors as side-wall friction, load increment duration, and load increment ratio have little effect on the values of compression index determined from laboratory consolidation tests. A. Casagrande and R. E. Fadum[23] state that "consolidation tests lasting several months give practically the same results (compression curves) as the standard tests performed in a few days." K. Langer[24], however, presented tests on the effect of speed of loading which showed that the compression index decreased considerably as the time required for the test was increased. K. Terzaghi[25] has used these results to support his hypothesis that when the clay is loaded very slowly, as in the natural deposition process, a "solid water" bond builds up which causes the compression index to be very small. Langer's tests were run on a highly precompressed clay which was very susceptible to swelling. The maximum pressures used in the tests were reported as about 6.5 kg per sq cm while the estimated precompression was 25 kg per sq cm. There is good reason to believe, then, that the swelling effects of the soil were greatest when the speed of loading was slowest.

From the preceding discussion it is not difficult to see the importance of the maximum past pressure determination insofar as it affects the value of the compression index to be used in the settlement analysis. Any graphical procedure which makes use of the shape of the laboratory consolidation curve to determine the maximum past pressure is subject to large errors. Geological evidence, if it is available, is an excellent method for determining whether or not the clay is precompressed but it is doubtful whether it can give reliable quantitative data. In the case of the Hayden Library, piezometers show that there are no artesian pressures at the bottom of the clay and, as a result, the maximum past pressure must be at least equal to the overburden pressure (see II).

Table I and Figure 5 give values of the compression index determined directly from consolidation tests. It must be remembered that these values are in all probability many times too large.

E. COEFFICIENT OF CONSOLIDATION, c_v: Even though very little has been written on this subject compared to the compression index, the magnitude of the coefficient of consolidation may be extremely important in foundation design. If a majority of the settlement due to building loads takes place during construction, larger ultimate settlements may be tolerated since settlement following the installation of utilities and interior finish causes the principal damage.

Swell following excavation generally occurs very rapidly but there are no references, to the writer's knowledge, where the actual swelling coefficient has been evaluated. One important point related to the speed at which swell occurs has been overlooked by most investigators. This factor, which concerns the net loads to be used in the settlement computations, is discussed in VIII.

According to the Terzaghi theory, the relationship which exists between the coefficient of consolidation and the coefficient of compressibility is given by

$$c_v = \frac{k(1+e)}{a_v \gamma_w}$$

where

k = coefficient of permeability

γ_w = unit weight of water.

At a given pressure a_v varies directly with the compression index, C_c. Therefore, the coefficient of consolidation varies inversely as the compression index - k and e varying only slightly for a small range in pressure. We have shown in the preceding section that the compression index determined from consolidation tests is, in all probability several times too large. From the inverse relationship, then, we would expect the coefficient of consolidation to be several times smaller than the field coefficient for a precompressed clay.

As in the case of the compression index, sample disturbance contributes a major share of this discrepancy. D. W. Taylor[12] shows c_v values of the order 0.5×10^{-4} to 3.0×10^{-4} cm² per sec for remolded Boston blue clay in the range of pressure from 1/4 to 2 kg per sq cm. Good "undisturbed" samples of the same clay give values 30 or 40 times these in the same pressure range. On the other hand, as in the case of the compression index, the coefficients of consolidation check closely in the virgin compression range.

Two methods are commonly used to determine the coefficient of consolidation from laboratory data. Both methods are described in detail in Reference (2) and will be referred to as the "square root of t" and the "log t" time fitting methods. Eight consolidation tests, conducted to determine the effect of enclosing the sample in a thin rubber membrane, have been run on undisturbed Boston blue clay. These tests, described in Part II, show that c_v determined by the square root method averages 1.25 to 1.5 times that of the log method.

Special tests with pore pressure measurements during consolidation, also described in Part II, indicate that the correct value of 100 per cent primary compression, based on actual pore pressures, lies midway between the values given by the square root method and the log method. This indicates that the coefficient of consolidation determined by the square root of t time fitting method is larger than the actual laboratory value while the log t time fitting method gives results which are too small. Since the field value of c_v for a precompressed clay is probably even larger than that determined by the square root method and since the square root method is simpler and requires less data, the writer recommends its use for precompressed Boston blue clay.

D. W. Taylor[12] has presented data on the effect of the load increment ratio on the coefficient of consolidation of a remolded clay. When test data are interpreted by the Terzaghi theory, c_v at a given pressure increases considerably as the load increment ratio is increased. This discrepancy is eliminated, however, when the consolidation data are interpreted according to Taylor's theory accounting for plastic resistance to compression.

The full extent of the effect of load increment ratio on an undisturbed clay is not known. If a clay stratum is normally consolidated or very slightly precompressed the magnitude of the net load applied to the clay may be extremely important. If small net loads are applied, c_v may be many times that given by laboratory tests but the reverse may be true if the net load is large.

Values of the coefficient of consolidation, determined by the square root method, for samples from the Hayden Library, are given in Table I and Figure 5. The average c_v for the range 1.0 to 3.0 kg per sq cm is about 50×10^{-4} sq cm per sec. Again, it should be pointed out that these values are probably several times smaller than the values which apply to the Library.

F. ASSUMPTIONS INVOLVED IN THE APPLICATION OF THE TERZAGHI CONSOLIDATION THEORY:

One-Dimensional Drainage: One of the most serious assumptions involved in the use of the Terzaghi theory to compute the time rate of settlement of a building is one-dimensional drainage. Consolidation is necessarily one-dimensional in the laboratory and the value of c_v determined there is truly a vertical coefficient. However, consolidation of a clay stratum caused by a more or less concentrated load may proceed at a considerably faster rate than predicted because of radial flow. Gould[4] found that the effect of radial flow at the Logan International Airport was to speed up the consolidation by approximately 3 times the theoretical.

The extent to which radial consolidation affects the settlement rate depends primarily on two factors. The first involves the ratio of the horizontal to the vertical permeability. This ratio may be 5 or more for a clay similar to Boston blue clay, in which case considerable radial flow may take place even under small horizontal gradients. The second factor involves the degree of concentration of the stresses transferred by the load to the clay stratum. For a thin stratum near the surface loaded by a fill of uniform depth over a wide area, the effects of radial flow will be nihil. The other extreme would be the case of a heavy monument or tall narrow building founded on a thick bed of clay.

The Hayden Library is a structure 190 by 220 feet in area, transmitting its load to a layer of clay extending 90 feet below the points of application of the load. Since the building is supported by caissons, each new load application will set up extreme horizontal gradients between the zone directly below each caisson and the area between caissons. This effect is most pronounced in the upper foundation clay. Further radial flow will take place during construction from a zone below a group of caissons which have just received a load, to surrounding areas. In addition there will be over-all flow during and after construction to areas surrounding the building.

Because of the complex pattern of horizontal gradients and uncertainties involved in estimating the horizontal permeability, the writer has not attempted to separate the vertical and horizontal effects quantitatively in analyzing the time rate of consolidation at the Hayden Library. The approach which has been used is outlined in IX.

Primary and Secondary Compression: The total compression which takes place within 24 hours after the application of a load to the laboratory consolidation sample, is composed of a sudden initial displacement and primary and secondary compression. Primary compression takes place as a result of drainage of the pore water because of the hydrostatic excess pressure. Following the primary compression and accompanying it to a certain extent is secondary compression, which is a plastic flow phenomenon of the type recognized in Theory B. The ratio of the primary to the total compression is known as the primary compression ratio, a small value indicating large secondary effects.

It was mentioned in VI-C that the total settlement, ρ_u, as computed from formula (F), was assumed to be distributed according to the Terzaghi theoretical time relationship which recognizes only primary compression. This assumption has the effect of predicting a greater settlement at a given time than actually occurs, all other things being equal. The discrepancy is greatest when the primary compression ratio is smallest.

Although the laboratory value of the primary compression ratio may be readily computed, the extent to which it applies to a thick stratum of clay subject to a small load is not known. Furthermore, the field value of the primary compression ratio, if it has any meaning, is difficult if not impossible to determine from field data. Without pore pressure measurements it may be difficult to determine the end of primary compression. In addition, the compression due to building loads must be separated from subsidence of the over-all area. Finally, and most important, what is ultimate settlement? Presumably the ultimate settlement is that which would occur after the building load has acted for a period equal to the age of the clay. Practically, although it cannot be proven, this settlement may not be much greater than that which occurs in the life of the building.

Bonds and Plastic Resistances, Theory B: The possibility of the existence of intergranular bonds in a soil mass have been recognized for some time. K. Terzaghi and R. B. Peck[21] have attributed to a structural bond lags in swell following excavation and in recompression when loading.

In 1941 Terzaghi[25] published an hypothesis on the possible existence of a "solid water" bond which may not be broken under small building loads. According to Terzaghi, the clay layer may have a very flat compression curve as a result of gradual loading by natural deposition of overlying soil. If a large load is applied to the clay it may pass into a "lubricated" state which gives a steeper compression curve, resembling that obtained in a laboratory consolidation test. On the other hand, the "solid water" bond may not be broken under a small load, in which case compressions will be very small. Terzaghi has used this hypothesis to explain inconsistencies in settlement predictions when small loads are applied. Similarly, he uses this hypothesis to explain the fact that certain natural clay deposits have little or no decrease in void ratio with depth.

D. W. Taylor's Theory B[12] recognizes the existence of a plastic structural resistance to compression which is composed of two parts - a bond associated with previous secondary compression and a viscous resistance which is a function of speed of compression. The reaction associated with bond and viscous resistance is an increase in intergranular pressure (and a corresponding decrease in excess pore water pressure) from that recognized in the Terzaghi consolidation theory. Consolidation tests on pressure measurements have verified the existence of plastic structural resistance.

Taylor defines bond, p_v, as that plastic structural resistance, p_p, existing at the termination of primary compression. If GAC (Figure 3a) represents the relationship between speed of compression, $(-\frac{de}{dt})$, and p_p. If AD is the speed at the termination of primary compression then OD is equal to the bond. The magnitude of the bond for a given increment of load depends on the amount of previous secondary compression. It should not be thought of as being broken under large loads.

Refer now to Figure 23b. AB represents a line plotted from void ratios determined at the end of primary compression and their corresponding intergranular pressures for a standard consolidation test. According to Taylor's bond hypothesis, lines CD, EF, and GH which represent void ratio vs pressure curves for tests of 1-year, 100-year, and 100,000-year durations respectively, would be approximately parallel to AB.

If point I represents the condition in situ, IJ is the bond as defined by Taylor. According to idealized concepts, when an increment of load, Δp_1, is applied, bond and a very small viscous resistance account for a sudden increase in intergranular pressure, equal to IK, as the clay begins to consolidate. At the end of primary compression the element is represented by point L and at the end of 100,000 years by point M. During the life span of the building the total settlement is represented by the void ratio change Δe_1.

Taylor has suggested the possibility that if the load increment is small enough, there may not be an appreciable excess water pressure associated with the load. If Δp_2 is applied, for example, bond may carry the entire increment and in the course of 100 years a negligible settlement represented by Δe_2 will occur as a very slow plastic flow. The compression index represented by a line connecting points I and O could never be obtained from a laboratory consolidation test while that for a line connecting I and

N may be closely approximated in the laboratory. Thus, this hypothesis would also explain the discrepancy between predicted ultimate settlement and actual settlement when the net building load is small.

Although the clay stratum beneath the Hayden Library is precompressed it is neverless subject to bond phenomena of types discussed above. The inclusion of precompression makes their effects all the more difficult to evaluate.

G. STRESS INCREMENT, Δp: An excellent discussion of the assumptions involved in the stress analysis is given by R. E. Fadum[26]. Stress transmission theories and considerations of continuity will be discussed here since they affect analyses which follow.

Stress Transmission Theories: At least two elastic theories are available for estimating the load increment Δp transmitted from a surface load to a point in the soil mass. They are the Boussinesq and Westergaard stress transmission theories. (A complete description of each as applied to soils is available in Reference 2.) It is believed that both theories give reasonable values of the vertical stress on a horizontal plane at a given point.

Although the accuracy of these stress transmission theories in computing stresses in soils is not known, the use of elastic theories represents the most rational approach to the problem. Some investigators use the older Boussinesq theory which generally gives somewhat larger values than the Westergaard approach. D. W. Taylor[2] and R. E. Fadum[26] believe that Westergaard gives the more reasonable values. All of the stress computations which follow are based on the Westergaard expressions and in particular on influence values for different types of loading as prepared by Fadum in Reference 27.

Continuity Considerations: The computation of vertical stresses throughout the soil by either theory is generally based on design footing loads without regard to possible redistribution of loads as the building settles. If the structure is ideally rigid the settlement at all points will be equal. In this case, if a building is founded above a thick stratum of cohesive soil, the center columns will distribute a portion of their load to the exterior footings.

A settlement analysis involving the effects of structural continuity is extremely tedious since it requires a process of successive approximations involving structural and settlement analyses. R. E. Fadum[26] is of the opinion that unless the structure is deliberately stiffened to reduce differential settlements, the building is probably so flexible that little load redistribution takes place.

The Hayden Library is a riveted steel frame structure 3 or 4 stories high spreading over an area 190 feet by 218 feet. The frame itself is undoubtedly flexible enough to withstand considerable differential settlement. Possible rigidity offered by a 3.5-foot slab which extends the length of the building (see Figure 1) has been investigated. This 218-foot slab is actually extremely flexible. A simple computation, based on a weightless simply supported beam uniformity loaded, shows that it requires a uniform load of only 10 lbs per sq ft to produce a deflection of 0.25 inch at the center. The weight of the slab itself is over 500 lbs per sq ft. The purpose of the slab is to distribute each column load to several caissons. It fulfills this purpose and undoubtedly causes a redistribution of the column load to the caissons immediately surrounding it as they settle. The over-all slab is so flexible, however, that little or no transfer of load from the center of the slab to the ends will take place.

On the other hand, redistribution of the exterior caisson loads due to the basement walls probably occurs. No quantitative attempt has been made, however, to account for this load transfer.

In the analyses which follow the building has been assumed to be completely flexible.

VIII

ANALYSIS OF THE MAGNITUDE OF THE LIBRARY SETTLEMENT

An introduction to the settlement analysis procedure and a discussion of the many factors which may affect the Library settlement prediction have been presented in the previous two sections. This section and the following one will be devoted to correlations between settlement and pore pressure data obtained at the Hayden Library and laboratory data determined from consolidation tests. This section is a study of the actual settlements which have occurred at the Library.

A. <u>NATURE OF THE PROBLEM</u>: In order to illustrate the nature of the problem, load intensity (Δp) vs depth curves below three caissons have been computed. Caisson 108 which is near the center of the heavy slab, caisson 44 which is in the court area, and caisson 50 which is located on the outside wall of the east wing have been selected as typical. The exact location of these caissons is shown in Figure 24.

Stress release at each caisson due to excavation has been computed on the basis of assumptions shown in Figure 24. Influence values for uniformly distributed loads on the basis of the Westergaard stress transmission theory were taken from Reference 27.

Present total caisson loads given in Appendix II have been used with a variety of influence charts to compute the stress increase due to building loads. For the stress below each of the caissons due to the caisson load itself, influence values for a point directly below the center of a circular loaded area were used. For caissons in the immediate vicinity of each of the three locations and for depths below the caisson bell of less than 30 feet, charts for uniformly loaded rectangular areas were used. In this case, a square equal in area to the caisson bell was assumed. Finally, for all other caissons and depths in excess of 30 feet, the caisson loads were assumed to act at a point.

Results of the stress analysis are shown in Figure 25. The crosshatched area represents the net load intensity - stresses due to Library loads minus the stress release due to excavation. The distance between the line of stress release due to excavation and the dashed curve represents the stress increase caused only by the caisson directly above. One can easily estimate the relative settlements of the three caissons by comparing either the shaded areas or the areas representing the stress increase caused only by the caisson in question.

The conventional method of computing the ultimate settlement of each caisson due to consolidation of the clay would involve the use of a net Δp determined from an intensity diagram similar to those in Figure 23. Since the net load is practically zero in the lower foundation clay, only the upper foundation clay would have to be considered. Using consolidation test results described in II and the net Δp, estimates of settlements are obtained which agree closely with the actual settlements for many caissons. Normally it would be concluded that the consolidation tests had given reasonable values of the compression index. However, it has been pointed out that these values are generally considerably larger than the field C_c for a precompressed clay in which case the above approach may be questioned.

One of the valuable contributions of the piezometer installation has been to show the rapidity at which swell takes place after excavation. The readings indicate that by January, 1949, or about 270 days after the start of construction, when the net load was still negative (see Figure 7) the piezometers were reading a positive excess pressure. Referring to Figure 25, this was the case when the Δp curve was that shown dotted for the three caissons. Settlement due to consolidation phenomena commenced at this time so the Δp to be used in the computation of ultimate settlements should be that represented by the <u>entire</u> area to the right of the dotted curve. It might be argued that the coefficient of compressibility applying to recompression from the dotted curve to zero net load, is negligible compared to that holding for the plus net loads. However, the clay stratum, as well as being precompressed, has recently undergone a loading and unloading cycle due to the wellpoint drawdown. This will tend to obscure any difference between the recompression value of a_v and the a_v applying to positive net loads.

The ultimate settlement of each caisson will come primarily from three sources; settlement due to compression of the sand, deformation of the clay caused by shearing distortions, and settlement due to volume change (consolidation) of the clay. The procedure discussed above considers settlement due to consolidation only. This is common practice since settlement from the other two sources is generally small by comparison. However, the settlements at the Library are so small that any of the three sources mentioned could contribute a major portion of measured displacement. Data from compression tests on samples of sand, taken at the bottom of caisson wells during construction, indicate that the sand may contribute as much as 1/4 inch to the total settlement. Unfortunately, no direct measurement of the compression of the sand as a result of caisson loads has been made.

Time-settlement curves, drawn from data in Appendix IV, show that nearly all of the present Library settlement took place during construction and that the settlement is now proceeding at an exceptionally slow rate. This might indicate that the settlement was largely due to shearing strains and/or compression of the sand. However, the piezometers show that whatever settlement took place due to consolidation also occurred during construction since very little excess pressure remained at the time the building was completed.

To separate the variables, the problem will be approached by two methods. First, a correlation will be made based on the assumption that all of the settlement is due to consolidation of the clay. This is followed by a study of caisson settlements assuming each acts as a spread footing independent of the surrounding caissons.

B. SETTLEMENT CORRELATION BASED ON CONSOLIDATION THEORY:

Preliminary Analyses vs Actual Settlement: The first complete settlement analysis of the Hayden Library was made by the foundation engineers, Moran, Proctor, Freeman and Mueser. Their estimate of settlement at the end of 6 years varied from 5 inches beneath the center of the Library heavy slab to about 2 1/2 inches in the northwest corner. This, however, was based on future loads (Appendix III) which include considerably greater loads in the court area and higher live loads than actually exist throughout the entire building. Furthermore, their estimate was based on the Boussinesq stress transmission theory which gives induced stresses as much as 50 per cent greater than Westergaard.

In the spring of 1949, B. A. Grand[28] made a settlement analysis of the Hayden Library. He predicted a settlement of 2 1/2 inches under the heavy slab and 3/4 inch in the court area. Grand corrected the Moran, Proctor, Freeman and Mueser design caisson loads for the future additions but still used live loads considerably larger than the actual. His analysis was based on the Westergaard theory.

Contrary to both predictions, the settlement contours shown in Figure 12 indicate that the maximum settlements have not occurred beneath the heavy slab but rather at the northwest and northeast corners. This difference, whether the settlement is caused by consolidation or not, is due to the difference between the design and existing live loads, the actual live loads being from 10 to 20 per cent of the design values. As a result, the average existing total load is about 70 per cent of the design total load.

Settlements which have occurred at the Library are smaller than predicted. The fact that the loads used in the prediction were larger than the actual ones would explain this difference _if_ the net Δp applied to this case.

It has been pointed out, however, that the use of net loads for the analysis is incorrect. (The importance of using the actual building loads is illustrated by noting the Δp vs depth plot for caisson 108 in Figure 25. Since the excavation release is about 90 per cent of the gross intensity of load, a 10 per cent increase in stresses caused by building loads could increase the net load nearly 100 per cent.)

Another major source of error in the computed settlements lies in the fact that laboratory values of compression index are larger than the field values. This point is discussed in VII and is analyzed quantitatively in the following section.

Correlation Based on Corrected Δp Curves: The writer has pointed out that there is no justification for the use of a net load diagram for Δp as given by the shaded portions of Figure 25. This becomes apparent when studying the piezometer curves and noting that positive excess pore water pressures were recorded when the load intensity was that shown by dotted lines in Figure 25. In this case, then, the Δp represented by the area to the right of the dotted curve must be used in the settlement estimate.

Consider caissons 108, 44, and 50 and consolidation test data described in II. The ultimate settlement is given by

$$\rho_u = \frac{\text{thickness}}{1+e_1} \; \frac{0.435\, C_c}{\frac{1}{2}(p_1+p_2)} \, \Delta p$$

We will use average values for the upper and lower foundation clay.

Upper Foundation Clay:

thickness = 20 feet, e_1 = 1.00, C_c = 0.06, and p_1 = 1.4 tons per sq ft

Caisson 108

Δp = 0.33 tons per sq ft

Caisson 44

Δp = 0.18 tons per sq ft

Caisson 50

Δp = 0.34 tons per sq ft

Lower Foundation Clay:

thickness = 70 feet, e_1 = 1.10, C_c = 0.20, and p_1 = 2.6 tons per sq ft

Caisson 108

Δp = 0.15 tons per sq ft

Caisson 44

Δp = 0.13 tons per sq ft

Caisson 50

Δp = 0.10 tons per sq ft

The ultimate settlements are

Caisson 108 $\rho_u = 0.66 + 1.96 = \underline{2.6 \text{ inches}}$

Caisson 44 $\rho_u = 0.38 + 1.70 = \underline{2.1 \text{ inches}}$

Caisson 50 $\rho_u = 0.68 + 1.31 = \underline{2.0 \text{ inches}}$

These estimates as well as being high are not in the correct relationship to each other since the actual settlements are 0.52, 0.22, and 0.72 inches respectively. (The actual settlements are those measured to date from a zero reading at 238 days since the start of construction. The Δp values used above are very nearly the added stresses from this time to the present. Although the computed settlements are ultimate values while the actual readings are 2-year settlements, it is believed the actual ones are not far from some undefined ultimate.)

Not even analyses based on actual loads and corrected Δp curves give settlement predictions approaching the actual settlements. If smaller values of C_c were used the <u>average</u> predicted settlements could be fitted to the actual ones but the distribution would be incorrect. If average C_c values are backfigured from actual settlements they are about 1/3 to 1/10 those given by laboratory consolidation data. If allowances were made for settlement from other sources, one could easily justify average C_c values in the field of 1/10 those determined from laboratory tests. In this case C_c may be equal to 0.006 for the upper foundation clay and 0.020 for the lower.

Meager data are available to compute a series of compression indices using the piezometer curves of Figure 17 and settlement observations from Appendix IV. The distance between any two isochrones at a given depth (Figure 17) represents a transfer of stress from water to the soil skeleton if no loads are applied during the period. The area between the curves divided by the height of the plot gives the average increase in intergranular pressure for the period. Settlement which takes place during the period represents an average void ratio decrease, Δe, which may be readily computed. Since

$$C_c = \frac{\Delta e}{\Delta(\log_{10} p)} ,$$

this method, given the name "pore pressure-area" method by J. P. Gould[4], may be used to estimate the field compression index.

Table III shows the steps involved in the computation of the compression index by the pore pressure-area method. The clay has been divided again into two layers, the upper foundation clay and the lower foundation clay. From 600 days to the present only a small amount of load, in the form of books, has been added. Therefore, the three periods - 600-700, 700-800, and 800-900 days represent intervals when the Library loads were essentially constant.

Computations for Table III have been made on the following basis:

Δu (ft) —— the average decrease in water pressure during the period, for the foundation layer in question - data from Figure 17.

Δp (tons per sq ft) —— the corresponding average increase in intergranular pressure.

$p_2 = p_1 = \Delta p$ —— p_1 has been assumed as shown at the bottom of the table. [During the consolidation process p_1 changes slightly but since differences, $\Delta(\log_{10} p)$, are used very little error is involved if p_1 is assumed constant.]

total ρ(in) —— settlement at the piezometer location during the period in question. Since no observation points are located exactly at the piezometer installation, an average settlement of the points immediately surrounding the piezometers has been used.

ρ(in) —— settlement of the upper or lower clay layer for the period, based on the following assumptions: Using the laboratory consolidation test results, settlement in terms of Δp may be expressed by

<u>Upper Clay</u>

$$\rho_{upper} = \frac{(20)(12)}{1+1.00} \cdot \frac{(0.435)(0.06)}{1.4} \Delta p_u$$

$$= 2.24 \Delta p_u$$

<u>Lower Clay</u>

$$\rho_{lower} = \frac{(70)(12)}{1+1.10} \cdot \frac{(0.435)(0.20)}{2.6} \Delta p_\ell$$

$$= 13.4 \Delta p_\ell$$

From consolidation test results, then, if Δp were the same for each layer the settlement of the lower clay would be $13.4 \div 2.24 = 6$ times that of the upper clay. Assuming this ratio holds for the clay in situ the actual Δp values for each layer, determined from piezometer curves described above, may be used to find the percentage of total ρ contributed by each layer. From the equations

$$\rho_u + \rho_\ell = \text{total } \rho$$

and

$$\rho_\ell = 6 \frac{\Delta p_\ell}{\Delta p_u} \rho_u$$

the expression for ρ_u is

$$\rho_u = \frac{\text{total } \rho}{1 + 6 \dfrac{\Delta p_\ell}{\Delta p_u}}$$

As an example, for the time interval 700-800 days at Group A piezometers

$$\rho_u = \frac{0.04}{1 + 6 \dfrac{.0097}{.0050}} = 0.003 \text{ inches}$$

and

$$\rho_\ell = 0.040 - 0.003 = 0.037 \text{ inches}$$

Δe —— void ratio change during the period computed from

$$\Delta e = \frac{\rho\,(1+e)}{\text{thickness}}$$

Average values of the compression index determined by the pore pressure-area method are 0.023 for the upper foundation clay and 0.060 for the lower foundation clay. These values, about 1/3 those given by the consolidation tests, would still give predicted settlements which are somewhat larger than the actual ones.

A principal source of error in the above analysis is the questionable values of settlements used. Settlements of a few hundredths of an inch appear, while the precision of the observations is the same order of magnitude.

Values of the coefficient of compressibility, a_v, corresponding to C_c equal to 0.023 and 0.060, are 0.007 and 0.010 sq ft per ton. J. P. Gould[4] used the pore pressure-area method at the Logan airport to determine an average a_v of 0.003 sq ft per ton. There is no reason to believe that the Library values should exactly agree with those which Gould determined. Nevertheless, it is probable that the Library values determined by the pore pressure-area method are 2 to 3 times larger than the true field compression index.

C. <u>SETTLEMENT CORRELATION BASED ON INDIVIDUAL BULBS OF PRESSURE</u>: The preceding analysis was based on the assumption that the Library settlement was due to consolidation phenomena. It was shown that the settlement trend, higher around the exterior of the building (see Figure 12), could not be entirely accounted for by this approach. The estimated settlements of the three caissons studied would be in the correct relationship, however, if settlement due to the upper foundation clay only was considered. This may be seen by comparing the contribution of the upper foundation clay to the ultimate settlements computed in the preceding section.

The stress intensity in the upper clay is controlled primarily by the caisson directly above the point in question. In addition, settlement due to compression of the sand depends directly on the individual caisson. It is reasonable then, to base a settlement correlation on isolated bulbs of pressure. This approach, simpler than a consolidation analysis, does not consider the proximity and loads of surrounding caissons.

Formulas based on Poisson's ratio and the modulus of elasticity of the soil are available for computing the elastic deformation caused by a footing load. Displacements obtained by these expressions are usually considerably greater than actually occur because the modulus of elasticity determined from laboratory tests is too large.

Loading tests on a cohesive soil show that the settlement is proportional to the intensity of pressure if that intensity is small compared to the ultimate bearing capacity of the clay. Similarly, all other things being equal, the settlement is approximately proportional to the diameter of the loaded area. Then

$$\rho \sim q\ d$$

where

ρ = settlement

q = intensity of pressure at surface of clay

d = diameter of loaded area at surface of clay.

Figure 26 has been prepared to show a plot of ρ in inches vs q_1d in kips per foot. The intensity of load q_1 is based on dead and live loads added since the start of settlement observations, December 1, 1948. Settlements have been taken from Appendix IV. Diameters of loaded areas at the surface of the clay have been computed from data in Appendix III, assuming that the caisson load spreads at an angle of 30 degrees to the vertical through the sand to the clay.

Only three points fall outside the area which has been shaded. These three settlement points —— 11, 16, and 21 all lie along the west wall of the building (Figure 9). Additional settlement at these points could be caused by 5 feet of fill which has been placed in the terrace area.

Points which have been circles in Figure 26 represent those settlement points in areas where the caissons are closely spaced —— under the heavy slab and the south wall (Figure 8). In general these caissons show greater settlements than those somewhat more isolated. This is to be expected if the approach considering only the upper foundation clay is valid. Actually, a strong argument can be presented for this approach especially in view of the following observation. The points plotted with an x in

Figure 26 represent the nine interior observation points. At these locations the stress in the lower foundation clay is higher than it is for exterior points. (Compare Δp curves of Figure 25.) <u>If</u> the lower foundation clay contributed a major portion of the actual caisson settlement then, the x points of Figure 26 would lie considerably below the shaded zone. The fact that they don't supports the writer's feeling that very little of the Library settlement is caused by the lower 70 feet of the clay stratum.

An equation representing the mean line through the shaded zone could be written

$$\rho = 0.1 + 0.25\ q_1 d$$

in which case the present "constant," 0.1 inch, is settlement which may be attributed to another source - perhaps consolidation of the lower foundation clay. The average settlement of the building since construction was completed has been 0.07 inches, which can account for a part of the 0.1 inch. In any event, the shaded area will gradually shift vertically downward and perhaps rotate clockwise slightly. In all probability the "constant" above will increase.

D. <u>COMPARISON OF LIBRARY SETTLEMENT WITH OTHER M.I.T. BUILDINGS</u>:

<u>Main M.I.T. Building</u>: When the main M.I.T. building (see Figure 27) was constructed in 1916 hundreds of

settlement observation points, of a type similar to those at the Library, were installed. Settlement readings, made occasionally from 1916 to the present, indicate that the settlements vary from a maximum of about 8 inches at Buildings 10 and 2 to a minimum of a little over 1 inch at Building 1.

The building is founded on wood piles which penetrate into the sand layer overlying the blue clay. In some instances where the sand is particularly thin, long friction piles were driven into the clay.

Building 2, which is nearest the Library, has foundation conditions similar to those at the Library with the exception that the sand at Building 2 is considerably thicker and lies nearer ground surface. Figure 28 shows a plan of the building and the foundation profile of the section nearest the Library.

In order to compare settlements of this structure with those of the Library a point at the center of Building 2, point A, will be selected. The computation of the stress vs depth curve below point A is based on the following assumptions. The profile of Figure 28 indicates that the net excavation is about 6 feet. Assuming a unit weight of 110 lbs per cu ft this represents a stress release of about 650 lbs per sq ft applied at El. +15.0 ft. The writer has estimated that the total dead and live building load averages 1400 lbs per sq ft over this area of the main building. This agrees closely with a value of 1500 quoted by D. W. Taylor[29]. It will be assumed that the piles transfer this stress uniformly to the sand at El. -10.0 ft.

The Westergaard stress transmission theory has been used to compute a Δp vs depth curve for point A. An estimate of the ultimate settlement using consolidation test results from samples at the Hayden Library follows:

$$\rho_u = \frac{\text{thickness}}{1+e_1} \frac{0.435\ C_c}{\frac{1}{2}(p_1+p_2)} \Delta p$$

For the upper foundation clay

thickness = 18 ft	p_1 = 1.40 tons per sq ft
e_1 = 1.00	Δp = 0.36 " " " "
C_c = 0.06	p = 1.76 " " " "

and for the lower foundation clay

thickness = 70 ft	p_1 = 2.60 tons per sq ft
e_1 = 1.10	Δp = 0.18 " " " "
C_c = 0.20	p_2 = 2.78 " " " "

in which case

$$\rho_u = 0.65 + 2.33 = \underline{3.0 \text{ inches}}$$

whereas the observed settlement is over 8 inches.

It has been demonstrated in an earlier section that laboratory consolidation tests give values of compression index, C_c, which are considerably higher than the field values for a precompressed clay. This may be true if the net loads are small but the opposite is a possibility when net loads are large. This is discussed in VII-F and IX-D. Nevertheless, the importance of the history of the M.I.T. site must not be overlooked as a possible explanation for discrepancy between the computed and observed settlement.

Prior to 1890 the M.I.T. site was a tidal flat with ground surface at about El. +7 on the Cambridge datum (mean sea level is about +11). The tip of a natural gravel point, called Whittimore's Point, projected across the present Massachusetts Avenue and touched Building 1. The dashed line of Figure 27 shows the shore line of this point where the sand and gravel overlying the blue clay is 20 to 30 feet thick.

The Harvard bridge was opened to traffic in 1891 which means that fill was placed about this time to present ground surface, El. +21, on both sides of Whittimore's Point for Massachusetts Avenue. According to the Annual Documents of the City of Cambridge for the year 1898, the sea wall south of the M.I.T. site had been constructed and the Esplanade (area now occupied by Memorial Drive) had been filled to El. +21 by hydraulic dredge from the Charles River basin. An 1898 map shows the M.I.T. site as "partly filled flats." It is believed that the remainder of the fill to El. +21 took place about 1908 when the Charles River Dam was built. In any event, by 1912 when W. O. Crosby of the Geology Department made borings throughout the area, the site was at El. +21. Between 1898 and 1912, then, about 14 feet of fill was placed at the site with the exception of the area near Building 1 where little filling was required.

Additional fill was placed, after the main building was constructed, to El. +26 to accommodate sidewalks around the interior courtyards of the building. Sometime prior to 1935 the remainder of the courtyards were filled to the present elevation, generally +26 except for a side strip down the center of the court which is about El. +24.

In view of the quantity of fill placed shortly before the Institute building was constructed and subsequent filling in the courtyards amounting to 4 or 5 feet, it is not difficult to see why the settlement of the main building is as much as 8 inches. In general the settlements increase going east from Massachusetts Avenue, which is to be expected because of Whittimore's Point and the older fill for Massachusetts Avenue. Building 1, with a settlement of 1 to 3 inches, is located above the thick bed of gravel of Whittimore's Point where the fill placed before construction is the least. Building 10, with somewhat higher building loads and founded on piles above 5 or 10 feet of gravel, has settled about 8 inches. Building 2 has about the same loads as Building 1 and the same sand and gravel as Building 10. It is located where the fill is the greatest and has settled 8 1/2 inches.

The quantity and distribution of fill placed before the main building was constructed is alone not enough to explain the <u>difference</u> between the settlements of Buildings 1 and 2. It is true that the net building load causing consolidation at Building 1 is smaller because of the thick sand stratum. What may be still more important, however, is the effect of this net load difference on the shapes of the field compression curves for the two buildings. The Terzaghi bond hypothesis and bonds related to Theory B, which have been discussed in VII-F, offer possible explanations, then, for the large differences between Buildings 1 and 2 as well as for the small settlements at the Library. A discussion of these points is withheld until IX-D.

Several other factors may contribute to the irregular settlements of the main building. One of these involves the effect of remolding on the strength of the clay caused by driving closely spaced friction piles. In 1932, A. Casagrande[30] pointed out that the maximum settlements were in areas where long piles were driven through the sand into the clay. (These areas, of course, correspond to zones where the sand stratum is thinnest.) While this is generally true, there are areas of the main building where short piles have been used, which have settled nearly as much as Buildings 10 and 2 which are supported by long friction piles. Thus, this remolding effect can be a partial explanation only.

It was assumed that the building load was distributed uniformly over its area. Actually, the load is carried to the sand by pile groups which will stress the clay in local areas considerably more than the average which were assumed in a previous computation. Where the sand is thinnest the stresses and consequently settlements will generally be greatest. Finally, as the recent fill overlying the sand and clay continues to settle, it will add load to the piles instead of contributing to their support. This effect is obviously more pronounced where the fill is deepest. As noted earlier, the areas of deepest fill are located where the main building settlement is greatest.

In view of the above discussion, it is evident that a correlation of the main M.I.T. building settlement with the Library settlement and consolidation test data, is next to impossible.

Alumni Pool Building: In 1944, D. W. Taylor[29] published a paper on the foundation of the Alumni Pool Building which is located about 700 feet north of the Hayden Library. The foundation conditions are more like those of the Library than were those of Building 2 since the sand varies in thickness from 0 to 5 feet. Gow type caissons were placed while the sand stratum was dewatered by well points. To make the similarity even closer, the net load on the building area as a whole is nearly zero.

Taylor's design criteria, to limit the differential settlement of opposite ends of the pool to a minimum, was to design each caisson so that the loaded diameter in feet at the top of the soft blue clay was equal to 0.15 times the net caisson load in tons. The average settlement of the two ends of the pool in 2 years after permanent settlement plugs were established was 0.7 inches. The differential settlement did not exceed 0.01 inches between these locations.

Limited data are available, then, to check results of the Library study based on individual bulbs of pressure, in particular to check the shaded zone of Figure 26. An average of q_1d equal to 26 kips per ft was computed for the four caissons at each end of the pool using data from an Anderson-Beckwith blueprint, number S-1x, dated June 15, 1943. This value when plotted on Figure 26 with a corresponding average settlement of 0.7 inches, gives a point in the middle of the shaded zone. This check provides further evidence, although meager, to support the conclusion that the source of the Library settlement is in the upper foundation clay.

E. <u>FUTURE SETTLEMENT OF THE LIBRARY</u>: Piezometer readings have shown that settlement at the Hayden Library due to primary consolidation is complete. If the loads remain essentially the same as they are at present, the writer believes that the Library will probably settle less than half an inch during the next 10 years. This will include general subsidence of the M.I.T. area as a whole. There is a possibility that future additions to the building will cause a substantial increase in settlement - perhaps even greater than the settlement which occurred under the present loads. It is unlikely, however, that the Library will follow a settlement pattern similar to Building 2 because of the difference in the net loads.

F. <u>CONCLUSIONS</u>: The following conclusions relative to the analysis of the magnitude of the Library settlement may be stated.

1. The use of a net Δp (stresses caused by building loads minus excavation stresses) in the ultimate settlement computation for a precompressed clay is not valid. A somewhat larger value holds, depending on the speed at which swell takes place following excavation. Since settlement estimates are often already too large, this accentuates the difference between the field compression curve and that estimated from consolidation data.

2. The small settlement which has occurred at the Library may be due to any of three sources of

settlement or a combination of them - compression of the sand, shear deformation of the clay, and consolidation of the clay. Since the settlement from all three sources occurred during construction no accurate breakdown is possible.

3. If Library settlement is due only to consolidation phenomena, the average field compression index is about 1/5 that determined from laboratory consolidation tests.

4. There is good evidence to support the hypothesis that the upper foundation clay contributed nearly all of the settlement, in which case individual caisson bulbs of pressure may be considered without the labor of a settlement analysis based on consolidation theory. If this is the case the consolidation tests have given values of compression index which average 10 or more times the field values for the lower foundation clay. Figure 26 provides a good working curve for caisson design using this approach.

5. No correlation between the Library settlement and that of the main institute building is possible largely because of the load history of the

site, the difference in foundation types, and the difference in net loads causing consolidation.

IX

ANALYSIS OF THE TIME RATE OF THE LIBRARY SETTLEMENT

The preceding section was devoted to a correlation between laboratory and field data with reference to the magnitude of the Library settlement which has occurred. This section will present an analysis of the speed at which that settlement has taken place with special emphasis on the use of pore pressure measurements and a consolidation analogy model in the correlation.

A. <u>NATURE OF THE PROBLEM</u>: It has been pointed out in an earlier section and it may be seen in Figure 17 that the piezometers recorded a change from negative to positive excess pressure about 270 days after the start of construction. (At this time the average net load at the top of the clay was about -0.15 tons per sq ft, see Figure 7.) Settlement due to consolidation phenomena commenced at 270 days, then, in which case ultimate settlement and time-settlement correlations based on conventional analyses must consider this point as the start of the loading period.

Actually, the date when settlement observations were begun, 238 days, is not far from that when positive excess pressures were initially recorded. It will be assumed that the two dates coincide and that the effective loading period begins at 238 days. Figure 29 shows the assumed linear variation of average pressure (as defined in Figure 7) over the 300-day effective loading period beginning at 238 days.

An analysis of the speed of compression is complex, primarily for two reasons. First, it is not known whether or not the Library settlement is a result of consolidation. Second, the effects of radial flow on the consolidation process are severe, especially in the upper foundation clay where the caissons stress isolated bulbs of soil which are adjacent to zones between caissons which receive no load.

Because of the uncertainties involved in the effects of radial flow in this case, the analyses which follow will deal with the determination of a modified coefficient of consolidation embracing radial as well as vertical consolidation. This coefficient, given the symbol c_m, may be thought of as the vertical coefficient of consolidation required to give the time-settlement characteristics or the pore pressure dissipation patterns if no radial flow were involved. There is no reason to believe that c_m will remain constant during the construction period as c_v might if consolidation were truly one-dimensional. The effects of radial flow will vary considerably depending on the horizontal gradients, their space rate of change, and some indeterminate radius effective at any given time.

As in the preceding section, the problem will first be approached from the standpoint that settlement is due to consolidation and the modified c_m required to give the observed settlement pattern will be computed from the Terzaghi Consolidation Theory. Use will then be made of the pore pressure curves for other determinations of c_m which are independent of the source of the settlement.

B. MODIFIED c_m BASED ON SETTLEMENT CORRELATIONS: Observed time-settlement curves for caissons 108, 44, and 50 (observation points 8, 18, and 20) have been plotted in Figure 29 from data of Appendix IV. The theoretical time-settlement pattern, based on consolidation test data, may be estimated

$$t(\text{days}) = \frac{TH^2}{c_v} = 4400T$$

where $H = 45$ ft and $c_v = 50 \times 10^{-4}$ cm² per sec.

The settlements which have occurred to date, 0.52, 0.22, and 0.72 inches respectively, represent settlement due to primary compression which is essentially complete. If these values of settlement are used, as described in VI-C, to determine the time-settlement curve it is not difficult to show that estimated settlements at the end of the loading period would be about one third those which have occurred. Consolidation has obviously progressed considerably faster than the laboratory coefficient would lead one to expect.

By a process of trial and error the dashed curves of Figure 29 have been fitted to the actual time-settlement curves using a c_v, in the above equation, equal to 800 x 10^{-4} cm^2 per sec. The curves have been corrected for the assumed loading period using a graphical process described by K. Terzaghi[15]. Actually, this elementary fitting is worthless. It gives no real indication of the comparison between laboratory and field consolidation characteristics because of the effect of radial flow and the uncertainty in the source of the settlement. If pore pressure curves were not available it would be the only approach. If the settlement were known to be caused by consolidation it would have to be concluded that consolidation took place 16 times as fast as would be predicted from laboratory tests. Note that the writer has not stated that the field c_v is 16 times the lab c_v but that c_m as previously defined is 16 times the lab c_v.

C. MODIFIED c_m BASED ON PIEZOMETER CURVES:

Introduction: The value of pore pressure measurements in the study of consolidation characteristics of a clay stratum cannot be overemphasized. These data have been used with settlement data in VIII to estimate the coefficient of compressibility of the clay. They will now be used with loading data to study the speed at which consolidation has occurred at the Hayden Library.

Analytical solutions of consolidation problems involving irregular patterns of hydrostatic excess pressures are generally too complex unless simplifying assumptions, generally at a considerable sacrifice in accuracy, are made. Graphical procedures and model analyses, however, may be adapted to nearly any condition of flow and variation in soil properties, to predict field consolidation characteristics or to study observed field data.

Given any two piezometric curves, excess pressure vs depth (isochrones), at given times t_1 and t_2, it is possible to estimate the field coefficient of consolidation c_v (if no lateral drainage occurs) by a graphical differentiation process based on the equation for one-dimensional consolidation

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

which may be approximated by

$$\Delta u = c_v \frac{\partial^2 u}{\partial z^2} \Delta t$$

The slope of the isochrone at any point represents the term $\frac{\partial u}{\partial z}$ and the rate of change of slope with respect to z is $\frac{\partial^2 u}{\partial z^2}$.

A plot of $\frac{\partial^2 u}{\partial z^2}$ vs z at time t_1 may be easily obtained using a graphical procedure described by J. P. Gould(4). For an assumed c_v, which may vary with depth, and a small increment of time Δt, a curve of Δu vs depth, which represents the excess pressure dissipated during the increment, may be determined from the formula given above. When this change is applied to the isochrone at t_1 a new isochrone is obtained for $t_1 + \Delta t$ which may be used for the next step. The isochrone obtained at time $t_2 = t_1 + n\Delta t$ may be compared with the known excess pressure curve at t_2 as a check on the assumed c_v. This process is repeated with a new c_v until the graphical procedure checks observed data.

Because of the general form of the differential equation, if a load is added between t_1 and t_2 it may be included by an increase in u at the desired time. Similarly, if the effects of radial flow are studied the differential equation representing radial consolidation may be used alternately with the expression for vertical consolidation in a step-by-step process.

The graphical procedure, being a trial-and-error process, is somewhat tedious and involves many hours of careful drafting. A second approach to the problem of analyzing field data may be made by using a model herein called the consolidation analogy model. A model has been built at the M.I.T. Soil Mechanics Laboratory by A. C. Rigas(31) under the supervision of the writer. The principle of the model is based on a device described by R. A. Barron in Reference 32. It replaces the tedious step-by-step graphical procedure by a continuous variation in excess pore water pressure. Although trial-and-error procedures are required under certain conditions and radial flow cannot be accounted for in the present model, it has proved to be invaluable in aiding the investigation which follows.

<u>The Consolidation Analogy Model</u>: The consolidation analogy model consists of a series of vertical standpipes

connected to each other by means of equal lengths of horizontal capillary tubes. The photograph of Figure 30 shows the model which was used in the following studies. A complete description of the model and a discussion of its analogy to consolidation is given by Rigas[(31)]. Only the important features will be presented here.

Each standpipe of the model represents a specific point in a consolidating clay layer. Figure 31 shows a sketch of the model and the elevations of points in the clay stratum beneath the Hayden Library which correspond to the nine model standpipes. The resistance of the soil to flow of water is simulated in the model by the capillary tubes connecting standpipes.

If the level of the water in the end two standpipes is held constant and defined as the model datum, the level of the water above or below this datum in any standpipe represents an excess pressure. Any known isochrone in a clay stratum may be represented on the model, then, by bringing the water surface in each standpipe to a level, above or below the model datum, equivalent to the excess pressure at corresponding points in the clay.

One-dimensional consolidation with a constant initial excess pressure and double drainage is represented in the model by filling the 7 interior standpipes to a common level above the model datum. The consolidation process begins when the valves are opened and water commences to flow, first from the outer standpipes and finally from the center standpipe after sufficient difference in head has been established to cause flow. Theoretically, an infinite gradient exists at the surfaces of the soil mass when consolidation begins. The initial gradient in the model is finite. The effect of this discrepancy disappears rapidly as the number of standpipes is increased. Rigas found that 3 were not sufficient but 7 gave good results.

The amount of water drained from each standpipe during an increment of consolidation represents drainage from a given soil layer. Therefore, the area of the standpipe is analogous to the compressibility, a_v, of the soil layer.

A relationship between model time and prototype time, see Reference 32, may be expressed by

$$t_p = \left(\frac{q\ \gamma_w\ a_v\ H^2}{a\ n^2\ \ell'\ k(1+e)} \right) t_m$$

where

t_p = time in the prototype

t_m = time in the model

q = rate of discharge for one capillary tube under a unit gradient

γ_w = unit weight of water

a_v = coefficient of compressibility of the soil

H = one half the thickness of the clay layer for double drainage

a = area of the standpipe (analogous to a_v)

n = number of capillary tubes

ℓ' = length of each capillary tube ($n\ell'$ analogous to the thickness of clay)

k = coefficient of permeability of soil

e = void ratio of soil

Although the model may be calibrated to a particular case in nature by solving the above expression, a more general approach is to determine t_m in terms of the time factor T. This may be done by simulating on the model the one-dimensional consolidation process with constant initial excess pressure and noting the time required for the water level in the standpipes to represent a given theoretical time factor curve. Rigas found the calibration factor for the M.I.T. model to be t_m (min.) = 70.5T at a temperature of 24°C. For a particular field problem, t_p may also be expressed as a function of T from $t_p = \frac{TH^2}{c_v}$, in which case the relationship between t_p and t_m is immediately available and applies for the c_v and H assumed.

Once this time relationship is determined, the model may be used in conjunction with building load data to predict the pore pressure curve throughout the clay at any time. This process is most easily accomplished by adding the "load" in steps. Each building load increment causes an increment of excess pressure throughout the clay which may be represented in the model by adding appropriate amounts of water to the standpipes. Drainage in the model is allowed to take place for the model equivalent of t_p before the valves are closed and the next step applied.

Although the consolidation model may be adapted to include cases where the soil undergoes both expansion and compression at different coefficients, and perhaps even radial consolidation, it has been used in the following analyses in the simple form described above.

Application of the Model: Three independent studies have been conducted using the consolidation analogy model to estimate the speed at which consolidation has occurred at the Hayden Library. These studies, called Case 1, 2, and 3, are based on loading data described in III and piezometer data presented in V.

The following general assumptions are made in these cases:

1. 100 per cent of any stress increment, Δp, transmitted to a given point in the soil mass is initially carried by the water as excess pressure.
2. The coefficient of expansibility, a_e, is equal to the coefficient of compressibility, a_v, and they are constant with depth.
3. The modified coefficient, c_m, determined by the model analyses is understood to be that average

value of c_v required to give the observed pore pressure dissipation pattern if the flow were one-dimensional in the field.

In order to calibrate the model to field conditions the following computations are made:

$$t_m(\text{min}) = 70.75T \text{ (from Rigas) - - - - - - (A)}$$

and for a value of c_m equal to 1×10^{-4} cm^2 per sec

$$t_p(\text{sec}) = \frac{TH^2}{c_m} = \frac{(45 \times 30.5)^2\ T}{1 \times 10^{-4}}$$

or $$t_p(\text{days}) = 218{,}000T \text{ - - - - - - - - - - - - (B)}$$

Combining (A) and (B):

$$t_m(\text{min}) = \left[3.24 \times c_m(\text{cm}^2 \text{ per sec})\right] t_p(\text{days}) \text{ - (C)}$$

for any value of c_m.

CASE 1

The first study, a preliminary one, was conducted primarily to estimate a modified c_m to be used as a first trial in Cases 2 and 3.

Pumping from a well point system at the site of the John Thompson Dorrance Laboratory, 500 feet north of the Library, lowered the water table about 3 feet at Group A piezometers and 1 1/2 feet at Group B. The effect of the drawdown on the piezometer readings may be seen in Figure starting at 970 days.

Let us assume in this preliminary study that the water table was lowered 3 feet over the entire Library and surrounding area. This corresponds to the application of an initial hydrostatic excess pressure at the top of the clay of 3 feet varying linearly to zero at the bottom of the clay. At 970 days the piezometers were showing very little change. Therefore, the excess pressure caused by the drawdown may be analyzed separately in the model for the short time involved.

The above condition may be represented on the model by simply filling each standpipe the appropriate amount above the model datum (2.625 units in #2, 2.25 in #3, etc., for a triangular distribution of initial excess pressure) before opening the valves and allowing drainage to commence. The model curve at any time, t_m, may be fitted to a prototype curve using the Group A piezometer readings from Figure 16a. When the t_p - t_m relationship is found, equation (C) may be used to determine the modified c_m.

This preliminary study indicated that the c_m applying to the first 30 days after drawdown was equal to 800 to 900 x 10^{-4} cm² per sec.

CASE 2

A second study, based on the period from 230 to 700 days, was made to demonstrate graphically the speed at which consolidation has progressed at the Hayden Library.

Before the test was started, the water level in each standpipe was lowered an amount equivalent to the negative excess pressures given by the 230-day curve in Figure 17 for Group B piezometers. In this case, 2 cm on the model standpipes represented 1 foot of excess pressure head. The loading curve shown in Figure 32 was assumed to be applied in steps. Influence values of excess pressure head caused by a step of 1 ton per sq ft applied at the surface of the clay, are recorded in the table of Figure 32 for points in the clay stratum corresponding to standpipe locations.

To illustrate the model procedure - after the 230-day excess pressure pattern was reproduced on the model, each standpipe was filled an amount representing the first step of 0.025 tons per sq ft (see Figure 32). This amount may be determined by multiplying the influence value shown in the table by 0.025. Valves were then opened and flow was allowed to take place for "45 days" ($11^m 40^s$ on the model for a c_m equal to 800×10^{-4} cm² per sec). At this time, the valves were closed and the next step of 0.05 tons per sq ft was applied. Readings, taken at the middle of several steps, are plotted in Figure 33 for comparison with the actual hydrostatic excess pressure patterns.

Although the actual piezometer curves fluctuate back and forth considerably there is reasonable agreement between the model and actual curves. If greater refinement were warranted, better agreement could be obtained by varying the sizes of the standpipes to simulate a varying c_m and by taking smaller steps. The value of 800×10^{-4} cm² per sec is believed to be within 10 per cent of an optimum value giving best agreement.

The average consolidation has progressed 14 to 18 times as fast as would be normally predicted from laboratory consolidation test results. The effect on the piezometer readings is apparent. While an excess pressure equivalent of 11 feet of head was applied during the period, a maximum of 2 feet was recorded by the piezometers.

CASE 3

A third study was made involving the period from the start of construction to the time when the first piezometer readings were taken (about 200 days). The object of this study was to determine the average modified c_m required to duplicate by means of the model the 197-day excess pressures recorded by Group B piezometers.

Loading data for this period are considerably more precise than those used in the second study. With the aid of data collected by H. de R. Gibbons[(6)], the stress intensity causing consolidation at the location of piezometers B-2, B-3, and B-4 has been computed by means of the Westergaard stress transmission theory. These curves are shown in Figure 34. The sudden jump at 37 days does not represent a rise in the piezometer levels but indicates a sudden change in the line of zero excess pressure as a result of the Library drawdown.

A step process was followed similar to that used in the previous study. The steps are shown in the table of Figure 24. The results of a trial value of c_m equal to 1000 x 10^{-4} cm² per sec are shown in Figure 35. The model curve at 197 days does not check the actual curve as well as it would if a slightly higher value of c_m were used. Rigas[31], however, estimated that it would require an average c_m of 950 x 10^{-4} cm² per sec to check the 197-day curve.

One of the general assumptions used in these analyses was equality of a_v and the swelling coefficient a_e. This assumption is probably very crude in this case since the a_e applying to swell after excavation and swell following the discontinuation of pumping is smaller than the compressibility coefficient a_v during drawdown. This fact, coupled with the smaller effects of radial flow during the drawdown, would lead to a considerably larger modified c_m during swell than that which applied during drawdown.

This case provides further emphasis on the speed at which consolidation took place at the Hayden Library. The fact that positive excess pressures were recorded when the net building load was still negative is a result of rapid foundation swell following excavation and discontinuation of pumping. Presumably, if enough time were allowed following the excavation for a building, negative excess pressures would be completely relieved. In this case, the first ton of building weight would cause positive excess pressure and consolidation of the clay.

Since the piezometer readings at the Hayden Library were very small, it was originally thought that only a small share of the building load was carried initially by the pore water. Although there is no definite proof that the pore water carried 100 per cent of the load, these studies indicate that the problem was much more likely one of speed of compression rather than bond. Unfortunately, there was no appreciable sudden load applied during construction which could be used as a check on possible bonds.

D. <u>COMPARISON OF TIME RATE OF LIBRARY SETTLEMENT WITH OTHER BUILDINGS</u>: The speed at which consolidation has taken place at the Hayden Library is indeed surprising. However, it is not as unusual as it might at first seem. The effects of radial consolidation have undoubtedly been pronounced. Furthermore, the performance of other buildings with very small net loads has shown that the high speeds of compression cannot be accounted for from laboratory consolidation tests. As an illustration, A. Casagrande and R. E. Fadum[23] published a paper in 1944 concerning the settlement of two large insurance buildings in Boston. Both of these buildings were designed with deep basements to reduce the net building load. Foundation conditions were similar to those at the Library except that the clay at the site of the insurance buildings is about 20 feet thinner. In the first year after construction both buildings had settled a maximum of less than 1 1/2 inches and settlement virtually ceased at this time.

As discussed in VIII-D, settlement records are available for the main M.I.T. building. Although the load history of the site is complex, it is of interest to examine the time-settlement curves, shown in Figure 36, for Buildings 1, 2, and 10. It is estimated that 90 per cent consolidation occurred in 10 years for Buildings 2 and 10, in which case:

$$c_v = \frac{T_{90} H^2}{t_{90}} = \frac{0.848\ H^2}{t_{90}}$$

where

$$H \sim 45 \text{ ft}$$

$$t = 10 \times 365 \times 24 \times 3600 \text{ sec}$$

then

$$c_v = 50 \times 10^{-4} \text{ cm}^2 \text{ per sec}$$

which is equal to the average value given by consolidation tests on the Library samples! Consolidation appears to have taken place about twice as fast at Building 1 where the net loads are considerably smaller because of the thick sand stratum overlying the clay.

Several cases of buildings with net loads ranging from 0.3 to 2.0 tons per sq ft are reported by A. Casagrande and R. E. Fadum[23]. In all cases the time-settlement curves indicate that consolidation has progressed at a rate similar to that of Buildings 2 and 10.

It is evident that the ultimate settlement and the speed at which it occurs are extremely sensitive to the magnitude of the net load. In the case of Boston blue clay it appears that if the net building load exceeds perhaps 0.2 or 0.3 tons per sq ft, the clay suddenly breaks down. The performance of the Library and the insurance buildings with small net loads, cannot be justified from standard consolidation tests but the settlement characteristics of buildings with large net loads may be predicted with reasonable accuracy.

It may be seen from Figure 36 that Building 1 has not settled measurably in the last 25 years while Buildings 2 and 10 have continued to settle. From this observation alone, questions are raised relative to secondary compression in the field when small net loads are involved. Secondary compression in the laboratory generally follows a sloping straight line on a logarithmic time scale. The settlements of Buildings 2 and 10 with <u>large</u> net loads tend to agree although many more years will be required before this relationship can be proved. On the other hand, Building 1 follows no such secondary time-compression relationship.

To the writer, the Terzaghi bond hypothesis seems to answer many of the points discussed in the previous paragraphs. The "solid water" bond, to which Terzaghi refers, may be a result of a preferred orientation of water molecules in the adsorbed water films surrounding clay particles. If sufficient load is applied to the clay, this bond and molecular orientation may be suddenly partially destroyed. According to Terzaghi the unbonded pressure-void ratio curves are parallel regardless of the magnitude of the pressure when the "solid water" bond was broken. D. W. Taylor has pointed out in Reference 12, with good reason, that a more rational action would be for the clay, in the unbonded state, to follow a unique pressure-void curve regardless of the magnitude of the pressure.

D. W. Taylor's bond hypothesis associated with Theory B (see VII-F) explains discrepancies between computed settlements and actual settlements when small loads are applied. Furthermore, from pore pressure tests described in Part II, the writer has found that under certain conditions bonds do carry a small part of the applied load. Studies at the Library, however, have shown that no pronounced bond of this type was effective. In addition there is no explanation in Theory B for the apparent sudden breakdown when the net building loads exceed 0.2 or 0.3 tons per sq ft.

It is the writer's belief that nearly all clay deposits, with the possible exception of fresh marine clays, are slightly precompressed. It is inconceivable to think that even the so-called "normally" consolidated clays have not experienced higher loads than they now carry. Precompression may have been caused by dry periods when the water level was lower by several feet, by temporary loads or by soil which has been eroded. It is indeed possible that most clays have preconsolidation pressures of at least a few tenths of a ton per sq ft. If the net building load is smaller than this precompression, the field e - log p curve will be practically horizontal and whatever small settlement occurred would take place during construction.

In the Boston area the upper part of the "blue" clay stratum is generally highly preconsolidated by drying. It is probable that the lower part of the clay is slightly precompressed even at the bottom. [Perhaps the true maximum past pressure curve (see Figure 6) is displaced slightly to the right of the overburden pressure curve in the bottom 40 feet of clay and is more nearly parallel to it than curve 3.] If the net building load exceeds this slight precompression, the lower clay will follow a virgin compression curve at a small coefficient of consolidation. The writer believes that if this precompression is exceeded, the building will exhibit a secondary time-settlement curve which follows a straight line on a logarithmic time scale. If it is not exceeded, the settlement may virtually cease in a few years.

A final decision on the validity of these hypotheses must be postponed until further laboratory research is conducted and additional field data are gathered under more ideal conditions than those encountered at the Library.

E. <u>CONCLUSIONS</u>: The following conclusions may be stated relative to the speed at which consolidation has occurred at the Hayden Library:

1. Any process of fitting theoretical time-settlement curves to observed readings in an attempt to determine the speed at which consolidation has occurred at the Library gives no real indication of the validity of laboratory consolidation test data. Since settlements at the Library took place during construction, there is no proof that the settlement is due to consolidation.

2. With the aid of piezometer readings, however, it has been possible to estimate that consolidation of the clay stratum took place 14 to 18 times as fast as would normally be predicted from one-dimensional consolidation tests. Since the effects of radial flow cannot be reasonably evaluated during construction, it is impossible to determine a value of c_v in situ to compare with the values determined in the laboratory. In all probability, however, the average field c_v is 5 or more times the laboratory value of 50×10^{-4} cm^2 per sec.

3. There is no evidence available that intergranular bonds in the clay stratum initially carried a portion of the building load.

4. The speed at which consolidation occurred at the Library is not surprising when comparing it with other large buildings which have small net loads. The reader is referred to IX-D for a discussion of the importance of the net building load magnitude.

PART I

GENERAL CONCLUSIONS

The piezometer and settlement point installation at the Hayden Library have given a great deal of valuable data which have been used in this investigation to correlate field performance with results of laboratory consolidation tests. The following general conclusions may be stated relative to these installations.

I. PIEZOMETER INSTALLATION:

1. The nonmetallic type piezometer devised by A. Casagrande has proved itself an economical and reliable device for obtaining pore water pressures in clay. All 10 piezometers continue to record the pore pressure fluctuations in the clay and at its surfaces after having been carefully installed nearly 3 years ago.

2. The piezometers have shown that there are no artesian pressures at the bottom of the clay in the M.I.T. area. As a result the maximum past pressures in the lower foundation clay must be at least equal to the overburden pressure. However, laboratory consolidation tests have given a maximum past pressure, determined by the Casagrande construction,

nearly 1 ton per sq ft less than the overburden pressure. Here is definite proof, then, that the graphical construction gives, in some instances, values of maximum past pressure which are too small.

3. The piezometers have demonstrated the speed at which foundation swell takes place following excavation. The point of zero net load for a building with small net loads must be taken at that point when the piezometers record a change from negative to positive excess pressures. Since swell usually occurs rapidly the net building load to be used in a settlement analysis may be very nearly equal to the gross building weight.

4. Without piezometer data it would have been impossible to obtain a reliable estimate of the speed at which consolidation occurred at the Hayden Library, since the settlements were small and occurred during construction. Studies based on piezometer data indicate that consolidation has taken place an average of 14 to 18 times as fast as would normally be predicted from consolidation test data.

5. Piezometer data have been used to a limited extent to support the writer's conclusion that the average in-situ compression index is about one-tenth that

which is obtained from a standard laboratory consolidation test.

II. SETTLEMENT POINT INSTALLATION:

1. The caisson type foundation designed by the firm of Moran, Proctor, Freeman and Mueser, foundation engineers, has been entirely satisfactory in transmitting the building loads to the foundation clay. The settlements which have occurred (0.25 to 0.94 inches) are considerably smaller than those which were expected. The maximum differential settlement of 0.7 inches is not considered large enough to cause any appreciable structural damage to the building. This is especially true since the majority of the settlement occurred during construction.

2. The maximum settlements in general occur around the exterior of the building. Normally, a building founded above a thick stratum of clay will settle more in the center. This discrepancy is explained in part by the smaller average loads in the center of the building because of the court. In addition, the existing exterior caisson loads are more nearly equal to the design loads than are the existing interior caisson loads.

3. The settlement pattern at the Hayden Library has led the writer to the conclusion that the majority of the settlement is due to compression of the upper 10 to 20 feet of the clay.

In view of the above conclusions and observations arrived at from a study of settlement and piezometer data, the following important conclusion has been reached.

Data from standard laboratory consolidation tests can in no way be used to predict the settlement and time-settlement characteristics of a building of this type with small net loads. A settlement analysis based on a stress transmission theory and using a laboratory value of compression index to determine the ultimate settlement at a point is a waste of the investigator's time.

More specific conclusions relative to the analyses of the magnitude of the Library settlement and the time rate at which it occurred are given in VIII-F and IX-E. The reader is also referred to VIII-D and especially IX-D for an important discussion of the comparison of the Library settlement with other buildings in the M.I.T. and Boston area.

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While a vast amount of valuable data are available as a result of the piezometer and settlement point installation at the Hayden Library, their full value cannot be realized because of the absence of certain critical information.

The most important data which are not available, the absence of which has frustrated the writer throughout this entire investigation, are data on the source of the Library settlement. It is not known to what extent the 90-foot thick layer of Boston blue clay contributed to the total measured settlement of the Library. If underground reference points could have been placed prior to excavation, the amount of foundation swell and subsequent recompression would be recorded. The value of this investigation, then, would have been increased several fold.

The fact that piezometer readings are not available for the early stages of construction is also unfortunate. The sudden load release due to excavation and the sudden drawdown when pumping was begun would have been ideal periods for studying the consolidation characteristics of the clay mass. In all probability some idea of the variation of the coefficient of consolidation with depth and the effects of radial flow could have been evaluated.

As a final hindrance to the original objectives of this investigation, the Library net loads are so small that the settlement and pore pressure dissipation took place almost entirely during construction.

It is not difficult to set up the condition for an ideal installation for studying the physical and mechanical properties of a clay stratum. These conditions would include:

1. A stratum of clay as homogeneous as possible and preferably slightly precompressed.

2. A program of carefully run consolidation tests with pore pressure measurements on undisturbed samples for studying bonds.

3. Piezometer installations throughout the clay at three locations ---, center of the building, edge of the building and at some distance away from the building. These installations to be made prior to any construction activity.

4. Underground reference points throughout the clay in at least 3 different locations --- also to be installed prior to construction.

5. A permanent benchmark established within the building area prior to excavation.

6. A fairly large, relatively flexible building with a one-story basement and a net load of perhaps 0.5 tons per sq ft or more. The building should be regular in size and shape and should transmit a

uniform load to the foundation soil. Settlement observation points to be placed throughout its basement as soon as possible.

The writer realizes that the above ideal conditions probably can never be met simultaneously. Nevertheless, it is believed that only through installations of this type will the soils engineer gain a thorough understanding of the validity of laboratory data and existing theories for predicting the action of a clay stratum in situ.

PART II

CONSOLIDATION TESTS ON UNDISTURBED SAMPLES WITH PORE PRESSURE MEASUREMENTS

X

INTRODUCTION

A. GENERAL: At the time the research described in this section was begun, there was considerable question as to whether the pore water in the foundation clay at the Hayden Library was initially carrying 100 per cent of the stress produced by the building load. The Terzaghi theory, and therefore analyses based on the theory, assume that the initial hydrostatic excess pressure in the pore water at a point is equal to the induced stress there. The fact that the piezometers recorded a maximum excess pressure head of only 2 feet was thought to be an indication that a major part of the building load was being carried directly by the soil grain structure in the form of a bond.

Further study described in IX has shown, however, that consolidation took place so rapidly that the excess pressure dissipated nearly as fast as the load was applied. Unfortunately, though, there is no field evidence available which can be used to definitely state what percentage, if any, of the building load was carried initially by intergranular pressure.

In the start, this research was aimed mainly at obtaining data on bond and viscous resistance during the laboratory consolidation test by means of pore pressure measurements. It was hoped that these data, used with Theory B developed by D. W. Taylor[12], would aid in the analysis of the pore pressure measurements at the Hayden Library. It became apparent, however, that the Library case was more a question of speed of consolidation rather than bond phenomena. In addition, the limited number of pore pressure tests conducted on undisturbed samples did not show the plastic resistance characteristics that Taylor found on remolded clay.

B. OBJECTIVES OF THIS RESEARCH: In view of the above introductory comments, the objectives of this research as reported in this thesis may be summarized as follows:

1. To develop apparatus and techniques for measuring pore pressures during laboratory consolidation tests.

2. To make preliminary comparisons of the pore pressures obtained on undisturbed samples with those which would be predicted from consolidation theories.

3. To compare the coefficients of consolidation obtained by the square root of time fitting method and the log time fitting method with those determined from pore pressure measurements.

4. To study the effect of sample size on the pore pressure and also on the coefficient of consolidation, compression index and maximum past pressure for an undisturbed clay.

5. To study the effect of a small load increment applied to a sample which had been loaded past its own maximum past pressure, rebounded to a lower pressure, and allowed to remain there for various periods of time.

C. SCOPE: The laboratory investigation described in this section is of secondary importance to the objective of this thesis which is to analyze foundation stresses and settlements at the Hayden Library. As such, the studies described will often be treated in a qualitative sense offering preliminary conclusions, in particular as they may affect the Library, rather than as final theoretical quantitative analyses.

It is believed that the consolidation tests with pore pressure measurements has exceptional research possibilities for studying many of the variables which affect the determination of field characteristics from laboratory tests. The description of apparatus and techniques used is therefore given considerable weight.

D. <u>REVIEW OF PAST PORE PRESSURE RESEARCH</u>: Although the Terzaghi theoretical consolidation process involves a stress transfer from excess water pressure to intergranular pressure, little has been done to investigate pore pressures during the laboratory consolidation test as a check on the theoretical approach. D. W. Taylor(12) published a theory in 1942, called Theory B, accounting for plastic resistance to compression which is based on pore pressure data. Thus, the first extensive program where pore pressures were measured during consolidation was reported.

Tests reported by Taylor were all run on remolded Boston blue clay with pore pressures measured at the bottom of a sample which was drained at its top surface only during consolidation. The pore pressure curves obtained were described as those which would exist at the center of a sample twice as thick with double drainage. A pore pressure measuring device(12,p.61), which consisted of a small capillary tube mounted vertically in a copper jacket, was attached to the base of the consolidation device. The bottom porous stone and passages between the base and the measuring device were filled with deaired water to a point half way up the capillary tube. Water pressures at the base of the sample were measured during consolidation by applying a balancing air pressure to the water meniscus in the capillary tube. The air pressure required to keep the meniscus at a given location (no flow or null conditions at the bottom of the sample) was taken as the pore water pressure.

Taylor found that on remolded clay there was a considerable amount of plastic resistance during consolidation which gave higher intergranular pressures than would normally be predicted by the Terzaghi theory. This effect was most pronounced when the load increment ratio was smallest. Theoretically there should be no variation in the coefficient of consolidation with load increment ratio at a given pressure. Consolidation test results showed, however, that when data were interpreted by the Terzaghi theory there was a definite variation - smaller load increment ratios giving lower coefficients of consolidation. This trend disappeared when results were computed by Theory B.

W. Enkeboll[13] has reported on an extensive series of consolidation tests run in the M.I.T. Soils Laboratory on remolded Boston blue clay. In his "Investigation of Consolidation and Structural Plasticity of Clay" pore pressures were measured in the same manner as described above. Enkeboll's primary objective was to study the effect of size of the test sample on the consolidation characteristics.

To the writer's knowledge the only other consolidation-pore pressure work has been done by P. J. Marsal(14) who conducted similar tests in the M.I.T. Laboratory in 1944. Although he was primarily interested in Theory B as applied to undisturbed samples, Marsal had difficulty correlating results because of scatter in the test data. In addition, the writer believes that Marsal was plagued by time lags in some of his more important tests on undisturbed clay.

To date then, the majority of pore-pressure research has been conducted on remolded soils. While this approach is the most feasible for gathering consistent data for comparison studies, it precludes the importance of the natural structure of an undisturbed clay. That this omission is critical may be seen easily by studying building settlement data and noting the importance of the load increment on the magnitude and speed of the settlement. This point has been discussed in detail in an earlier section.

The writer hopes that the preliminary research on undisturbed samples described in the following section, which employs a different procedure for measuring pore pressures, will stimulate general interest in the value of such studies.

XI

TESTING PROGRAM

A. <u>OUTLINE OF CONSOLIDATION TESTS CONDUCTED</u>: The testing program for this investigation consisted of 2 series of tests. The first, Series III-V, is the main exhibit of this research. Three standard size samples and 3 samples 9.55 inches in diameter were tested with pore pressures measured during consolidation. In this series the clay sample was encircled by a thin cylindrical rubber membrane during the test. The contact surfaces of the membrane and the consolidation barrel were lubricated with colloidal graphite to reduce side friction. The primary purpose of the membrane was to allow the taking of pore pressure measurements.

A second series of tests, called Series F, consisted of 8 tests run specifically to justify the use and to study the effect of the rubber membrane. No pore pressures were observed during these tests.

A more complete description of the 2 series of tests, all run on undisturbed Boston blue clay, is given in the following outline form:

<u>SERIES III-V</u>: Nos. III-1 to III-3, V-1 to V-3

Number of tests = 6

Load Increment Ratio $\frac{\Delta p}{p_1}$ = 1 (Generally)

Load Increment Duration = 24 hrs (Generally)

Set III - Diameter = 4.25 in, Thickness = 1.25 in

Set V - Diameter = 9.55 in, Thickness = 4.00 in

All samples loaded to 8 kg per sq cm then rebounded to 4. Various times were allowed on rebound load before reloading in two increments, 4-5 and 5-6 kg per sq cm. No. 1 test of each set identical in every respect except size; similarly No. 2 and No. 3.

Pore pressures measured during consolidation.

Tabulated data and compression and pore pressure curves given in Appendix VI.

<u>SERIES F</u>: Nos. F-1 to F-8

Number of tests = 8 (4 with membrane, 4 without)

Diameter = 4.25 in, Thickness = 1.25 in

Load Increment Ratio $\frac{\Delta p}{p_1}$ = 1 (Generally)

Load Increment Duration:

Tests F-1 to F-4 = 12 hrs (Generally)

Tests F-5 to F-8 = 24 hrs (Generally)

Various load increment ratios and durations were used on rebound and reloading to study effect of friction.

Tests were run side by side in sets of 2, one with membrane and one without. Both samples were cut from the same soil stratum and loaded identically.

No pore pressure measurements.

Tabulated data given in Appendix VII.

Summary of results given in Figure 41.

B. SOIL USED FOR CONSOLIDATION TESTS:

Location and Description: All undisturbed Boston blue clay samples used in this investigation were cut from large "pit" samples which were carefully carved from a slope at the New England Brick Company pit in Cambridge, Mass. The cut has been exposed by a steam shovel shortly before the samples were taken.

Series III-V samples came from the same 6-inch stratum. The writer was fortunate in locating a layer which was remarkably homogenous. Figure 37 shows a true scale photograph of sample D which was used for the small diameter tests, Set III. This specimen was partially dried to bring out the stratification shown. Plotted beside the photograph are 4 natural water content vs depth curves. The first is that for sample D itself while the other 3 are natural water content curves for pit samples which were used for Set V tests. These water contents alone point out the homogenity of the samples used in Series III-V tests.

Series F samples were taken from a different stratum which had somewhat more stratification. A large enough sample was originally cut, however, so that samples for each pair of tests could be cut side by side.

<u>Classification Tests</u>: The specific gravity of the clay was assumed to be equal to 2.78. This value has been found to be fairly consistent for Boston blue clay.

Two Atterberg Limit tests were run on a portion of the clay used in Series III-V tests which has a natural water content of 50 per cent. The following average results were obtained:

Liquid Limit	=	54.0%
Plastic Limit	=	27.5%
Plasticity Index	=	26.5%

The liquid limit of the clay is only slightly higher than the natural water content which indicates that the clay is extremely sensitive to disturbance. The clay is very "brittle," becoming soft and sticky on remolding.

<u>Unconfined Compression Tests</u>: In order to obtain a measure of the undisturbed shear strength and the sensitivity of the clay used in Series III-V tests, 4 unconfined compression tests were run on undisturbed samples. These tests, run at a uniform rate of axial strain, were performed on samples 1.44 inches in diameter and 3.5 inches high. Results of the 4 tests follow:

Test No.	$\frac{1}{2}$ Unconfined Compressive Strength, $\frac{1}{2}\frac{P}{A}$	Axial Strain at $\frac{1}{2}\frac{P}{A}$
1	688 psf	3.4%
2	623 psf	1.1%
3	812 psf	0.9%
4	680 psf	1.1%

One test on a remolded sample showed no strength whatsoever.

The fact that the clay is extremely brittle is further emphasized by the very small axial strains when the peak point was reached on the undisturbed samples. The writer has no explanation for the large axial strain in Test No. 1, unless it was caused by improper trimming of the ends of the sample.

C. APPARATUS: One of the first problems which was encountered in this research was that of verifying previous pore pressure readings which had been taken by measuring the water pressure in the bottom porous stone chamber of a single drained sample. In preliminary pore pressure tests of this type, the writer had difficulty deairing the chamber. Serious time lags resulted which caused pore pressures to be considerably smaller than those which would normally be expected. Further discussion of time lags is given in E of this section. It suffices to say at this time that because of the questionable values of pore pressures obtained and the difficulty of getting readings shortly after the load was applied (especially important in undisturbed samples where primary compression is likely to occur in a few minutes) the writer found it necessary to develop a new technique for measuring pore pressures.

Pore Pilot and Pressure Measuring System: This method of taking pore pressure measurements utilizes a thin brass pore pilot which is thrust into the center of the sample. It is similar to that which has been used in triaxial pore pressure research at M.I.T. since 1942. At the outer end of the pore pilot, shown in Figure 38b, are 3 layers of 200-mesh screen. Water pressures are transmitted from the clay through the pore pilot and a short length of small diameter Saran tubing to a capillary tube measuring device of the same type used in previous research. The pore pilot and capillary tube assembly used in Set V tests are shown in Figure 39b. Figure 38a shows a close-up of the assembly with the pore pilot in the center of a small diameter sample (Set III tests). The pore pilot moves freely down in a slot in the side of the barrel as the sample consolidates. A thin sliding disk (Figure 38b) fits over a nipple in the rubber membrane through which the pore pilot is inserted. The disk serves to prevent the clay from bulging the membrane at the slot in the side of the barrel.

The pressure measuring system used for Series III-V tests is shown in the photograph of Figure 39a. The pore pressure was determined simply by applying a balancing pressure from a nitrogen tank to the top of the meniscus in the capillary tube. When the water level in the tube remained stationary, indicating a no flow condition, the correct pore pressure was read from a water manometer, mercury manometer or pressure gage depending on the magnitude of the pressure.

Consolidometers: Consolidometers for Series F tests were identical with those used in standard consolidation tests at M.I.T. A description of the fixed-ring unit is given in Reference 2, page 212. A consolidation barrel 2.5 inches high was used for Set III tests. This was necessary, even though Set III samples were only 1.25 inches thick, because a 1.25-inch barrel did not allow sufficient clearance above the base for inserting a pore pilot.

Figure 39b shows the consolidation equipment which was used for the large diameter tests, Set V. Since samples in this set were 4 inches thick while the barrel was approximately 7 inches tall, several loading plates, as shown in the figure, were required. The top and bottom porous stones were made of sand cemented with Portland cement in the following proportions: Sand 1100 cc, cement 300 cc, and water 190 cc. The top stone had a cast-in-place brass ring giving about 0.02-inch clearance with the barrel.

Rubber Membrane: All samples of Series III-V tests were contained in a 0.01-inch thick cylindrical rubber membrane during testing. This cylindrical membrane was cut from a piece of #30 rubber sheeting and cemented with tire patch. Colloidal graphite was smeared on the outside of the membrane and the inside of the consolidation barrel to reduce side friction.

The bottom edge of the rubber membrane was wrapped around the lower projection of the consolidation barrel and was bound to the barrel with elastic bands. The membrane passed between the soil sample and the consolidation barrel and extended slightly above the barrel where it was free to move down as the sample consolidated.

Figure 38b shows a photograph of the membrane used in the small diameter tests. A thin rubber nipple, through which the pore pilot was inserted, was cemented to the outside of the membrane. A seal was effected during testing by binding the nipple to the pore pilot by means of rubber banding.

Loading Devices: The standard M.I.T. loading device[2] was used for all small diameter tests. This loading unit is essentially a platform scale with a capacity of 1000 pounds. It may be overloaded to 1600 pounds to obtain a pressure of 8 kg per sq cm on the 4.25-inch diameter samples.

A larger scale, shown in Figure 39a, was used for the 9.55-inch diameter tests. In order to obtain 8 kg per sq cm on these samples, it was necessary to employ a leverage system designed by Enkeboll and described by him in Reference 13. The ratio of total load on the platform scale to reading on the balancing arm as calibrated by the writer is 4.25:1.

D. TECHNIQUES:

Dealring the Pore Pilot and Capillary Tube Assembly: In order to obtain reliable pore pressure readings extreme care has to be exercised in deairing and saturating the pore pilot and capillary tube assembly. For the following discussion, reference will be made to Figure 39. The pore pilot is detached from the capillary tube and Saran tubing and deaired separately by boiling in distilled water. To deair the remainder of the unit the connection labeled "to pressure system" is attached to an auxiliary line running to a vacuum bottle which is connected to a vacuum source. The free end of the Saran tubing is also connected to the vacuum bottle by means of an additional length of tubing. The connection labeled "to deaired water supply" is attached to the glass standpipe shown to the right of the manometer. With the valve "to deaired water supply" closed, a vacuum is applied to the system for 10 minutes or more. Boiling distilled water is then poured in the glass standpipe and allowed to be drawn through various passages of the capillary tube assembly, as the vacuum is gradually released.

After the system has cooled a small flow is started from the deaired water supply while the lines to the vacuum bottle are disconnected. The free end of the Saran tubing may be placed in a beaker of deaired water until the unit is ready to be connected to the pore pilot just before the test is begun. Water is brought to a given level in the capillary tube by lowering the deaired water supply below the capillary tube. After the unit has been connected to the pressure measuring system, this part of the preliminary preparation is complete.

Preparing the Sample: Two different methods were used, depending on the specimen size, to prepare the sample for testing. The techniques are treated separately below:

Set III (small diameter): In this case the sample is initially trimmed to the proper diameter and thickness. It is then placed in the cylindrical rubber membrane which has been stretched by mounting it on the inside of a barrel somewhat larger than the specimen. After removing the barrel, the outside of the membrane is given a heavy coating of colloidal graphite. The rubber nipple is pulled through the sliding disk (Figure 38b) and the combination is slid into the consolidation barrel.

Considerable care must be exercised in trimming the sample to a diameter such that it will slide snugly into the barrel without wrinkling and displacing the membrane. A split consolidation barrel was used in one preliminary test but the barrel broke apart when the 8 kg per sq cm load was applied.

Set V (large diameter): This method involves sliding the sample into the consolidation barrel which already has the coated membrane mounted inside it. After the sample has been trimmed to the proper size, the sides are coated with a thin slurry of clay and water so the sample will slide easily along the membrane without adhering to it. The effect of coating the sample is considered to be negligible especially in view of the specimen size.

Inserting the Pore Pilot: Before preparing the soil samples in the humid room, the pore pilot, which has been boiled in distilled water, is attached to a deaired distilled water reservoir by means of small diameter rubber tubing. While inserting the pore pilot through the rubber nipple into the sample, a small head is applied to create an outward flow from the pore pilot. The nipple is bound to the pore pilot, after it has been inserted, with a short length of rubber banding.

This temporary connection to the pore pilot is used until the sample is brought to the loading unit and nearly ready for testing. At this point, the capillary tube unit is attached to the pore pilot while a small flow of water is maintained at all times from the Saran tubing.

The presence of air in the line can be easily detected by applying a sudden pressure to the meniscus in the capillary tube. A sudden drop in the water level followed by a steady fall indicates air. The procedure outlined above is simple enough that if a reasonable amount of care is used there will be no noticeable air in the system. The same general techniques have been used in triaxial pore pressure research at M.I.T. since about 1942. The most recent techniques are described by Clough[33].

Observing Data During Test: Loading schedules for the various tests are given under XI-A. Three types of data were observed during each load increment - time, compression dial readings, and pore pressure readings. The technique used for each load increment may be outlined as follows:

1. A pressure equal to the pore pressure which was expected to develop initially was applied to the top of the meniscus. (The valve to the pore pilot is closed.) This pressure could be predicted closely after the first test had been run.

2. A timer was started back of zero and the load was applied as the timer passed zero.

3. As the valve to the pore pilot was gradually opened, the pressure was adjusted until the meniscus in the capillary tube did not move.

The first pore pressure reading was occasionally made at 15 seconds, but was generally taken at 34 seconds.

4. Readings of time, dial, and pore pressures were taken at given times until the pore pressure was nearly zero. The meniscus in the capillary tube was maintained at its starting level for the entire increment.

E. <u>VERIFICATION OF DATA</u>: There are two important differences between the consolidation - pore pressure tests which have been run for this investigation and standard consolidation tests. They are the actual determination of pore pressures and the use of a thin rubber membrane. It is evident that the pore pressure data must be verified as being correct and the effect of the rubber membrane must be studied to justify its use.

<u>Pore Pressure Measurements</u>: Perhaps the most important argument which can be presented to verify the pore pressure readings is that pore pressure curves for all 6 tests show similar shapes and magnitudes for a given increment and that they check theoretical curves in many instances. Although there is considerable variation in the shapes of the pore pressure curves for different increments

of pressure, the plots given in Appendix VI illustrate the similarity between tests for a given increment. During test V-1 the loading bar tilted considerably, giving initial pore pressures nearly 150 per cent of the applied load in the latter increments. Except for this test the results are as consistent as can be expected from tests on undisturbed clay.

There may be questions concerning the effect of the presence of the pore pilot on consolidation data. In the 4.25-inch diameter samples the pore pilot has a greater relative importance than it does in the large samples. However, the pore pressure curves and time curves for the two sizes show no more variation than that expected of standard consolidation tests which are run on various sizes of undisturbed samples.

One of the most important factors which affects the validity of pore pressure measurements is time lag. A principal cause of time lag is air in the capillary tube-pore pilot system. The effect may be illustrated as follows. Before the load increment is applied the pore pressures are essentially atmospheric. With a sudden application of load the pore pressure in the sample will increase an amount $u = u_i$. If u remained constant with time it would effect a decrease in the volume of air in the line by ΔV but only after a volume of water equal to ΔV (if the remainder of the system is infinitely rigid) has flowed from the sample into the pore pilot. Because of the low permeability of the clay and the small screen area at the end of the pore pilot, a given increment of time is required before the water pressure in the capillary tube equals u_i. This time increment is known as the time lag. Actually, u is not a constant but decreases with time. As a result, a water pressure as high as u_i will never be recorded. The effect of time lag, then, is to give the impression of smaller initial pore pressures than actually exist in the sample. This effect is more pronounced in a small undisturbed sample, all other things being equal, since consolidation occurs more rapidly.

Air in the line is not the only factor which can cause time lags. If the system expands considerably under pressure the same effect may occur. Furthermore, if the system contains a large volume of water the compressibility of the water becomes an important consideration.

In a preliminary consolidation-pore pressure test on a large sample, a 3-foot length of small diameter Saran tubing was used to connect the pore pilot with the capillary tube measuring device. The effect of time lag may be seen in Figure 40 which shows the compression curve and pore pressure curve for the $\frac{1}{2}$ to 1 kg per sq cm increment for this test. A maximum midplane pore pressure reading of only 66 per cent of the applied load was recorded, <u>but</u> this maximum occurred 4 minutes after the load was applied when the sample as a whole was about 50 per cent consolidated. The shape of this curve may be contrasted to those in Appendix VI. In the reloading cycles of this test, when the primary compression took only 9 minutes, a maximum pore pressure of about 50 per cent of the applied load was measured. A simple experiment which was run by filling a 3-foot length of Saran tubing with water and applying a pressure to one end, showed that the volumetric expansion of the length of tubing was sufficient to cause measurable time lags.

In previous consolidation-pore pressure research at M.I.T. pore pressures have been measured through the bottom porous stone chamber of the consolidometer. While there is considerably more opportunity for time lags in this method, it is believed that the majority of tests run on remolded clay were carefully set up to avoid appreciable lags. There is ample evidence, however, that P. J. Marsal[14], in at least one test on undisturbed clay, had considerable time lags. In his Test U-9, pore pressures of only 10 per cent of the applied load were obtained but these pressures occured 10 or more minutes after the load was applied.

In the tests reported in this investigation initial pore pressures, often taken within 30 seconds after the load was applied, were the maximum or near the maximum which occurred. This is believed by the writer to be sufficient evidence that time lags were negligible.

Effect of the Rubber Membrane: Series F tests were run primarily to study the effect of the rubber membrane on the consolidation results. Since no pore pressure measurements were taken, the comparison between tests with membrane and tests without will be made on standard test results - e vs log p curves, coefficient of consolidation, c_v, initial compression ratio, r_o, and primary compression ratio, r_p. The test results are shown in Figure 41 and summarized in Table IV.

The key plot of this special study is Figure 42 which shows agreement ratios for each pair of tests. Agreement ratio is defined in this case as the value of the soil characteristic from the standard test divided by that from the test with membrane. A number larger than one indicates, then, that the standard test has a higher value than the test with membrane.

Comparative results and observations may be tabulated as follows:

1. <u>e log p curves</u> (see Figure 41)

   a. In general there is good agreement in the shapes of curves for each pair of tests. However, the standard test in all cases has given lower void ratios than the test with membrane. This trend is contrary to that which was expected. D. W. Taylor[12] has demonstrated that the effect of side friction is to reduce the average load causing consolidation in which case the laboratory e - log p curve is displaced to the right since pressures larger than the actual are assumed to act. This minor discrepancy may be due to a consistent error in consolidometer dimensions when the membrane is used.

   b. The rebound from 8 to 7 kg per sq cm is considerably greater for the membrane tests than for the standard tests. This is difficult to see from Figure 41 but may be found in column 13 of Table IV. The fact that the slope of the rebound curve at the reversal point is often considerably flatter than the remainder of the curve has been attributed to side friction. Here is the first indication of the reduction of side friction when the membrane is used.

2. <u>Initial Compression Ratio</u> (see Figure 42)

   Agreement ratios for the initial compression ratio show an average of about 2 which indicates that the standard tests have nearly twice the initial compression that the tests with membrane have.

3. <u>Primary Compression Ratio</u> (see Figure 42)

   The primary compression ratio for the standard test is about 95 per cent of that for the test with membrane.

4. <u>Coefficient of Consolidation</u> (see Figure 42)

   The coefficient of consolidation determined by the square root of time fitting method is about 1.25 times larger in the standard tests than it is in the tests with membrane.

5. <u>Ratio of Coefficient of Consolidation, Square Root of t to Log t</u> (see Figure 42)

   The above ratio tends to be about 10 per cent higher for tests without the membrane. In other words, there is better agreement between the square root of t and log t methods of determining the coefficient of consolidation when a membrane is used.

6. Slope of Straight-Line Portion of Dial vs $\sqrt{t}$ Curve (see column 14, Table IV)

   The standard test shows a somewhat greater slope, especially in the initial laboratory loading cycle of the test.

7. Slope of Straight-Line Portion of Dial vs log t Curve in Secondary Compression (see column 15, Table IV)

   The slope of this line, which is generally straight, is somewhat steeper for the tests with membrane.

Figure 43 has been plotted in order to demonstrate graphically results (2) through (7) above. Since the difference between the two types of tests is in the use of a rubber membrane, the variation in the soil properties is a result of side-wall friction, not only the magnitude but its variation throughout the increment.

R. E. Burrows[34] in "An Experimental Study of Side Friction in the Consolidation Test," found that the average load causing consolidation was about 90 per cent of the applied load. Although no direct measurements of side friction were made in the writer's investigation, the total friction is believed to be considerably less.

The very fact that samples enclosed in a rubber membrane slide easily from their consolidation barrels at the end of a test is evidence of smaller side friction.

While an evaluation of the distribution of side friction throughout the increment is beyond the scope of the present work, it may be stated that the total frictional force on the consolidation container is a function of lateral pressure and coefficient of friction, neither of which is constant for the increment or constant with depth of sample. The total friction for the standard test depends on the intergranular lateral pressure while that for the membrane test is a function of total lateral pressure. As a result the standard test will have an increasing total friction during the increment while the friction in the membrane test may remain essentially constant.

The above comments may be easily correlated with results (2) through (7). On sudden application of load less initial compression (2) will take place in the membrane test since there is a sudden increase in lateral pressure and friction. The standard test will initially have a larger average load causing consolidation, giving a somewhat steeper primary slope (6). However, as the frictional force increases with increasing intergranular pressure in the standard test, the average load causing consolidation will decrease and the compression dial vs

$\sqrt{t}$ curve will round over, giving a shorter time for 90 per cent primary compression and, therefore, a larger coefficient of consolidation (4). Less effective load will cause smaller secondary compression and flatter slopes on the compression dial vs log t curve (7). (In research on remolded clay, D. W. Taylor[12] has found that side friction increases further during secondary compression in the standard test.)

The following general conclusions relative to the use of a thin rubber membrane in consolidation tests may be stated:

1. Use of the membrane as standard procedure is not recommended. However, it serves to reduce the effects of side friction somewhat and is required equipment for the writer's consolidation-pore pressure tests.

2. There is evidence that side friction is one of the contributing factors to the discrepancy between the coefficient of consolidation determined by the square root of t and the log t fitting methods.

3. Actual measurements of side friction during consolidation are needed as a final justification of the use of a membrane.

The apparatus and techniques described in the preceding paragraphs have been found to be entirely satisfactory for obtaining reliable pore pressures during consolidation. A discussion of the results of 6 tests is presented in the following section.

XII

ANALYSIS OF RESULTS

A. GENERAL: Results of the 6 consolidation tests with pore pressure measurements described in the previous section have raised a great many questions relative to consolidation theories, fitting methods for determining c_v, specimen size effects, etc. Because of the limited data available, comments must be limited largely to trends and preliminary conclusions.

The results of this research which are of particular interest may be classified under several general headings:

1. Comparison of shapes of pore pressure curves with those given by consolidation theories, namely the Terzaghi theory and Theory B.
2. Comparison of the square root of t and log t time fitting methods for determining 100 per cent primary compression with the value given by pore pressure curves.
3. General comparison between results of Set III (small samples) and Set V(large specimens) consolidation tests.

These items will be considered in order following a brief explanation of the data and results.

B. <u>PRESENTATION OF DATA</u>: Data from the consolidation tests, Series III-V, are given in Appendix VI in table and curve form. Data for each test consist of one page of general information which includes a table of the loading schedule and final void ratios for each applied pressure, three sheets of compression and pore pressure curves, and one table of additional compression data used to plot the log t curves. The compression curves are presented in the form of dial reading vs $\sqrt{t}$ and the pore pressure curves as dial vs $u_H/\Delta p$ where u_H is the midplane pore pressure. The pore pressure curves constitute the most important data of this research.

C. <u>PRESENTATION OF RESULTS</u>: Results of the consolidation-pore pressure tests are in general given in Table V. Most of the items presented in this table are discussed in the following paragraphs. e vs log p and c_v (determined by the $\sqrt{t}$ method) vs log p curves are given in Figure 44 for all 6 tests.

D. <u>COMPARISON OF PORE PRESSURE CURVES WITH CONSOLIDATION THEORIES</u>: In order to show the characteristic shapes of the pore pressure curves for typical load increments, a general comparison with a modified form* of the Terzaghi theoretical curves will first be shown. Following this general orientation a more detailed comparison between observed slopes of the pore pressure curves and the maximum $u_H/\Delta p$ will be made with the modified Terzaghi theory and Theory B.

Figure 45 shows a comparison between 3 pore pressure curves and Theory T_p. These curves, from Test III-2, are typical for the increments they represent. The light lines are the theoretical U vs $\sqrt{t}$ and U vs U_H curves. (Figures 6 and 3 of Reference 12 may be referred to for these curves.) The dots represent the observed laboratory data fitted to the theoretical curves in the following manner. The point of zero primary compression for each increment, U = 0, has been taken as the dial reading at the intersection of the straight-line portion of the dial vs $\sqrt{t}$ curve with $\sqrt{t}$ equal to zero. The most rational value of 100 per

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* The Terzaghi theory as originally presented assumes that the void ratio change caused by an increment of load is entirely the result of primary compression. Actually, the primary compression, given by r_o times the void ratio change for the increment, is only a part of the total compression, the remainder of which is largely due to secondary compression. A modification of the Terzaghi theory, which is commonly used and generally understood when fitting laboratory compression curves to theoretical curves, assumes that the theoretical curves apply only to the primary part of the total compression. In the following pages of this investigation this modified form of the Terzaghi theory will be called Theory T_p, for the Terzaghi theory applied to primary compression.

cent primary compression, U = 1.0, is given by the dial reading at the intersection of the straight-line portion of the $u_H/\Delta p$ curve with $u_H/\Delta p$ equal to zero. (There is no straight line given in the 2-4 increment. Rather than select an arbitrary point of tangency, 100 per cent primary compression has been computed from the $\sqrt{t}$ fitting method in this one instance.) The observed compression curve has been fitted to the theoretical U vs $\sqrt{T}$ curve by first determining the dial reading at 50 per cent primary compression. This dial reading and $\sqrt{t}$ associated with it, correspond to U = 50 per cent and $\sqrt{T}$ = 0.444. Thus, the scale ratio to be used in the fitting is determined.

Agreement between the actual and theoretical pore pressure curves of Figure 45 is in general somewhat better than in other tests. In general, the pore pressure curves for the laboratory recompression increments 1/2-1 and 1-2 agree fairly closely with Theory T_p.

The shapes of the pore pressure curves for the 2-4 increment, which is the "break-over" into virgin compression, do not resemble any theoretical curve. This may be a result of an extreme variation from the assumed straight-line relationship between void ratio and pressure for this increment. Initial pore pressures in the 4-8 increment average about 80 per cent of the applied load indicating, perhaps, some form of intergranular bond. The slope of these curves after about 60 per cent primary consolidation is not far from the theoretical, however.

In the laboratory reloading increments, 4-5 and 5-6 kg per sq cm (see data in Appendix VI) the slopes of the pore pressure curves are considerably steeper than the theoretical. In addition, maximum $u_H/\Delta p$ values average only 85 per cent.

In order to make specific comparisons between the actual pore pressure curves and those given by the Theory T_p and Theory B, ratios of initial pore pressures and ratios of slopes of the pore pressure curves will be set up. Figure 46 has been drawn to illustrate the procedure. The following notation for various dial readings will be used:

d_o = initial dial reading for the increment

d_s = corrected zero point based on the back projection of the straight-line portion of the dial vs $\sqrt{t}$ curve

d_1 = 100 per cent primary compression determined from the pore pressure curve (described in a previous paragraph)

$d_{100}(\sqrt{t})$ = 100 per cent primary compression determined by the $\sqrt{t}$ fitting method

$d_{100}(\log t)$ = same by the log t fitting method

d_f = final dial reading at the end of the increment

d_{1p} = d_1 of the previous increment (used in Theory B)

d_t = dial reading at back projection of pore pressure curve to $u_H/\Delta p = 1.0$.

The observed compression and pore pressure curves for load increment 5-6 of consolidation test V-2 are shown by curves through the plotted points in Figure 46. Using d_s and d_1 as the beginning and end of primary compression the theoretical pore pressure curve according to Theory T_p may be drawn. Characteristics of this curve are:

(1) $\frac{u_H}{\Delta p} = 1.00$ initially

(2) $\frac{QL}{AL} = \frac{2}{\pi} = 0.636$

The theoretical pore pressure curve according to D. W. Taylor's Theory B has been determined in the past by a procedure called the slope-ratio fitting method. This procedure is explained in detail in Reference 12 and may be described as follows:

A slope ratio is defined as

$$S.R. = \frac{d_t - d_1}{d_{1p} - d_1} = \frac{NL}{ML}$$

From Figure 46

$$\text{S.R.} = \frac{.13781 - .13516}{.13957 - .13516} = 0.601$$

Using a S.R. equal to 0.601 the values of two important terms, J and C, used in Theory B may be determined from Figure 49 of Reference 12. If S.R. < 0.636 then J and C are assumed equal to zero.*

The primary compression ratio in Theory B is defined as

$$r_p' = \frac{d_s - d_1}{(1+c)(d_{1p} - d_1)} = \frac{.13840 - .13516}{.13957 - .13516} = 0.735$$

and the effective load causing consolidation is given by $r_p\,\Delta p$. The width of the theoretical plot becomes FB where

$$\frac{FB}{LB} = 0.735$$

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*In all load increments in this investigation but two, the slope ratio was less than 0.636. Refer to Reference 12: The ratio of time factors, $\underline{J}$, is given by formula (90). If $J = 0$, then $\bar{\eta} = 0$ where $\bar{\eta}$ is an assumed constant of proportionality between viscous resistance to compression and speed of compression, formula 61. The resulting effect of J equal to zero in Theory B, then, is to give equations identical to those given by the Terzaghi theory. Only one difference remains: Theory B still recognizes an intergranular bond which depends on the amount of previous secondary compression. As such, the effective load increment causing consolidation is smaller than the applied increment Δp by r_p' which is defined as AL/ML in Figure 46 of this thesis. The width of the theoretical plot considered in Theory B is then $FB = r_p'(LB)$. (Note the similar triangles JFB and MLB.) If there is no previous secondary compression, then, AM equals zero, r_p' equals 1, FB = LB and the Theory B pore pressure curve coincides with the Terzaghi Theory applied to primary compression.

A theoretical pore pressure curve may be drawn as shown in Figure 46 using Figure 47 of Reference 12. For J equal to zero it is identical in shape to the curve given by Theory T_p.

Generally, the actual pore pressure curves in the writer's tests fall between the two theoretical curves. A measure of the agreement of the theoretical shapes with the observed pore pressure curve will be shown by a comparison of the maximum $u_H/\Delta p$ and the slope of the pore pressure curves. Refer to Figure 46 for the following sample computation:

$$\max. \frac{u_H}{\Delta p} \text{ (observed)} = 0.89$$

$$\max. \frac{u_H}{\Delta p} \text{ (Theory } T_p\text{)} = 1.00 \text{ (constant)}$$

$$\max. \frac{u_H}{\Delta p} \text{ (Theory B)} = 0.735 \text{ (numerically equal to } r_p'\text{)}$$

To compare the slopes of pore pressure curves, the following ratios will be used:

$$\text{Theory } T_p\text{:} \quad \frac{NL}{AL} \div \frac{QL}{AL} = \frac{NL}{0.636AL}$$

$$\text{Theory B:} \quad \frac{NL}{AL} \div \frac{PL}{AL} = \frac{NL}{PL}$$

From Figure 46

$$\text{Theory } T_p\text{:} \quad \frac{.13781 - .13516}{(0.636)(.13840 - .13516)} = 1.29$$

$$\text{Theory B:} \quad \frac{.13781 - .13516}{.13799 - .13516} = 0.94$$

Values of $u_H/\Delta p$ (observed) and NL/AL have been determined for the 6 consolidation tests and are tabulated in columns 1 and 3 of Table V. Since the first pore pressure determinations may be erratic, the absolute maximum $u_H/\Delta p$ has not been taken. Instead, the maximum determined from a smooth curve shown, as an example, by the dashed curve in Figure 45c, has been used. Because of the absence of a straight-line portion of the $u_H/\Delta p$ curve in the 2-4 increment (see Figure 45b), no values of NL/AL have been recorded for this increment.

Columns 4 and 5 of Table V give values of the Theory $T_p\left(\frac{NL}{0.636AL}\right)$ and Theory $B\left(\frac{NL}{PL}\right)$ which are measures of agreement between slopes of pore pressure curves. Values greater than 1 indicate that the slope of the actual pressure curve is steeper than that given by theory. As noted earlier, a determination of the theoretical curve according to Theory B requires the dial reading of the previous 100 per cent primary compression point, d_{1p}. Good results of d_{1p} for the load 1/2-1 are not available. In addition d_{1p} does not have much significance in the load increment 4-5 since the previous increment was rebound. These increments, 1/2-1 and 4-5, are therefore omitted from column 4. The theoretical pore pressure curve for the 4-8 increment was determined using a d_{1p} found by drawing a tangent to the $u_H/\Delta p$ curve of the previous increment at $u_H/\Delta p$ = 0.3 for Set III tests and 0.4 for Set V tests. Maximum values of $u_H \Delta p$ given by Theory B are shown by column 2.

An inspection of column 1 of Table V shows that maximum pore pressures in the recompression range of the consolidation test are very nearly equal to the applied load increment. Values in excess of unity are believed to be a result of an initial small eccentricity in loading. Intergranular bonds apparently exist in the 4-8 and reloading increments and if they do they initially carry an average of 15 to 20 per cent of the applied load. Quantitative agreement with Theory B, column 2, is not too good but it appears that the bonds which the theory recognizes are nevertheless present.

Reasonably good agreement in slopes of pore pressure curves is obtained with Theory T_p in all increments except the reloading 4-5 and 5-6 increments. Excellent agreement, column 5, is obtained with Theory B in the large diameter samples, the average NL/PL for these tests being 0.96.

General Discussion: It has been shown from these consolidation tests on undisturbed samples that the factor J considered in Theory B is generally equal to zero. When this is the case the only difference between the modified form of the Terzaghi theory and Theory B is the inclusion of bond. Physically, this means that on the application of a load increment the Terzaghi theory assumes that the initial pore water pressure is constant and equal to Δp. Theory B assumes it is constant and equal to $r'_p \Delta p$. The coefficient of consolidation is the same for both cases and the pore pressure dissipation follows the same laws.

Interpretation of laboratory consolidation tests by means of Theory B require, as originally set up, pore pressure measurements during consolidation. However, if J is equal to zero, the only requirement is that d_{1p} may be known for a given increment in which case the bond may be evaluated. The value of d_{1p} may be determined by either the $\sqrt{t}$ or log t time fitting methods. Standard consolidation test data are sufficient, then, to allow an estimate of the bond.

The inclusion of bond does not affect the ultimate settlement and time-settlement predictions based on a_v and c_v determined from laboratory consolidation tests.* It does, however, affect a study of pore pressures in the consolidating clay stratum which is seldom a practical consideration.

Taylor found that the plastic structural resistance to compression had the greatest relative effect for small

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* According to Theory B this is strictly true. It should be noted, however, that according to D. W. Taylor's bond hypothesis associated with Theory B, there are considerable differences especially if small loads are applied. This hypothesis has been discussed in Part I, VII-F.

load increment ratios. The inconsistent variation in c_v with load increment ratio when consolidation tests on remolded clay were interpreted by the Terzaghi theory, was accounted for by Theory B. Slope ratios were found to be considerably greater than 0.636 giving J values greater than zero for small load increment ratios. There is a slight trend in this direction for the reloading increments 4-5 and 5-6 in the consolidation tests described in this thesis.

Before final conclusions can be made regarding Theory B, consolidation-pore pressure tests on undisturbed samples using small load increments must be run. If J is found to be equal to zero, then the only adjustment which must be made in consolidation thinking is to include bond.

E. COMPARISON OF 100 PER CENT PRIMARY COMPRESSION BY THE $\sqrt{t}$ AND LOG t METHODS: The value of 100 per cent primary compression given by the pore pressure curves offers an excellent opportunity to check the validity of the $\sqrt{t}$ and log t methods for determining this point. Series F consolidation tests have shown that for undisturbed Boston blue clay the coefficient of consolidation determined by the $\sqrt{t}$ method is generally about 1/25 to 1.5 times that given by the log t method. This may be seen in column 8 of Table IV and in Figure 41. Rather than compare c_v by the two methods for the pore pressure tests, a comparison on the basis of primary compression ratios will be made.

The primary compression ratio, r_p, has been computed by three methods. In every case the start of primary compression is taken at d_s as previously defined. Using the notation of Figure 46, r_p is given by

$$r_p\ (\sqrt{t}) = \frac{d_s - d_{100}(\sqrt{t})}{d_o - d_f} = 0.667$$

$$r_p(\log\ t) = \frac{d_s - d_{100}(\log\ t)}{d_o - d_f} = 0.643$$

$$r_p(\frac{u_H}{\Delta p}) = \frac{d_s - d_1}{d_o - d_f} = 0.659$$

A summary of these values for the pore pressure tests is given in columns 6, 7 and 8 of Table V. Columns 9 and 10 are agreement ratios defined as

$$R(\sqrt{t}) = \frac{r_p(\sqrt{t})}{r_p(\frac{u_H}{\Delta p})} = 1.012$$

$$R(\log\ t) = \frac{r_p(\log\ t)}{r_p(\frac{u_H}{\Delta p})} = 0.975$$

Agreement ratios and $r_p(\frac{u_H}{\Delta p})$ are plotted in Figure 47 against pressure. The following observations are evident from the R plots:

1. In general $R(\sqrt{t}) < 1$ and $R(\log t) > 1$ which was expected. However, no trend is evident in the reloading increments.

2. On the average the true primary compression ratio lies midway between the ratios given by the two fitting methods. This is generally so except for the 4-8 increment in the large samples where r_p by the two fitting methods is consistently about 95 per cent of that given from pore pressure curves.

These observations indicate that, in general, the square root of time fitting method gives values of c_v which are too high while the log t values are too small. The true magnitude of the coefficient of consolidation is probably not far from an average of the values determined by the two fitting methods.

F. GENERAL COMPARISON BETWEEN SET III AND SET V TESTS: The reader is referred to Figures 44 and 47 for the following observations:

1. There are no extreme differences between the small and large samples in the e vs log p curves. There is a trend, however, for the large samples to show somewhat greater compression during the 2-4 increment and somewhat less for the 4-8 increment.

It was expected that the slopes of the small sample compression curves would be steeper than the large because of disturbance during sample preparation when planing the top and bottom surfaces, but this was not the case.

2. During rebound and reloading the large samples gave 1/2 to 2/3 the void ratio change that the small samples gave. These changes are not tabulated but may be found in the data of Appendix VI.

3. For the 2-4 load increment the large samples gave about 1/2 the coefficient of consolidation that the small samples gave. This may be a result of more secondary compression during primary compression in the large samples.

   The significance of this observation is important. If the trend in c_v with sample size is extrapolated to a thick stratum of clay, it is possible that compressions in nature will occur considerably slower than predicted <u>if</u> the net load increment is large enough to take the clay stratum into a virgin compression curve.

4. There is a trend toward higher coefficients of consolidation for the large samples in the initial

recompression and laboratory reloading increments. The significance of this observation may be even more important than that of (3). A thick precompressed clay stratum may consolidate considerably faster than estimated from results of laboratory tests on small samples. This has already been shown in Part I to be the case at the Hayden Library.

PART II

SUMMARY AND GENERAL DISCUSSION

The consolidation-pore pressure tests described in Part II of this investigation have thrown new light on the validity of consolidation theories and the interpretation of these theories. These studies are meant to be preliminary ones only, presented at this time in such form that they will raise questions rather than answer them.

The apparatus and techniques described herein are believed to be adequate for obtaining reliable pore pressure measurements during consolidation.

Preliminary results of the 6 consolidation-pore pressure tests show generally good agreement with the Terzaghi theory with the exception that certain load increments show initial pore water pressures which are considerably smaller than the applied load. This may be due in part to a form of bond similar to that recognized in Theory B.

A comparison of the coefficients of consolidation given by the square root of time and log time fitting methods with the coefficient of consolidation which could be determined from pore pressure curves has given the following conclusion. Generally the square root method gives values which are too large while the log method values are too small. The true value of the coefficient of consolidation lies about midway between the two values.

The reader is referred to XII-F for a summary and discussion of the important differences between results of consolidation tests on large and on small samples.

Several points relative to pore pressure measurements during consolidation will be mentioned at this time so their effects will not be overlooked in future research.

One of these important considerations is side friction. Even though the rubber membrane coated with colloidal graphite is believed to reduce side friction considerably, no proof can be given at this time that there is not a sufficient initial increase in sidewall friction to carry part of the applied load. With side friction, the distribution of stress across the midplane of the sample is anything but uniform. The horizontal gradients which are set up will cause varying amounts of lateral flow, the effects of which are difficult to analyze.

In addition to side friction the possible effects of shearing strains on pore water pressures in <u>one</u>-dimensional consolidation must be considered. R. H. Clough(3) has recently investigated the distribution of pore water pressures in undrained triaxial tests on undisturbed clay samples. He found that considerably higher pore water pressures were associated with the zones of largest shearing strains in the sample. In one-dimensional consolidation the rate of shearing strain is initially largest near the surfaces of the sample and finally in the center of the sample. In other words, it is indeed possible that the distribution of pore water pressure throughout the height of the sample is considerably different from the theoretical pattern as a result of nonuniform rates of shearing strains.

Finally, the theoretical pore pressure dissipation curves are based on a constant permeability throughout the height of the sample. However, the permeability at the surfaces of the sample is smaller since consolidation there is more advanced. In addition, the surfaces may be at considerably lower void ratios because of the remolded zone. The net effect of the permeability variation is to increase gradients at the surface. The pore pressure vs depth curve is therefore flatter than that given by consolidation theories - or - at any given average consolidation ratio, the midplane consolidation ratio is smaller than the theoretical.

These points are just a few of the many which must be carefully considered before final conclusions relative to the validity of Theory B can be made.

LIST OF APPENDICES

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APPENDIX I

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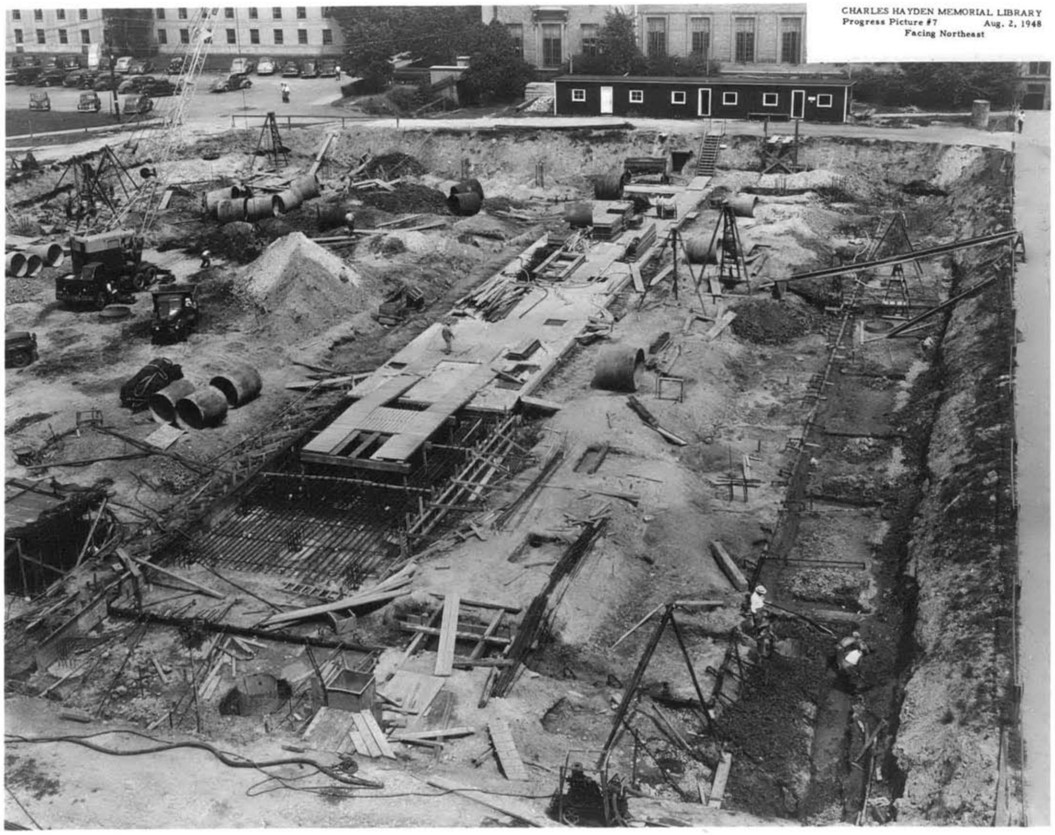
II PROGRESS PHOTOGRAPHS OF CONSTRUCTION

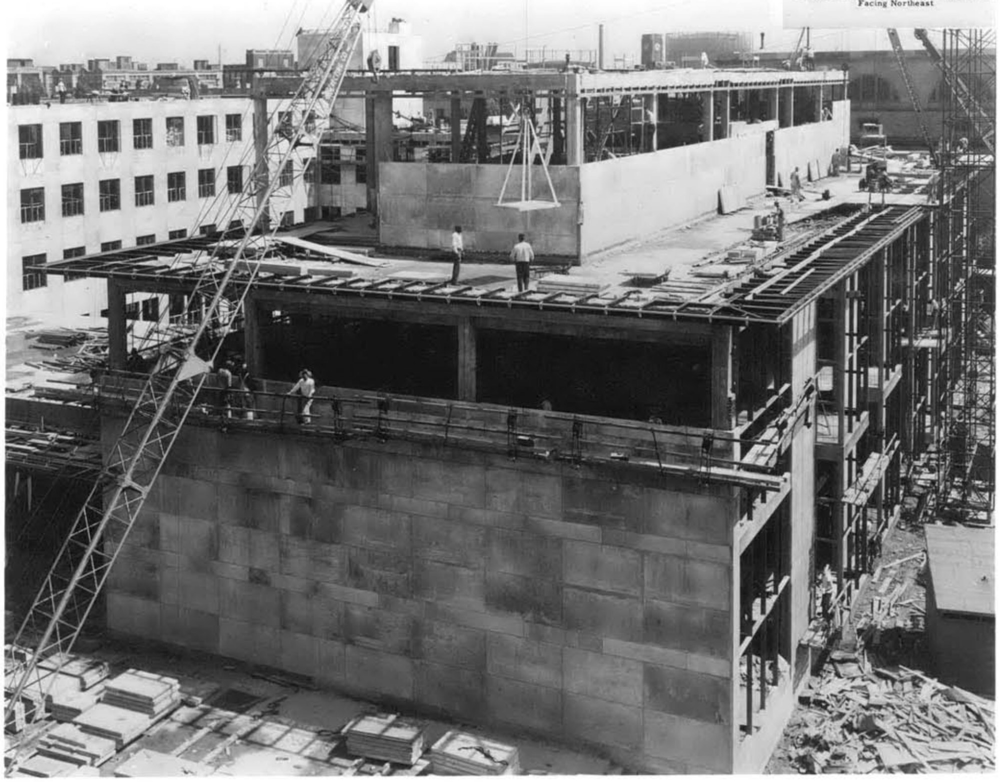


CHARLES HAYDEN MEMORIAL LIBRARY
Progress Picture #7 Aug. 2, 1948
Facing Northeast

CHARLES HAYDEN MEMORIAL LIBRARY
Progress Picture #13 Nov. 1, 1948
Facing Northeast

CHARLES HAYDEN MEMORIAL LIBRARY
Progress Picture #19 Feb. 3, 1949
Facing Northeast.

CHARLES HAYDEN MEMORIAL LIBRARY
Progress Picture #25 May 4, 1949
Facing Northeast



III CAISSON SCHEDULE

APPENDIX III

CAISSON SCHEDULE[1]

Caisson No.	Diameter of Bell Ft. In.	Elevation Bottom of Bell Ft.	Dist. from Bottom of Bell to Top of Clay Ft.	Design Load[2] Kips	Present Load[3] Dead Kips	Present Load[3] Live Kips	Present Load[3] Total Kips
1	9- 0	- 9.5	3.1	353	329	6	335
2	13- 4	-11.0	1.5	503	423	6	429
3	13- 2	-11.4	1.1	488	383	6	389
4	11- 0	-12.3	1.2	427	345	9	354
5	11- 6	-11.7	1.5	427	345	11	356
6	10- 4	- 9.5	2.5	427	345	9	354
7	11- 0	-10.2	1.1	427	345	6	351
8	11- 2	- 9.1	1.4	427	339	6	345
9	11-10	- 9.4	1.0	427	343	6	349
10	9- 2	- 9.2	1.9	371	339	6	345
11	10- 0	- 9.5	2.6	366	344	6	350
12	10- 0	- 8.6	1.4	329	315	6	321
13	11- 4	-11.2	1.0	389	316	7	323
14	13- 0	-11.8	0.9	542	375	12	387
15	12- 8	-12.4	0.4	508	323	15	338
16	8- 0	- 9.6	3.8+	397	263	17	380
17	8- 0	- 9.3	3.5+	397	263	17	380
18	8- 0	- 8.0	3.4	397	259	15	374
19	8- 0	- 7.6	4.0+	397	263	13	376
20	8- 0	- 8.0	3.4	397	235	13	348
21	8- 4	- 7.8	2.9	397	237	13	350
22	10- 0	- 9.1	1.2	367	235	13	348
23	9- 0	- 9.4	1.8	364	248	16	364
24	11- 0	- 9.4	1.6	421	383	3	386
25	9- 5	- 9.3	3.4	422	373	7	380
26	10- 0	- 7.7	1.2	315	264	7	271
27	9- 6	-11.0	1.2	200	178	6	184
28	10- 2	-12.0	0.6	320	262	6	268
29	11-10	-12.6	1.5	477	163	8	171
30	9- 0	- 9.4	4.0	423	288	10	298
31	8- 9	-11.2	3.0+	413	288	10	298
32	9- 6	- 8.6	4.0+	444	317	10	327
33	9-10	- 7.7	3.5+	466	350	11	361
34	9- 6	- 7.7	4.2+	433	310	11	321

(1) See Figure 8 for caisson locations.

(2) Moran, Proctor, Freeman and Mueser values (see Hayden Library Plan 1203-F-5, revision 7).

(3) As of April 1, 1951

CAISSON SCHEDULE (Contd.)

Caisson No.	Diameter of Bell Ft. In.	Elevation Bottom of Bell Ft.	Dist. from Bottom of Bell to Top of Clay Ft.	Design Load (2) Kips	Present Load (3) Dead Kips	Live Kips	Total Kips
35	10- 5	- 8.5	2.6	433	310	11	321
36	10- 0	- 8.8	3.1	451	320	11	331
37	11- 2	- 8.8	2.1	458	560	8	568
38	15- 6	- 9.8	1.4	749	537	13	550
39	11- 0	- 8.4	2.0	460	319	5	324
40	9-10	-11.3	1.4	362	170	4	174
41	8- 8	-11.7	1.9	320	212	1	213
42	10-10	-11.1	1.9	453	220	1	221
43	8- 8	- 7.8	4.0+	369	183	6	189
44	7-10	- 7.8	4.0+	306	167	6	173
45	7-10	- 7.8	4.0+	306	167	6	173
46	8- 4	-10.0	2.5	306	167	6	173
47	11- 2	-10.0	1.4	457	288	9	297
48	10- 6	- 9.0	2.8	457	284	9	293
49	12- 0	-10.8	1.6	554	348	10	358
50	11- 0	- 9.8	2.3	467	341	6	347
51	9- 0	-12.3	1.1	322	207	4	211
52	12- 0	-12.5	0.6	454	208	4	212
53	8-10	- 9.4	4.0+	370	179	10	189
54	7-10	- 8.6	4.0+	305	159	10	169
55	8-10	-10.1	2.1	305	159	10	169
56	8- 0	- 8.7	3.5+	305	159	10	169
57	10- 6	- 9.8	2.8	457	330	8	338
58	11- 8	-11.6	1.2	460	278	11	289
59	10- 2	-10.6	1.8	419	328	6	334
60	9- 4	-12.4	1.8	357	170	4	174
61	9- 2	-12.0	1.5	324	217	4	221
62	12- 4	-12.9	0.9	454	292	7	299
63	10- 0	-11.7	2.1	370	190	10	200
64	8-10	-10.8	2.3	305	154	10	164
65	9- 8	-11.1	1.7	305	159	10	169
66	8-10	-10.3	1.3	305	159	10	169
67	10- 6	-10.4	2.3	457	385	8	393
68	11- 0	-10.8	1.8	466	280	11	291
69	10- 6	-10.8	2.0	419	320	6	326
70	11- 4	-12.5	2.0	444	398	4	402
71	11-10	-13.3	1.0 (Ave.)	358	256	6	262
72	11- 0	-12.1	1.2	358	221	10	231
73	10- 4	-13.0	1.1	320	198	11	209
74	10- 4	-12.8	1.7	322	197	11	208
75	11- 0	-13.4	0.8	323	177	11	188
76	10- 8	-12.7	0.7	323	177	11	188
77	8- 2	-12.3	0.5	316	167	11	178

CAISSON SCHEDULE (Contd.)

Caisson No.	Diameter of Bell	Elevation Bottom of Bell	Dist. from Bottom of Bell to Top of Clay	Design Load(2)	Present Load(3) Dead	Live	Total
	Ft. In.	Ft.	Ft.	Kips	Kips	Kips	Kips
78	9- 8	-11.3	1.7	315	167	11	178
79	10- 8	-12.4	0.6	317	167	11	178
80	9-10	-11.2	1.9	317	167	11	178
81	9- 0	- 9.9	2.2	319	171	13	184
82	9- 0	-10.6	2.2	319	201	9	210
83	8- 6	- 8.6	3.5+	346	204	10	214
84	8- 6	-10.1	2.3	346	265	10	275
85	10- 2	-10.8	2.0	422	417	7	424
86	13- 9	-13.2	0.5	445	424	5	429
87	11-10	-12.8	0.5	361	228	6	234
88	9- 3	-12.0	2.5	341	219	6	225
89	10- 6	-13.4	0.8	323	219	11	230
90	10-10	-13.1	0.5	324	219	13	232
91	11- 6	-13.8	0.1	323	209	13	222
92	10-10	-12.2	0.8	318	219	13	232
93	10- 8	-12.8	0.6	314	219	15	234
94	9- 8	-11.8	1.6	314	219	15	234
95	10- 4	-12.2	1.0	316	219	15	234
96	9- 2	-11.0	2.0	318	219	15	234
97	8- 2	- 9.0	3.5	320	219	13	232
98	8- 4	- 9.1	3.5+	334	224	6	230
99	8- 6	- 9.0	4.0+	346	238	10	248
100	10- 9	-10.4	1.7	421	398	6	404
101	13'0"x13'3"	-12.4	Rests on clay	443	379	6	385
102	10- 2	-11.7	2.2	361	249	8	257
103	11-10	-12.7	Rests on clay	362	213	6	219
104	8- 0	-12.3	3.5+	323	213	7	220
105	11- 4	-12.8	Rests on clay	323	213	11	224
106	10- 4	-13.3	1.0	324	213	11	224
107	11- 8	-13.4	0.2	322	213	11	224
108	11- 0	-12.8	0.7	316	213	13	226
109	11- 6	-13.4	0.4	314	213	13	226
110	11- 8	-12.0	1.5	316	213	13	226
111	9-10	-11.5	1.3	316	213	13	226
112	8- 0	-10.0	3.0	318	213	15	228
113	8- 0	-10.6	3.0	318	218	9	227
114	9- 4	-10.4	2.2	346	218	11	229
115	8- 4	- 9.5	4.0+	346	256	11	267
116	9- 4	- 9.3	3.7	422	347	7	354
117	9- 4	- 8.3	3.5+	382	245	4	249
118	12-10	-13.0	Rests on clay	389	192	14	206
119	11- 8	-13.0	1.2	389	192	8	200
120	12- 4	-14.0	0.5	389	192	6	198

CAISSON SCHEDULE (Contd.)

Caisson No.	Diameter of Bell Ft. In.	Elevation Bottom of Bell Ft.	Dist. from Bottom of Bell to Top of Clay Ft.	Design Load (2) Kips	Present Load (3) Dead Kips	Live Kips	Total Kips
121	12-10	-14.1	Rests on clay	389	192	8	200
122	12- 8	-13.4	Rests on clay	389	192	6	198
123	11- 6	-12.7	1.1	389	192	14	206
124	11- 3	-12.1	1.5	389	192	14	206
125	10- 4	-11.5	1.9	385	244	5	249
126	8-10	- 7.8	4.0+	381	180	2	182
127	9- 0	- 4.4	4.7+	381	274	1	275
128	9- 0	- 6.7	6.4	370	252	5	257
129	9- 1	- 6.2	4.8+	370	255	3	258
130	9- 2	- 6.5	4.5+	370	246	3	249
131	11- 6	-11.2	1.1	377	254	5	259
132	9- 2	- 7.0	3.8+	377	246	3	249
133	10- 0	- 9.7	2.8	377	246	3	249
134	12'0"x11'8"	-14.1	0.5	391	267	5	272
135	13- 2	-13.3	Rests on clay	390	261	3	264
136	12- 0	-13.6	0.8	389	261	3	264
137	12- 2	-14.0	0.8	373	255	5	260
138	12- 2	-13.8	0.9	373	249	3	252
139	11- 8	-13.6	1.0	373	251	3	254
140	12- 0	-13.5	0.8	390	267	5	272
141	12-10	-14.4	0.8	390	263	3	266
142	10-10	-13.4	1.7	391	263	3	266
143	11- 0	-13.1	1.2	379	252	5	257
144	10- 8	-13.0	1.9	378	248	3	251
145	10- 0	-12.6	2.3	379	248	4	252
146	11- 4	-12.8	1.0	372	252	4	256
147	10- 8	-13.5	1.5	372	248	4	252
148	10- 0	-12.2	2.6	372	248	2	250
149	9- 2	-12.5	3.7+	383	276	3	279
150	11- 4	-13.2	1.1	383	180	5	185
151	11- 0	-13.4	0.7	356	175	4	179
152	9- 0	- 9.1	2.1	365	80	2	82

IV SUMMARY OF SETTLEMENT OBSERVATIONS

APPENDIX IV

SUMMARY OF SETTLEMENT OBSERVATIONS(1)(2)

Days Since Start of Construction	Initial Elev.(3) →	1		2		3		4		5		6		7 □(4)		8 □		9 □		10		11		12 □		13 □		14 □		15		16		← Initial Elev.(3)	Days Since Start of Construction
		7.491		7.475		7.460		7.462				7.470		7.488		7.499		7.502				7.470		7.491		7.493		7.511		7.472		7.477			
		Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)		
241	Dec. 4, '48	7.487	.05	7.468	.08	7.453	.08	7.456	.07	No		7.466	.05									7.461	.11									7.476	.01	Dec. 4, '48	241
255	Dec. 18, '48	7.492	-.01	7.474	.01	7.459	.01	7.462	.00	hole		7.479	-.11	7.487	.01	7.495	.05	7.502	.00			7.466	.05	7.490	.01	7.489	.05	7.507	.05	7.471	.01	7.475	.02	Dec. 18, '48	255
283	Jan. 15, '49	7.486	.06					7.453	.11	drilled																				7.461	.13			Jan. 15, '49	283
305	Feb. 6, '49	7.475	.19	7.456	.23	7.443	.20	7.453	.11	until		7.453	.20	7.476	.14	7.484	.18	7.484	.22							7.478	.18	7.494	.20	7.455	.20			Feb. 6, '49	305
318	Feb. 19, '49	7.476	.18	7.466	.11	7.451	.11	7.459	.04	March		7.465	.06	7.479	.11	7.491	.10	7.490	.14			7.453	.20			7.505	-.14	7.503	.10	7.468	.05	7.460	.20	Feb. 19, '49	318
345	Mar. 18, '49			7.467	.10	7.458	.02	7.458	.05	1950								7.488	.17			7.450	.24	7.481	.12	7.487	.07	7.492	.23			7.453	.29	Mar. 18, '49	345
369	Apr. 11, '49	7.484	.08	7.468	.08	7.450	.12	7.453	.11					7.478	.12	7.489	.12	7.493	.11	Inac-cessible		7.453	.20			7.486	.08	7.497	.17	7.455	.20	7.453	.29	Apr. 11, '49	369
439	June 20, '49			7.451	.29	7.436	.29	7.442	.24			7.444	.31	7.465	.28	7.479	.24	7.477	.30			7.435	.42	7.475	.19	7.478	.18	7.485	.31	7.441	.37	7.445	.32	June 20, '49	439
579	Nov. 7, '49	7.458	.40	7.442	.40	7.432	.34	7.435	.32			7.428	.50	7.455	.40	7.467	.38	7.462	.48			7.425	.54	7.467	.29	7.469	.29	7.472	.47	7.429	.49	7.434	.43	Nov. 7, '49	579
660	Jan. 27, '49	7.454	.44	7.435	.48	7.427	.40	7.429	.40			7.422	.58	7.447	.49	7.459	.48	7.456	.55			7.421	.59	7.462	.35	7.465	.34	7.469	.50	7.430	.50	7.431	.55	Jan. 27, '49	660
709	Mar. 17, '50	7.450	.49	7.433	.50	7.428	.38	7.432	.36	7.441		7.419	.61	7.445	.52	7.458	.49	7.456	.55			7.419	.61	7.460	.37	7.464	.35	7.467	.53	7.427	.54	7.427	.60	Mar. 17, '50	709
793	June 9, '50	7.447	.53	7.430	.54	7.426	.41	7.429	.40	7.438		7.417	.64	7.445	.52	7.457	.50	7.454	.58			7.418	.62	7.459	.38	7.465	.34	7.467	.53	7.426	.55	7.426	.61	June 9, '50	793
999	Jan. 1, '51	7.447	.53	7.432	.52	7.424	.43	7.427	.42	7.436		7.415	.66	7.443	.54	7.455	.53	7.452	.60			7.416	.65	7.458	.40	7.463	.36	7.466	.54	7.423	.59	7.423	.65	Jan. 1, '51	999

Days Since Start of Construction	Initial Elev.(3) →	17 □		18 □		19 □		20		21		22		23		24		25		26		27		28		29		30		31		32		← Initial Elev.(3)	Days Since Start of Construction
		7.492		7.473		7.486		7.470		7.530				7.496		7.520		7.472		7.460		7.462		7.467											
		Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)	Elev.	ρ(in.)		
241	Dec. 4, '48									7.523	.08			7.493	.04	7.517	.04	7.471	.01	7.458	.02													Dec. 4, '48	241
255	Dec. 18, '48	7.489	.04	7.475	-.02			7.469	.01	7.525	.06			7.495	.01	7.519	.01	7.471	.01	7.458	.02	7.465	-.04											Dec. 18, '48	255
283	Jan. 15, '49					7.473	.16	7.453	.20	7.511	.23			7.480	.19	7.512	.10	7.463	.11			7.438	.29											Jan. 15, '49	283
305	Feb. 6, '49	7.481	.13	7.464	.11	7.472	.17	7.450	.24	7.515	.18			7.470	.31	7.499	.25	7.451	.25			7.441	.25											Feb. 6, '49	305
318	Feb. 19, '49			7.472	.01	7.481	.06	7.462	.10	7.504	.31			7.473	.28	7.505	.18	7.458	.17			7.454	.10											Feb. 19, '49	318
345	Mar. 18, '49			7.466	.08			7.446	.29	7.492	.46			7.469	.32	7.500	.24	7.453	.23	7.430	.36	7.444	.22											Mar. 18, '49	345
369	Apr. 11, '49	7.490	.02	7.474	-.01			7.449	.25	7.488	.50	Inac-cessible		7.465	.37	7.497	.28	7.459	.16	7.435	.30					7.438		7.449		10.068		10.077		Apr. 11, '49	369
439	June 20, '49	7.478	.17	7.467	.07	7.465	.25	7.432	.46	7.477	.64			7.455	.49	7.492	.34	7.448	.29	7.421	.47	7.418	.53	7.428	.47	7.426		7.433		10.068		10.077		June 20, '49	439
579	Nov. 7, '49	7.470	.26					7.418	.62	7.468	.74			7.447	.69	7.486	.41	7.442	.36	7.408	.62	7.405	.68	7.411	.67	7.416		7.426		10.054		10.066		Nov. 7, '49	579
660	Jan. 27, '49	7.468	.29	7.458	.18	7.453	.40	7.415	.66					7.444	.62	7.488	.38	7.443	.35	7.408	.62	7.403	.70	7.407	.72	7.407		7.422		10.052		10.063		Jan. 27, '49	660
709	Mar. 17, '50	7.467	.30	7.456	.20	7.449	.44	7.414	.67	7.455	.90			7.438	.70	7.475	.54	7.432	.48	7.402	.70	7.399	.76	7.406	.73									Mar. 17, '50	709
793	June 9, '50	7.466	.31	7.457	.19	7.449	.44	7.412	.70	7.453	.93			7.436	.72	7.474	.55	7.432	.48	7.398	.74	7.398	.77	7.399	.82	7.402		7.410		10.044		10.055		June 9, '50	793
999	Jan. 1, '51	7.464	.34	7.455	.22	7.448	.46	7.410	.72	7.452	.94			7.435	.73	7.474	.55	7.431	.49	7.400	.72	7.396	.79	7.401	.79									Jan. 1, '51	999

(1) All elevations in this table are based on the USCGS mean sea level datum. (Tech Bench = +9.551 ft.) Add 10.84 ft. to obtain elevations on the Cambridge datum.

(2) Location of observation points given in Figure 9.

(3) Based on the extrapolation of settlement curves to Dec. 1, 1948.

(4) Square represents column cross section and dot refers to the position of the observation point on the column, see Figure 9.

V PIEZOMETERS FOR PORE PRESSURE MEASUREMENTS

APPENDIX V

PIEZOMETER FOR PORE PRESSURE MEASUREMENTS

NOTE:

The descriptive material which follows is based largely on the original paper by A. Casagrande which is published in the Journal of the Boston Society of Civil Engineers, April 1949, page 214. As such the sections in quotes and the figures referred to herein are those in the above reference. Anyone desiring to install this type of piezometer should therefore use A. Casagrande's paper and this Appendix side by side.

I. DESIGN AND CONSTRUCTION OF THE POROUS POINT ASSEMBLY:

"The porous point consists of a 2-foot length of fine grade Norton Porous Tube, 1.5" O.D. by 1.0" I.D." A predetermined length of 1/2" O.D. Saran tubing is connected to one end by means of a soft rubber bushing. The other end is plugged with a #5 1/2 rubber stopper. A cross section of this assembly is shown in Figure 16.

To install the Saran tubing, proceed as follows:

(a) Cut a 4" length of 3/8" I.D. by 5/16" wall Neoprene or rubber bushing from a length of tubing of this size.

(b) Bevel one end of the Saran tubing on the outside, lubricate it with water, and insert it about 1" into the Neoprene bushing.

(c) Insert the bushing as far as possible into one end of the porous tube (approximately 3").

(d) With a twisting motion force the Saran tubing 3" further into the Neoprene bushing.

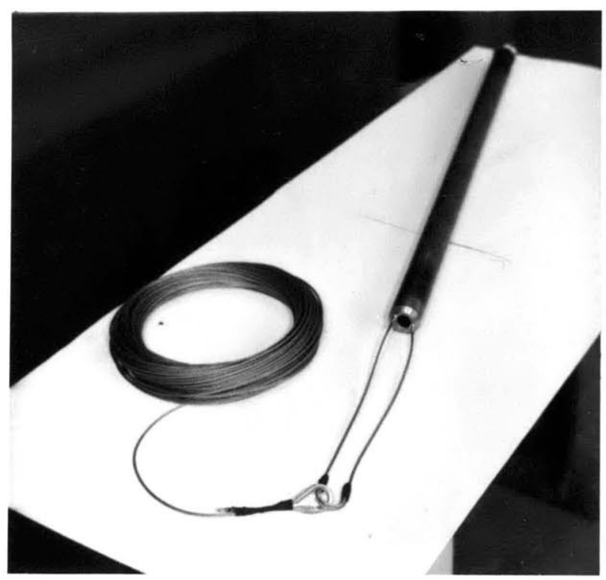
Since "it requires considerable effort to force the Saran tubing into the Neoprene bushing, it is advisable to make a strap wrench for gripping the Saran tubing." An improvised strap wrench can be made from a 3-foot piece of strong twine and a piece of wood as shown in Figure 17. When "the ends of the twine are held tightly and the wooden handle is turned in the" correct direction, "the twine tightens on the Saran tubing and acts as a wrench." To prevent twisting the Saran tubing in two while using the wrench, insert a 3/8" or 5/16" brass or steel rod 3 feet long through the porous tube and into the Saran tubing. The accompanying photograph shows the porous point assembly.

When the Saran tubing is properly installed, it is impossible for one man to pull the joint apart with his hands.

II. DESIGN AND CONSTRUCTION OF THE TAMPING HAMMER:

The purpose of the hammer is to tamp the bentonite seal in place and at the same time center the Saran tubing in the casing. In addition the tamping hammer and cable is used to measure distances from the ground to the porous tube, sand plugs, and bentonite seals.

"The hammer is made of a 3-foot length of"seamless steel tubing 1 5/8" O.D. by 1/2" wall to fit the 2" I.D. casing and the 1/2" O.D. Saran tubing. The tamping face of the hammer is made 3/16" larger in diameter by brazing a brass ring to the hammer. A loop of 1/8" 7 x 9 Galvanized Preformed Aircord is silver soldered to the upper end of the hammer. By means of two galvanized thimbles a single strand of Aircord is attached to the loop as shown in the adjacent photograph.



The connection is secured by first wrapping the cable with soft iron wire, then covering with black friction tape. A load in excess of 1500 pounds was applied to a connection, similar to that shown in the photograph, without causing failure. Figure 18 shows the design of the tamping hammer. The top of the hammer should be provided with a 45 degree bevel (see photograph) so the hammer will not catch on the casing joints while being withdrawn.

To calibrate the Aircord for taking measurements, apply approximately 20 pounds tension and mark 5-foot intervals with chalk beginning with zero at the tamping face of the hammer. As a permanent mark cut a 5" length of No. 22 soft iron wire and wedge it under two of the six strands of the Aircord. Wrap the wire around the outside of the Aircord several times, then pass the two ends once more under the two strands. Trim the wire as close to the cable as possible.

The tamping hammer may be operated with a winch, by hand or by using a pulley attached to the drill rig frame.

III. INSTALLATION OF THE PIEZOMETER IN CLAY

(a) Drive a cased hole "to the elevation planned for the bottom of the permeable space." 2" I.D. extra strong pipe casing should be used unless the installation is made at a shallow depth through soft clay or silt. In this case, standard 2" I.D. casing may be used. The bottom section of casing "should be at least 10 feet long" and should not have a coupling or drive shoe at its lower end. All sections of pipe should be reamed at both ends to remove sharp edges which may damage the Saran tubing. Measure and record the exact length of casing as the pipe is driven. File marks on the casing are the most permanent means of marking the casing as it is driven. When driving the casing, no washing should be done below the bottom. This will assure a tight contact between the outside of the casing and the surrounding soil.

(b) Measure the length of wash pipe and wash the inside of the casing clean to the bottom. Replace the dirty water in the casing with clean water by reversing the flow and pumping the dirty water up through the wash pipe. Keep the casing filled with clean water until all the dirty water has been pumped out.

If the wash bit is provided with jets projected to the side or upward the water may be changed without reversing the flow. The object is to keep from washing below the bottom of the casing since this point represents the bottom of the permeable space over which the water pressure is measured.

(c) Retract the casing 2 feet, either by jacking or by using a slip weight. Backfill the 2 feet with a measured amount of washed and screened sand passing a #20 and retained on a #35 mesh sieve. Standard Ottawa Sand (C-190) is ideal for this purpose. To prevent the sand from carrying air bubbles to the bottom, saturate it and pour into the casing by means of a funnel. Allow a minute or two for the sand to reach the bottom, then check the elevation by means of the tamping hammer. Since continued tamping will work the hammer down into the sand, care must be taken to obtain a true elevation. However, keep the hammer in motion so any sand above it will work past the tamping face and not settle and wedge around the brass ring.

The volume of sand needed for backfilling should be computed and closely controlled. A quart jar may be marked to show the volumes required for the different stages of installation.

(d) To install the porous point, first saturate the Norton porous tube by lowering it <u>slowly</u> into the clean water in the casing. Lay the roll of Saran tubing flat on the ground and connect the end to a piece of rubber tubing in turn connected to an aspirator bottle. By means of a small pump, apply a vacuum to the bottle to fill the Saran tubing completely and to obtain a reservoir in the aspirator bottle. Disconnect the pump and lower the porous point in the casing, at all times maintaining a small excess head with the reservoir. This will assure a flow of water out of the point while it is lowered into place.

(e) With the porous point resting on the sand in the bottom of the hole, retract the casing 2 feet more. Measure the elevation of the top of the Norton tube with the tamping hammer, then backfill around the tube with a measured amount of saturated sand. It is very unlikely that the retained clay will cave in after the casing is pulled. However, the sand backfill should be placed as soon as possible. To prevent dirt from getting inside the Saran tubing, plug the end with a No. 00 rubber stopper. When used as a permanent plug, the rubber stopper must have a small hole drilled through it for an air vent.

(f) Pull the casing 1 foot to its final position and backfill with saturated sand to a point 2 to 3 feet above the bottom of the casing. Tamp with the hammer and record the elevation. "The purpose of this sand plug is to minimize the effect of swelling pressures"caused by the bentonite seal.

(g) Prepare commercial bentonite in a stiff putty-like state and roll into balls about 1/2" in diameter. Lower the water in the casing until it is 3" from the top, then drop the bentonite balls one by one into the casing until the water level is raised back to the top. This will give approximately a 3" layer after the bentonite is tamped. To keep the bentonite from sticking to the hammer, drop enough rounded pebbles 1/4" to 3/8" in diameter to form a 3/4" layer on top of the bentonite balls. <u>The importance of using only rounded pebbles of this size cannot be overemphasized</u>. The pebbles should be rounded to prevent injury to the Saran tubing as the seal is tamped in place. <u>None</u> should be smaller than 1/4", to prevent their by-passing the tamping face of the hammer and wedging the hammer in the casing.

After allowing time for the pebbles to reach the bottom, lower the hammer and tamp the seal with 20 blows "applied by raising the hammer about 6" and allowing it to drop freely."

Repeat the process until 5 well tamped layers of bentonite have been placed. Measure the elevation of the final layer for a check on the length of seal which should be about 18".

(h) As an additional precaution against leakage, a second seal of 5 layers of bentonite may be added after placing and tamping a 2-foot plug of sand. In any case the final seal should be capped with several feet of sand to confine the swelling bentonite. See Figure 19 for a cross section of the piezometer installation.

IV. INSTALLATION OF THE PIEZOMETER IN SAND:

If at any time it is desirable to install a piezometer in a stratum of sand, the following precautions and changes in procedure are offered.

After the 2" casing has been driven to the desired depth and washed clean, the withdrawal of the wash pipe will lower the water in the casing to a point where a quick condition is possible in the sand at the bottom of the casing. Sand is likely to wash into the casing to a considerable height. To prevent this, the casing should be kept full of water as the wash pipe is withdrawn.

The object of having as much as 5 feet of clay exposed to the piezometer is for the purpose of increasing its sensitivity. Since an installation in sand presents no such problem, the amount of sand above and below the porous tube may be cut to a minimum. In general if any part of the porous point is within the sand stratum, it should function properly.

When retracting the casing, the sand in some cases will cave in. This should be of no consequence, however, as long as the porous point has been placed before the casing is retracted from around it.

It is no longer important to wash to a point exactly at the bottom of the casing since the piezometer reading does not depend on the point at which the piezometer is located in the sand.

V. EQUIPMENT FOR MEASURING THE WATER LEVEL:

Methods of measuring the water level under three conditions will be discussed. First, when the water level stands at a point in the transparent Saran tubing to permit direct observations; second, when the water level is below a point permitting direct readings, and; third, when the water level tends to rise higher than the top of the tube and pressure apparatus must be installed.

The first case is obvious. Whenever possible, direct readings should be taken.

A simple electrical sounding device may be constructed for measuring the water level for the second case. Two strands of insulated wire are taped together and marked off at regular intervals for taking measurements. The wires are bared at one end and connected at the other to a suitable volt-ohmeter which deflects the instant contact is made with the water. Figure 20 shows the design of the contact point.

To construct the sounding device, obtain a sufficient length of #22 B and S 7/30 Tinned Copper Neoprene wire. Double the wire and prepare one end of the doubled wire as follows: Bare the ends for about 1/4" and space them 3/16" apart by applying a plug of sealing wax above the bared ends. As an alternate means of spacing, a 1/2" piece of 1/4" round plastic rod may be used. Two holes are drilled for the wire which is cemented in place.

Cut 15 1" square pieces of sheet lead approximately 1/32" thick and wrap around the two wires at intervals of 1". This will provide the weight necessary to carry the contact point down the tube and keep the wire taut as readings are taken. When placing the weights on the wires care must be exercised not to damage the insulation. All sharp edges on the lead weights should be filed smooth.

Connect the other ends of the insulated wire to a Weston Electric Volt-Ohmeter, Model #564, Type 3C. If connected to the proper poles the volt-ohmeter will give distinct readings about half way across the scale when the contact points touch the water.

To mark off the insulated wire for taking measurements, ordinary 1/2" adhesive tape may be used at 5-foot or 1-foot intervals. Numbers written on the tape with India ink are waterproof and relatively permanent. The spacing of the marks should be checked occasionally to assure that none has slipped. The best grip is obtained by wrapping one turn of the tape around one strand of the wire before wrapping both strands.

The contact point should be covered with a thin coating of grease at all times. Since the depth reading is checked by making and breaking the water contact several times in succession, it is essential that the water break free of the points as they are withdrawn.

For the third case, that where water tends to overflow the top of the tube, the excess pressure can be measured with a Bourdon Gage. To install a Bourdon Gage proceed as follows: Attach a 2-foot length of 3/8" I.D. rubber pressure tubing to the end of the plastic tubing. Connect the other end of the rubber tube to the Bourdon Gage after filling the threads at the connection with rubber cement or Pliobond to prevent leaking. Fasten the ends of the rubber tubing to the standpipe and Bourdon Gage by wrapping them with soft iron wire. Finally, fasten the Bourdon Gage to a wall or stand such that the center of the gage is not higher than the standpipe.

Whenever the pressure in the gage drops to zero, the Bourdon Gage should be detached from the standpipe and observations taken directly.

VI SERIES III - V CONSOLIDATION TEST DATA

CONSOLIDATION TEST III-1

Jan. 21, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Natural Water Content:
Top = 50.1% Middle = 49.1% Bottom = 46.6%

Approximate Initial Thickness = 1.28 in.

Weight Soil Solids = 355.1 g

Height Soil Solids = 0.542 in.

Apparatus:

Barrel: Height z_1 = 2.500 in., Diameter = 4.29 in.

Thickness of Rubber Membrane = 0.01 in.

Distance Equivalent to z_2 = 1.729

Loading Unit No. III

Loading:

1 kg. per sq. cm. = 205 lbs. Scale Load, Tare = 37.0 lbs.

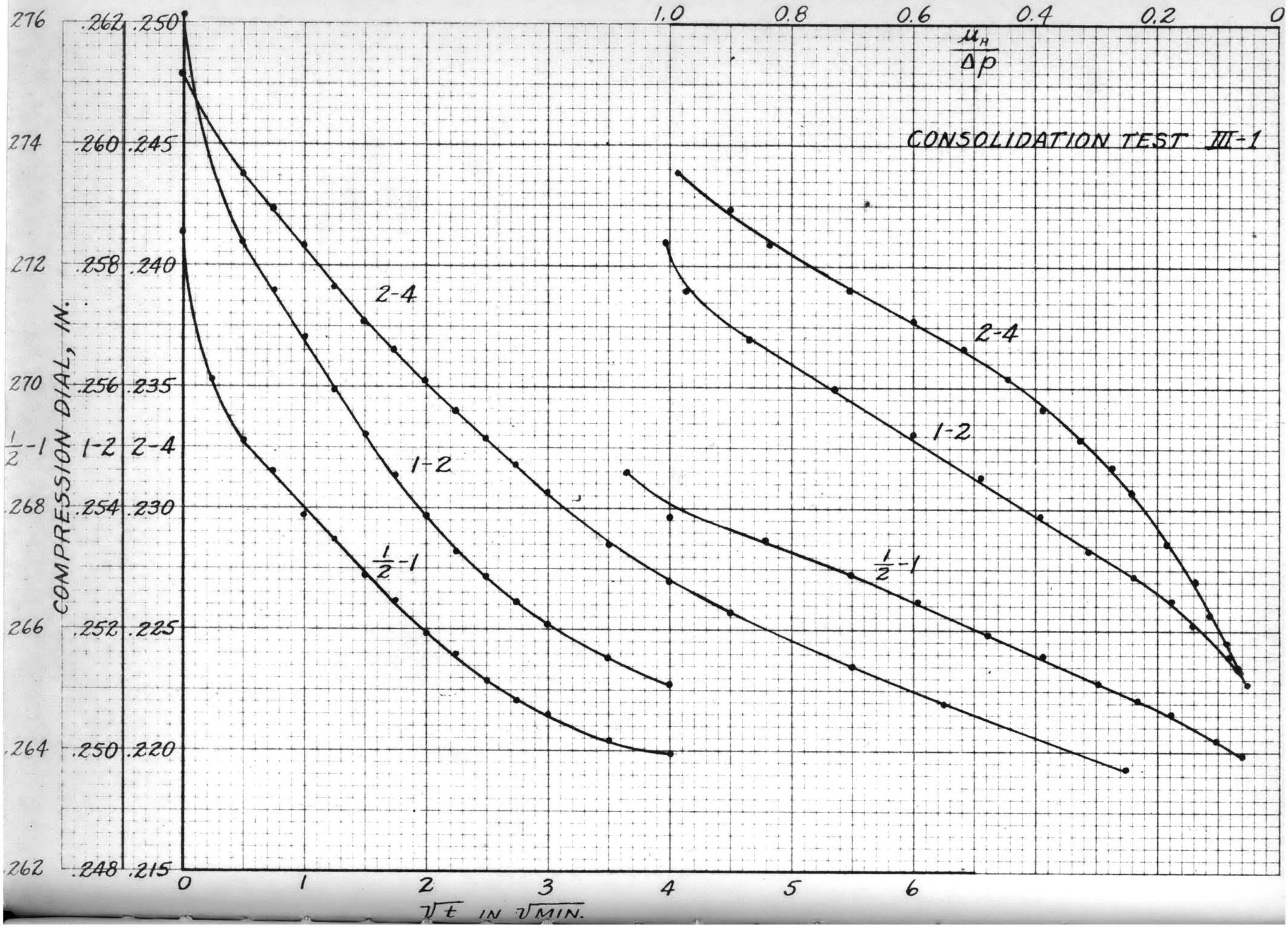
Increment Duration: (See table)

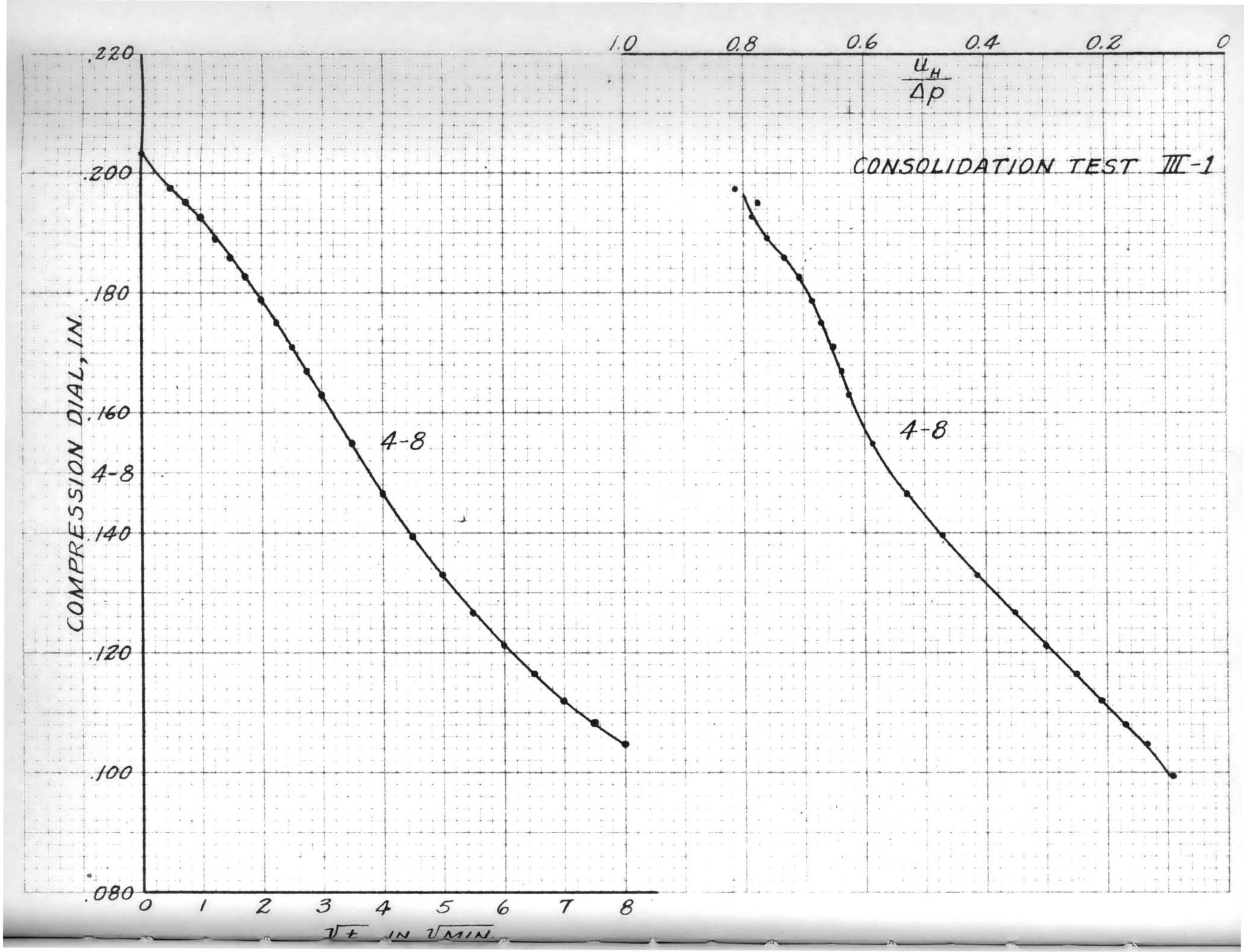
Date	Time	Applied Load kg./sq.cm.	Final Dial in.	Void Ratio
1/31/51	9:45 P.M.	0.25	.27857	1.372
2/1/51	1:08 P.M.	0.475	.27256	1.362
2/2/51	1:04 P.M.	1	.26215	1.342
2/3/51	1:02 P.M.	2	.24802	1.317
2/4/51	1:00 P.M.	4	.20357	1.234
2/5/51	1:05 P.M.	8	.07810	1.004
2/6/51	1:07 P.M.	4	.08703	1.020
2/6/51	2:19 P.M.	5	.08513	1.016
2/7/51	2:36 P.M.	6	.08315	1.012

Remarks:

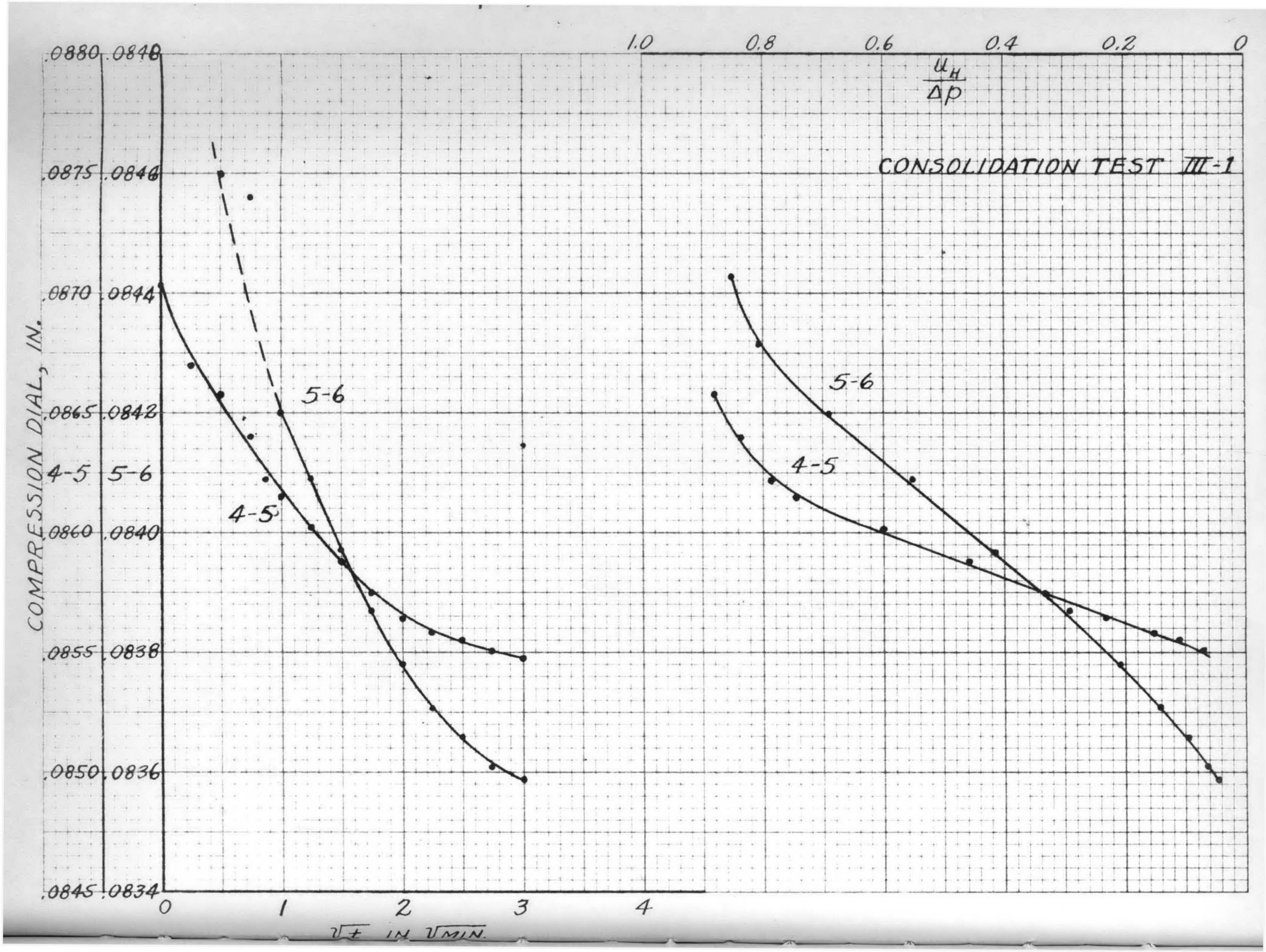
z_3 = 0.485 in. at Dial = .24802 in.

Had difficulty inserting pore pilot. Test believed to be satisfactory however.


CONSOLIDATION TEST III-1
$\frac{u_H}{\Delta p}$
1.0
0.8
0.6
0.4
0.2
0
276 .262 .250
274 .260 .245
272 .258 .240
270 .256 .235
$\frac{1}{2}$-1 1-2 2-4
268 .254 .230
266 .252 .225
264 .250 .220
262 .248 .215
COMPRESSION DIAL, IN.
0
1
2
3
4
5
6
$\sqrt{t}$ IN $\sqrt{MIN.}$
2-4
1-2
$\frac{1}{2}$-1
2-4
1-2
$\frac{1}{2}$-1



CONSOLIDATION TEST III-1
COMPRESSION DIAL, IN.
.220
.200
.180
.160
4-8
.140
.120
.100
.080
0
1
2
3
4
5
6
7
8
$\sqrt{t}$ IN $\sqrt{MIN.}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
4-8
4-8



CONSOLIDATION TEST III-1
$\frac{u_H}{\Delta p}$
1.0
0.8
0.6
0.4
0.2
0
COMPRESSION DIAL, IN.
.0880 .0848
.0875 .0846
.0870 .0844
.0865 .0842
4-5 5-6
.0860 .0840
.0855 .0838
.0850 .0836
.0845 .0834
0
1
2
3
4
$\sqrt{t}$ IN $\sqrt{MIN.}$
5-6
4-5
5-6
4-5

CONSOLIDATION TEST III-1

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$- 1 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
30	.26357
54	.26319
85	.26307
118	.26296
204	.26277
344	.26265
404	.26261
536	.26259
1264	.26259 =.26220
1428	.26215

Load Increment 1- 2 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
41	.25011
62	.24980
225	.24891
1342	.24807
1425	.24802

Load Increment 2- 4 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
124	.21555
140	.21488
310	.21075
1150	.20458
1357	.20375
1440	.20357

Load Increment 4- 8 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
81	.0996
106	.0950
139	.0916
175	.08882
238	.08627
372	.08365
610	.08113
1150	.07885
1440	.07810

Load Increment 4- 5 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
16	.08537
25	.08529
51	.08521
108	.08518
333	.08517
420	.08516
1119	.08515
1440	.08513

Load Increment 5- 6 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
15	.08356
66	.08352
76	.08351 ?
110	.08315 ?

CONSOLIDATION TEST III-2

Feb. 10, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Natural Water Content:
Top = 45.4% Middle = 45.1% Bottom = 50.3%

Approximate Initial Thickness = 1.28 in.

Weight Soil Solids = 351.0 g

Height Soil Solids = 0.535 in.

Apparatus:

Barrel: Height z_1 = 2.500 in., Diameter = 4.29 in.

Thickness of Rubber Membrane = 0.01 in.

Distance Equivalent to z_2 = 1.729

Loading Unit No. III

Loading:

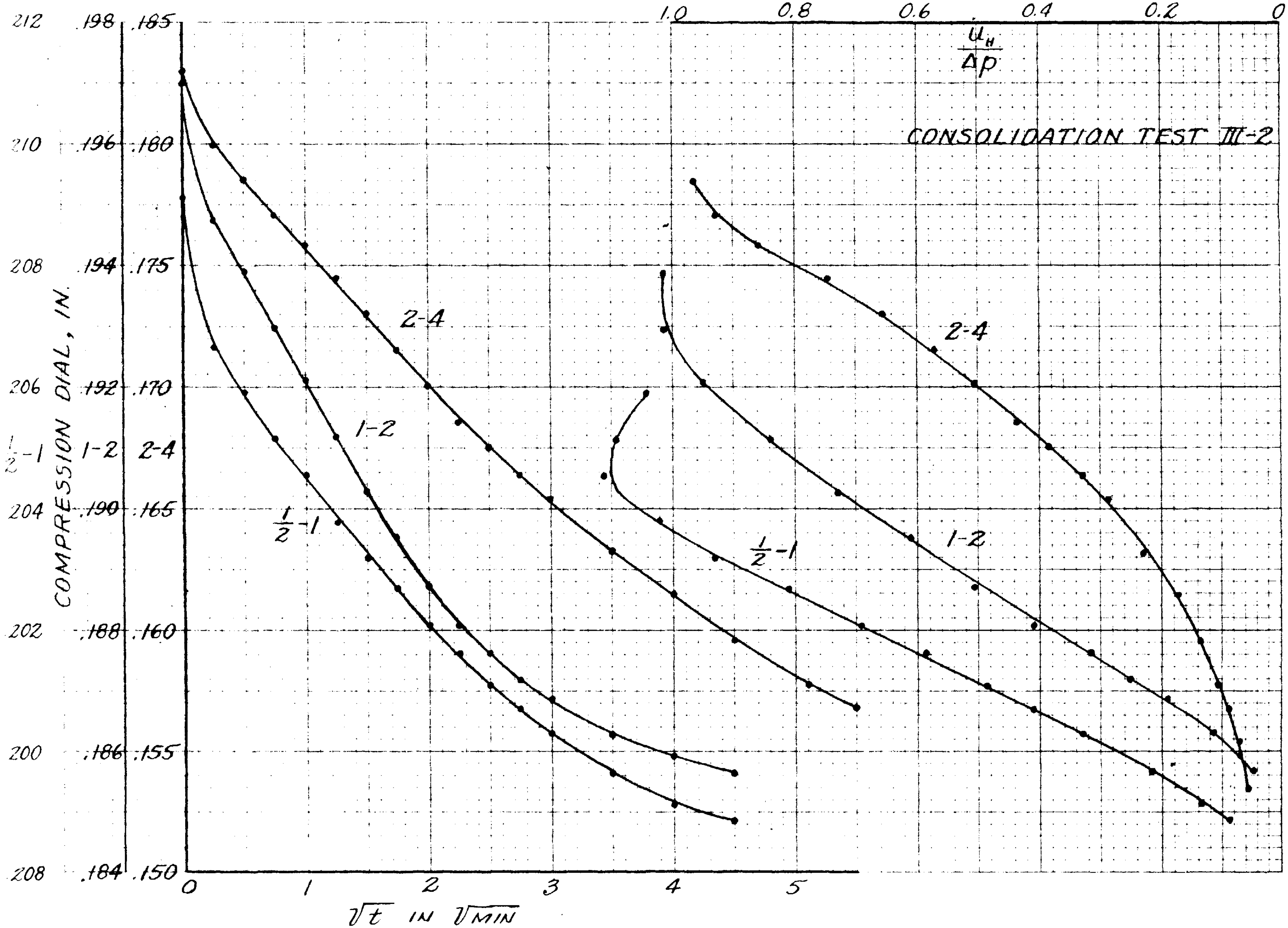
1 kg. per sq. cm. = 205 lbs. Scale Load, Tare = 37.0 lbs.

Increment Duration: (See table)

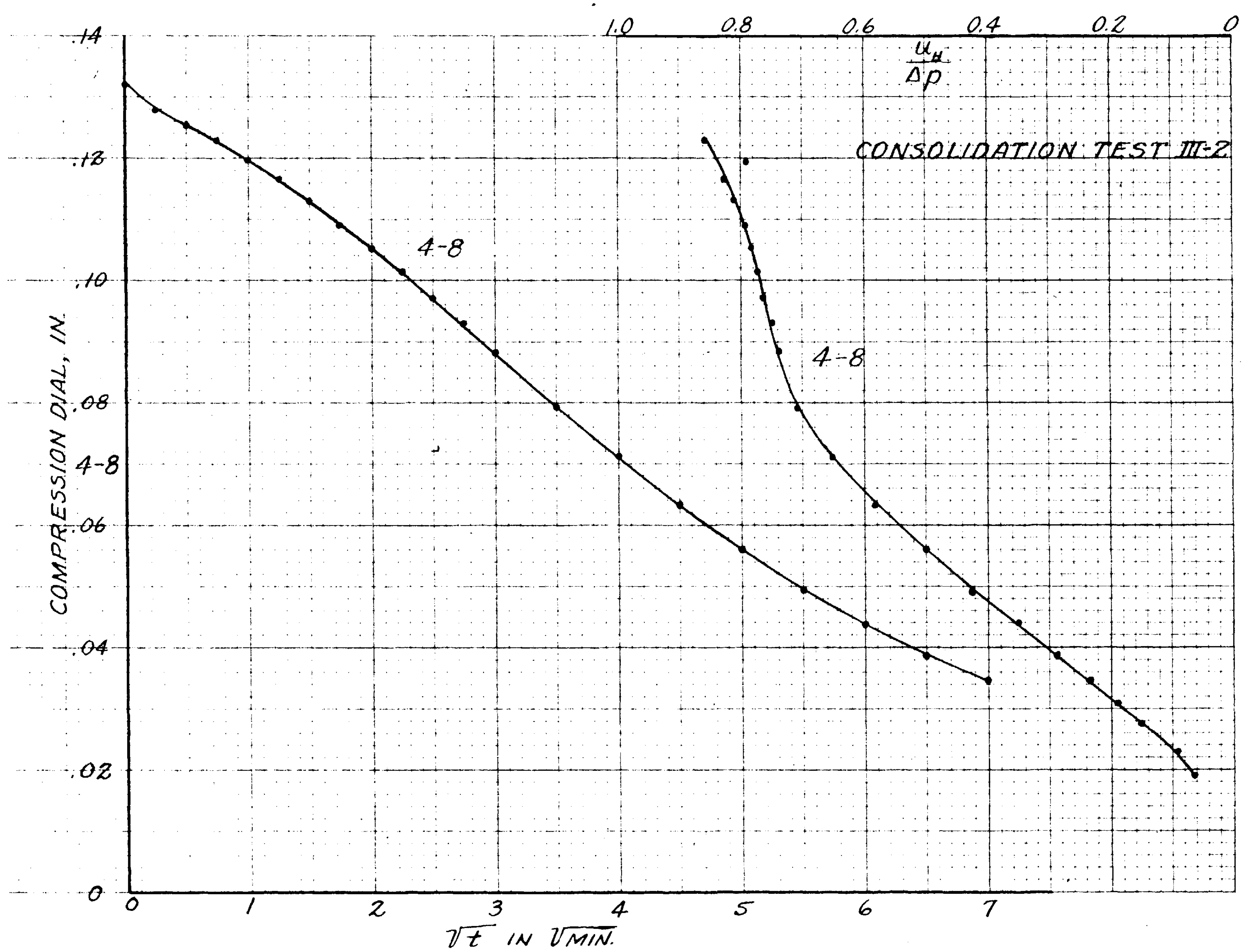
Date	Time	Applied Load kg./sq. cm.	Final Dial in.	Void Ratio
2/10/51	11:30 A.M.	0.1	.24100	1.439
2/10/51	1:47 P.M.	0.25	.22716	1.412
2/11/51	1:50 P.M.	0.5	.20913	1.379
2/12/51	2:09 P.M.	1	.19702	1.357
2/13/51	2:18 P.M.	2	.18308	1.331
2/14/51	2:20 P.M.	4	.13200	1.235
2/15/51	2:19 P.M.	8	.00390	.995
2/16/51	2:22 P.M.	4	.01319	1.013
2/18/51	2:09 P.M.	5	.01115	1.009
2/19/51	2:24 P.M.	6	.00806	1.003

Remarks:

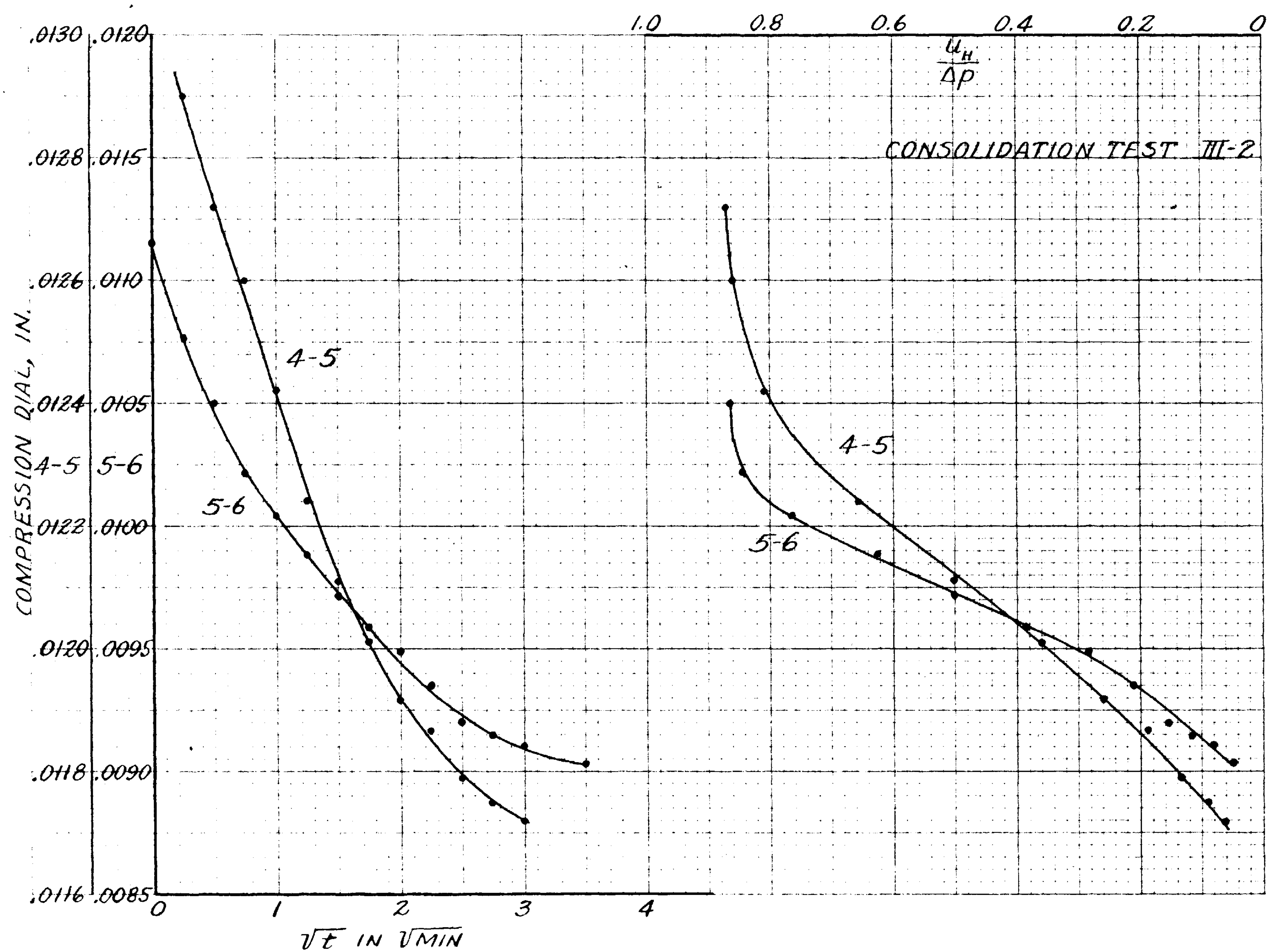
z_3 = 0.425 in. at Dial = .13200 in.



CONSOLIDATION TEST III-2
COMPRESSION DIAL, IN.
212 .198 .185
210 .196 .180
208 .194 .175
206 .192 .170
1/2-1 1-2 2-4
204 .190 .165
202 .188 .160
200 .186 .155
208 .184 .150
0 1 2 3 4 5
√t IN √MIN
1.0 0.8 0.6 0.4 0.2 0
$\frac{u_H}{\Delta p}$
2-4
1-2
1/2-1



CONSOLIDATION TEST III-2
COMPRESSION DIAL, IN.
4-8
4-8
4-8
$\frac{u_H}{\Delta p}$
1.0
0.8
0.6
0.4
0.2
0
.14
.12
.10
.08
.06
.04
.02
0
0
1
2
3
4
5
6
7
$\sqrt{t}$ IN $\sqrt{MIN.}$


CONSOLIDATION TEST III-2
COMPRESSION DIAL, IN.
4-5
5-6
.0130
.0128
.0126
.0124
.0122
.0120
.0118
.0116
.0120
.0115
.0110
.0105
.0100
.0095
.0090
.0085
0
1
2
3
4
$\sqrt{t}$ IN $\sqrt{MIN}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
4-5
5-6
4-5
5-6

CONSOLIDATION TEST III-2

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$ - 1 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
37	.19837
84	.19792
202	.19761
246	.19755
311	.19750
476	.19740
1091	.19712
1440	.19702

Load Increment 1 - 2 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
36	.18511
83	.18454
161	.18412
288	.18386
443	.18363
1072	.18323
1338	.18312
1440	.18308

Load Increment 2 - 4 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
37	.15548
49	.15357
60	.15210
91	.14901
161	.14516
1085	.13360
1172	.13307
1406	.13212
1440	.13200

Load Increment 4 - 8 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
56	.0312
64	.0277
81	.0236
100	.0195
135	.0159
262	.0107
489	.0072
1106	.0046
1440	.0039

Load Increment 4 - 5 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
15	.01166
29	.01155
45	.01152
153	.01140
1081	.01119
1440	.01115

Load Increment 5 - 6 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
31	.00887
54	.00878
138	.00861
293	.00844
546	.00819
1056	.00808
1254	.00806

CONSOLIDATION TEST III-3

March 1, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Natural Water Content:
Top = 41.0% Middle = 41.3% Bottom = 40.0%

Approximate Initial Thickness = 1.28 in.

Weight Soil Solids = 382.8 g

Height Soil Solids = 0.584 in.

Apparatus:

Barrel: Height z_1 = 2.50 in., Diameter = 4.29 in.

Thickness of Rubber Membrane = 0.01 in.

Distance Equivalent to z_2 = 1.729

Loading Unit No. III

Loading:

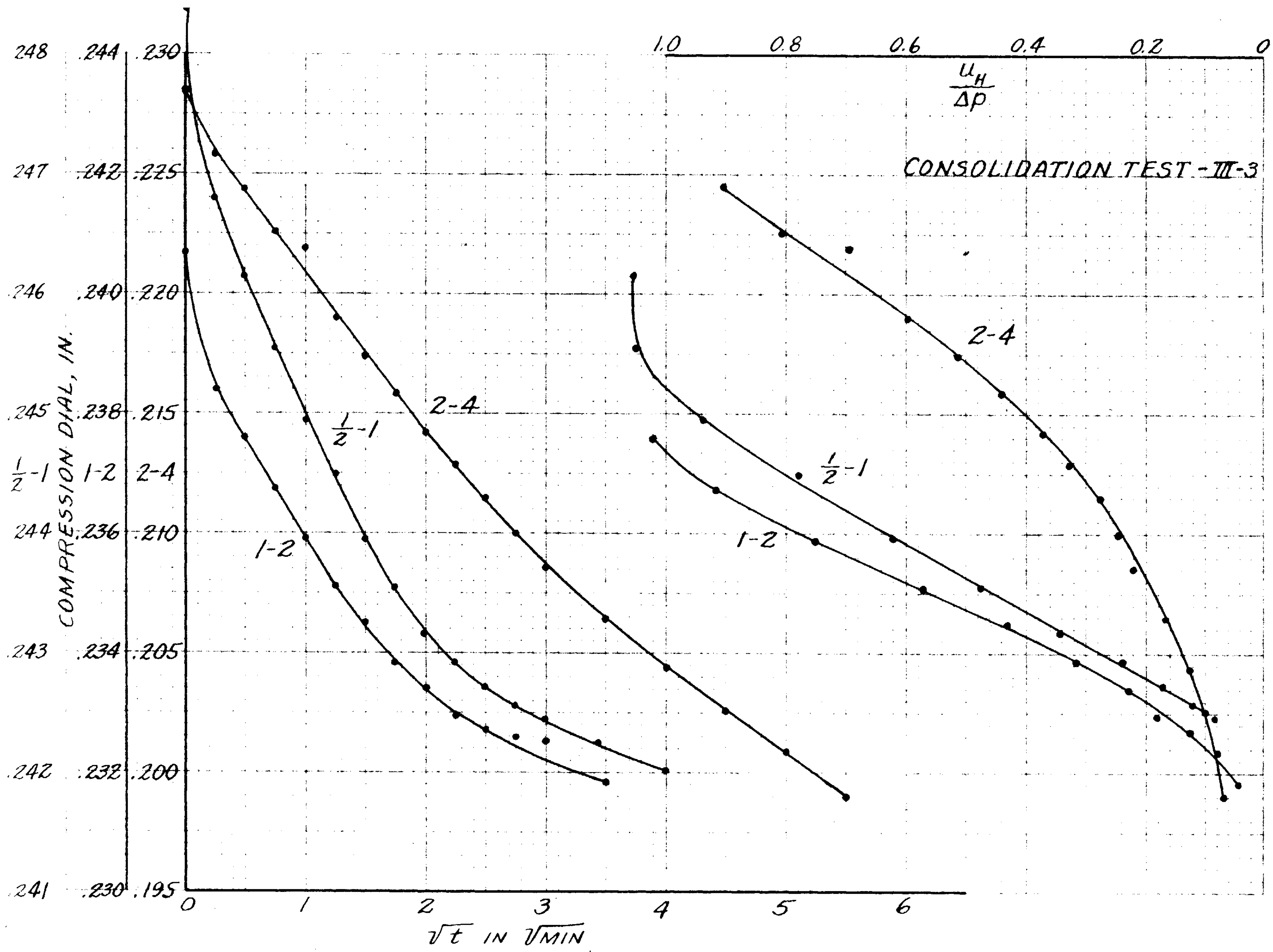
1 kg. per sq. cm. = 205 lbs. Scale Load, Tare = 37.0 lbs.

Increment Duration: (See table)

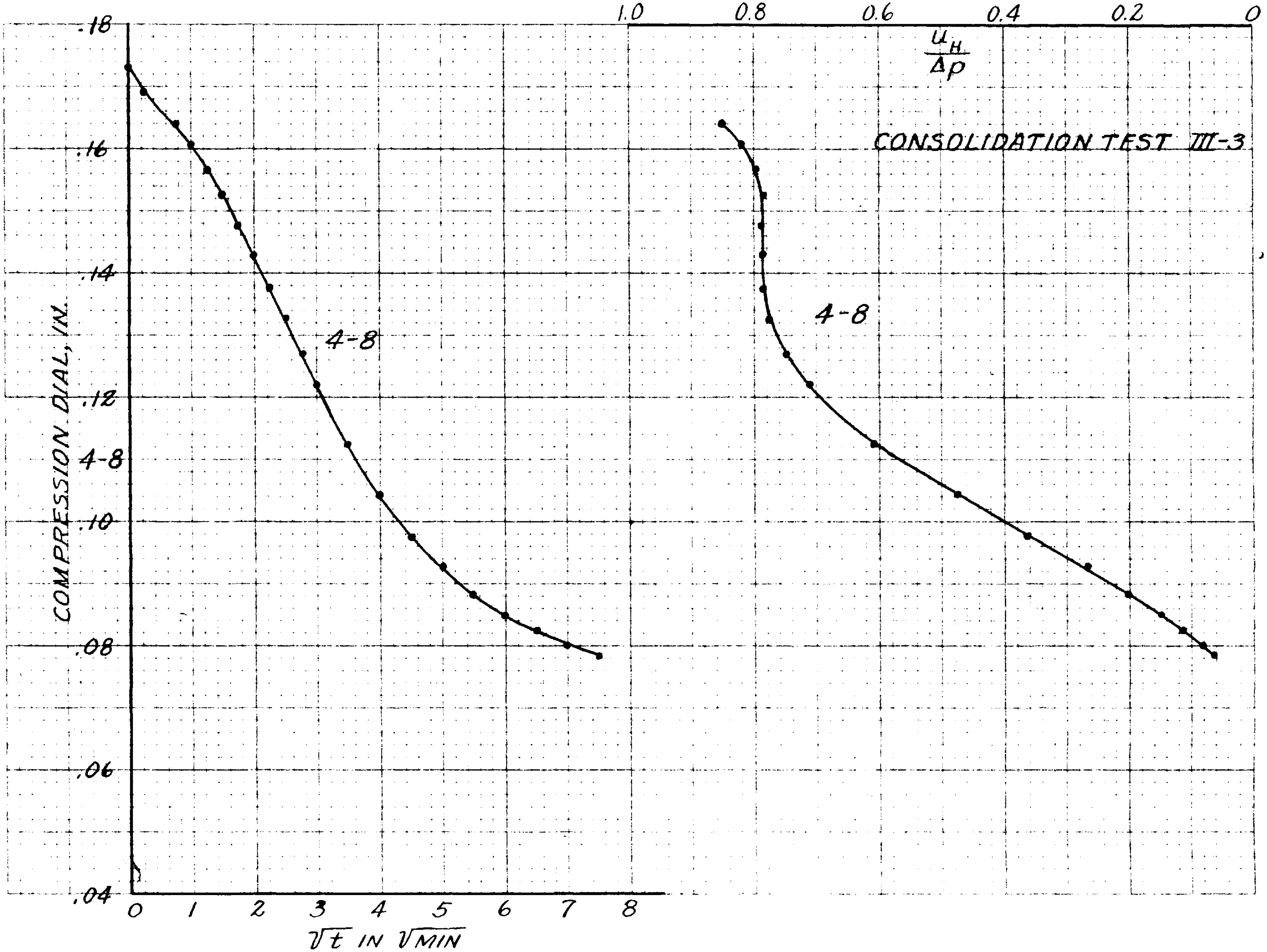
Date	Time	Applied Load kg./sq. cm.	Final Dial in.	Void Ratio
3/1/51	3:50 P.M.	0.1	.26159	1.186
3/1/51	7:45 P.M.	0.25	.25511	1.176
3/2/51	7:43 P.M.	0.5	.24848	1.163
3/3/51	7:55 P.M.	1	.24070	1.150
3/4/51	7:52 P.M.	2	.22853	1.129
3/5/51	7:47 P.M.	4	.17355	1.036
3/6/51	7:41 P.M.	8	.06370	0.848
3/7/51	7:26 P.M.	4	.07098 [= .07139	0.860
3/24/51	8:47 A.M.	5	.06972	0.857
3/25/51	8:37 A.M.	6	.06734	0.854

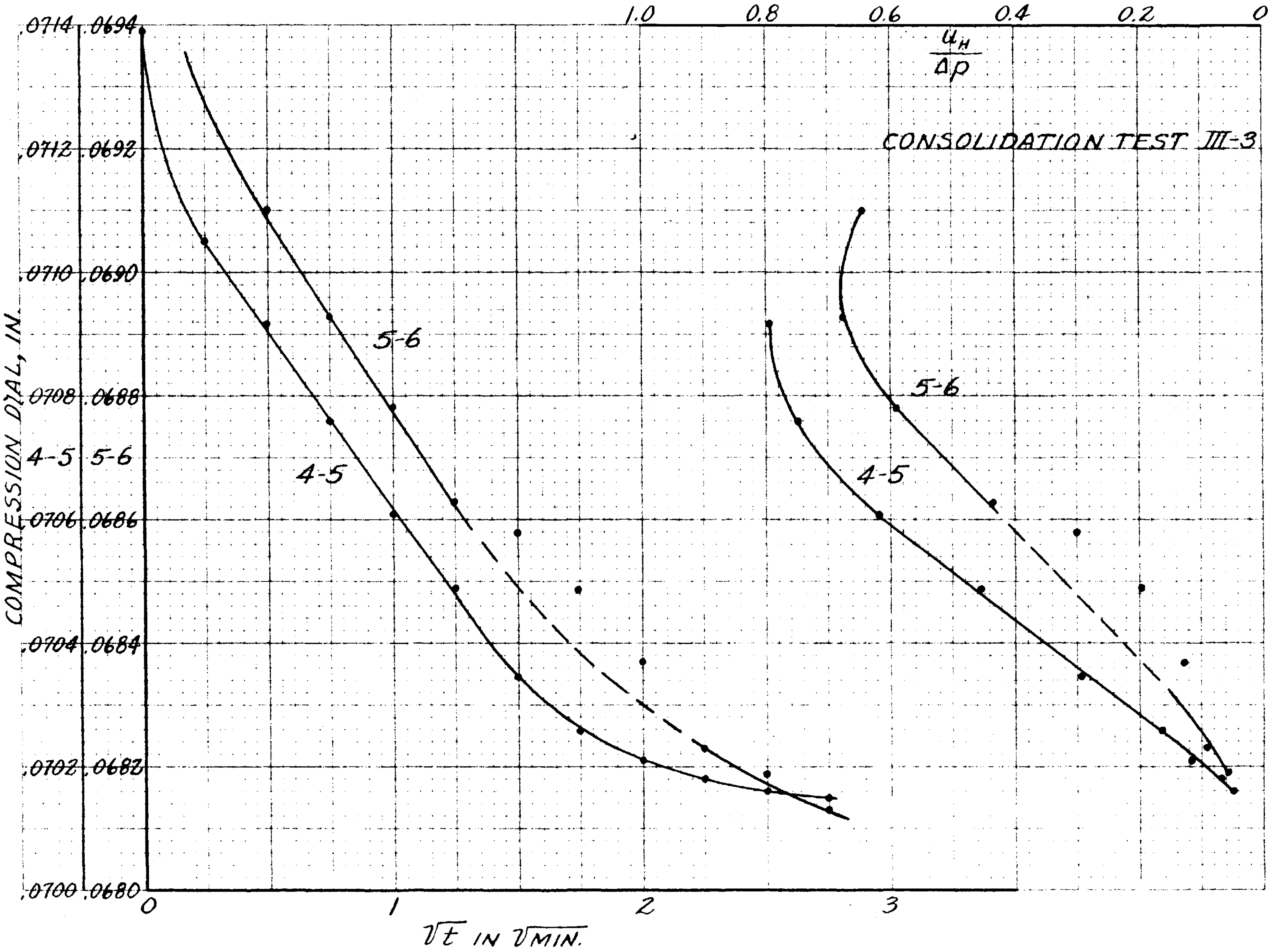
Remarks:

z_3 = 0.473 in. at Dial = .22853 in.



CONSOLIDATION TEST - III-3
COMPRESSION DIAL, IN.
$\frac{1}{2}$-1 1-2 2-4
248 .244 .230
247 .242 .225
246 .240 .220
245 .238 .215
244 .236 .210
243 .234 .205
242 .232 .200
241 .230 .195
$\sqrt{t}$ IN $\sqrt{MIN}$
0 1 2 3 4 5 6
$\frac{u_H}{\Delta p}$
1.0 0.8 0.6 0.4 0.2 0
2-4
$\frac{1}{2}$-1
1-2


CONSOLIDATION TEST III-3
COMPRESSION DIAL, IN.
.18
.16
.14
.12
4-8
.10
.08
.06
.04
0
1
2
3
4
5
6
7
8
√t IN √MIN
4-8
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
4-8



CONSOLIDATION TEST III-3
COMPRESSION DIAL, IN.
4-5
5-6
.0714
.0694
.0712
.0692
.0710
.0690
.0708
.0688
.0706
.0686
.0704
.0684
.0702
.0682
.0700
.0680
0
1
2
3
$\sqrt{t}$ IN $\sqrt{MIN.}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
5-6
4-5
5-6
4-5

CONSOLIDATION TEST III-3

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$ - 1 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
33	.24171
59	.24157
805	.24083
1080	.24078
1430	.24070

Load Increment 1 - 2 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
27	.23111
44	.23076
92	.23019
132	.22998
173	.22980
768	.22888
1283	.22858
1430	.22853

Load Increment 2 - 4 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
62	.1937
127	.1879
162	.1862
755	.1768
1118	.1745
1430	.1736

Load Increment 4 - 8 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
106	.0729
208	.0690
755	.0651
1419	.0637

Load Increment 4 - 5 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
16	.07011
36	.07006
49	.07004
155	.07997
448	.07989
1415	.06972

Load Increment 5 - 6 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
16	.06808
30	.06801
118	.06788
247	.06772
572	.06759
1454	.06749
2082	.06734

CONSOLIDATION TEST V-1

Jan. 7, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Average Natural Water Content = 46.4%
(See Figure for Distribution)

Approximate Initial Thickness = 4.00 in.

Weight Soil Solids = 5646 g

Height Soil Solids = 1.738 in.

Apparatus:

Barrel: Height z_1 = 7.465 in., Diameter = 9.55 in.

Thickness of Rubber Membrane = 0.01 in.

Average Thickness Top Porous Stone and First Cover Plate = 1.325 in.

Loading Unit No. V with Special Leverage System

Loading:

1 kg. per sq.cm. = 238 lbs. Scale Load, Tare = 31.2 lbs.

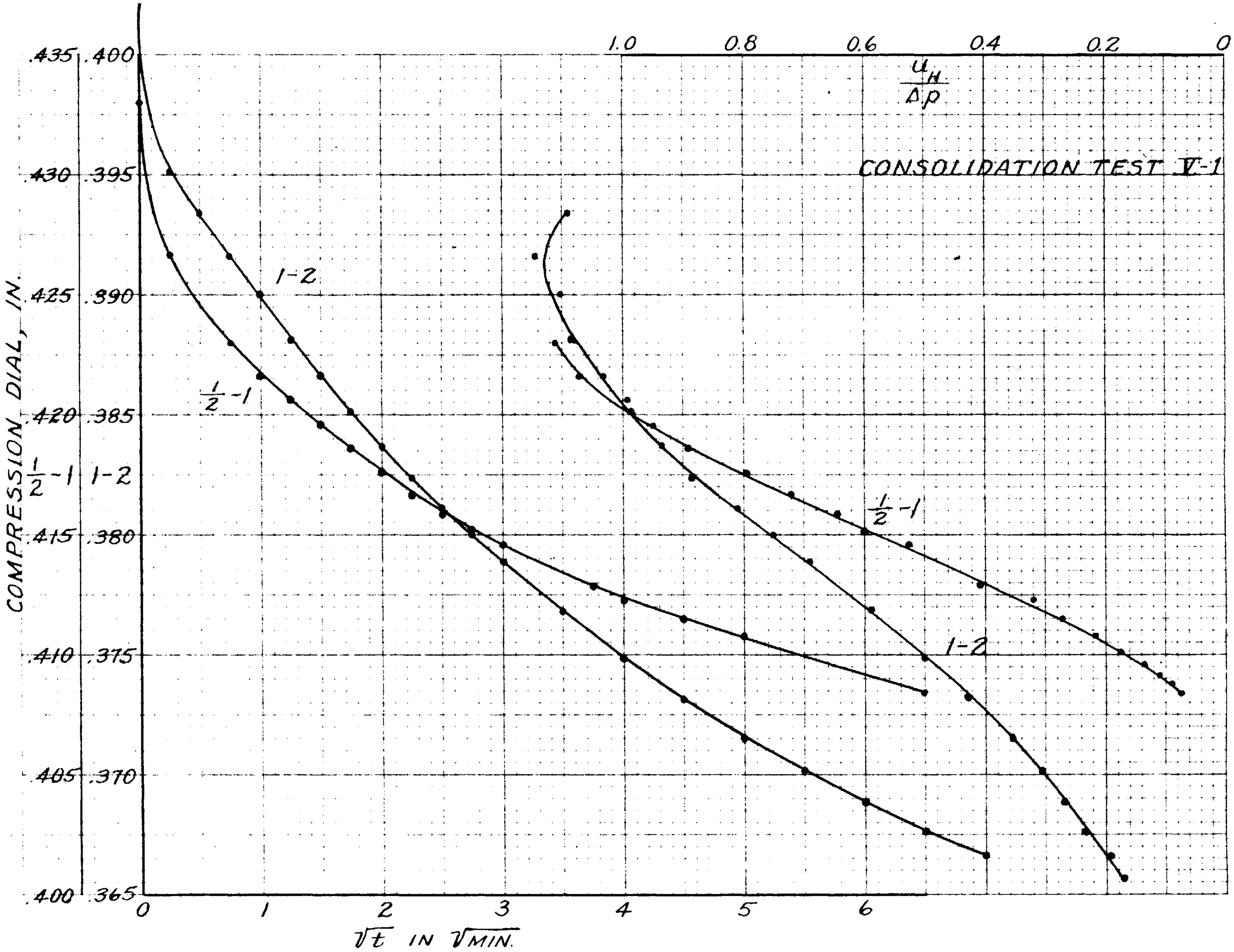
Increment Duration: (See table)

Date	Time	Applied Load kg./sq.cm.	Final Void in.	Void Ratio
1/7/51		0.058	.47300	1.273
1/8/51	11:03 A.M.	0.25	.45170	1.261
1/9/51	10:59 A.M.	0.5	.43300	1.252
1/10/51	10:57 A.M.	1	.40355	1.234
1/11/51	11:03 A.M.	2	.35300	1.204
1/12/51	11:01 A.M.	4	[.17810 =.42320	1.103
1/13/51	11:15 A.M.	8	.09580	0.916
1/14/51	11:09 A.M.	4	.11350	0.926
1/14/51	1:29 P.M.	5	.10992	0.924
1/14/51	10:40 P.M.	6	.09900	0.918

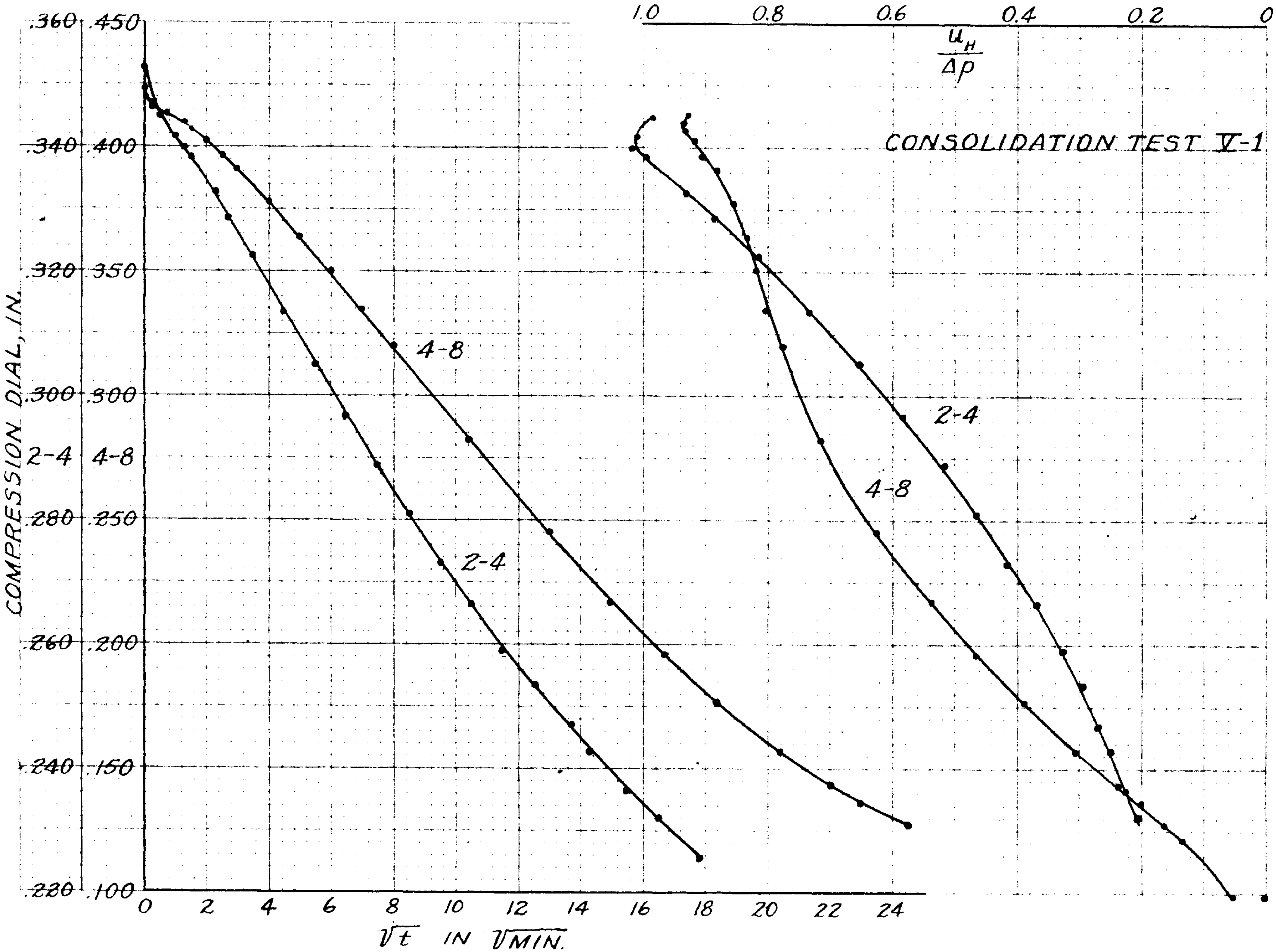
Remarks:

z_3 = (-)2.807 in. at Dial = .09858 in.

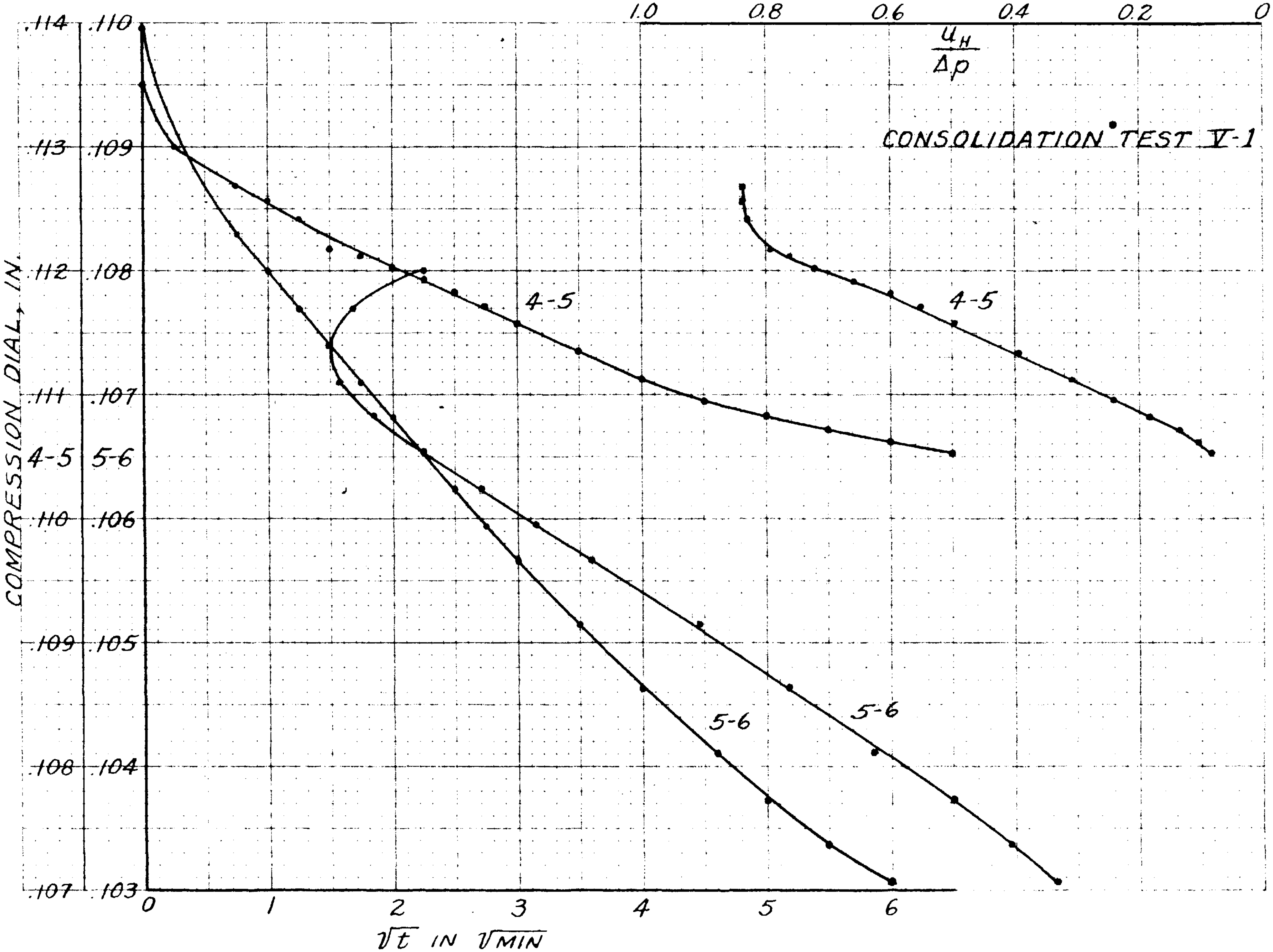
Loading beam tilted considerably in load increments from 4-8 to end of test.



CONSOLIDATION TEST V-1
$\frac{u_H}{\Delta p}$
1.0
0.8
0.6
0.4
0.2
0
COMPRESSION DIAL, IN.
$\frac{1}{2}$-1
1-2
.435
.400
.430
.395
.425
.390
.420
.385
.415
.380
.410
.375
.405
.370
.400
.365
0
1
2
3
4
5
6
$\sqrt{t}$ IN $\sqrt{MIN.}$



CONSOLIDATION TEST V-1
COMPRESSION DIAL, IN.
2-4
4-8
.360 .450
.340 .400
.320 .350
.300 .300
.280 .250
.260 .200
.240 .150
.220 .100
0
2
4
6
8
10
12
14
16
18
20
22
24
$\sqrt{t}$ IN $\sqrt{MIN.}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
4-8
2-4
2-4
4-8



CONSOLIDATION TEST V-1
COMPRESSION DIAL, IN.
.114 .113 .112 .111 .110 .109 .108 .107
.110 .109 .108 .107 .106 .105 .104 .103
4-5 5-6
0 1 2 3 4 5 6
√t IN √MIN
1.0 0.8 0.6 0.4 0.2 0
$\frac{u_H}{\Delta p}$
4-5
4-5
5-6
5-6

CONSOLIDATION TEST V-1

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$ - 1 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
30	.41012
36	.40959
42	.40913
49	.40877
56	.40843
145	.40656
263	.40556
383	.40509
1446	.40355

Load Increment 1 - 2 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
56	.36570
64	.36495
80	.36360
144	.36059
288	.35758
319	.35674
626	.35531
1427	.35300

Load Increment 2 - 4 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
350	.2215
365	.2200
437	.2135
586	.2042
646	.2002
1305	.1820
1454	.1781

Load Increment 4 - 8 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
682	.1198
1293	.0990
1434	.0958

Load Increment 4 - 5 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
154	.11005
290	.10995
407	.10994
1440	.10992

Load Increment 5 - 6 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
42	.10272
605	.09950

CONSOLIDATION TEST V-2

Jan. 24, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Average Natural Water Content = 46.0%
(See Figure for Distribution)

Approximate Initial Thickness = 4.00 in.

Weight Soil Solids = 5651 g

Height Soil Solids = 1.741 in.

Apparatus:

Barrel: Height z_1 = 7.465 in., Diameter = 9.55 in.

Thickness of Rubber Membrane = 0.01 in.

Average Thickness Top Porous Stone and First Cover Plate = 1.281 in.

Loading Unit No. V with Special Leverage System

Loading:

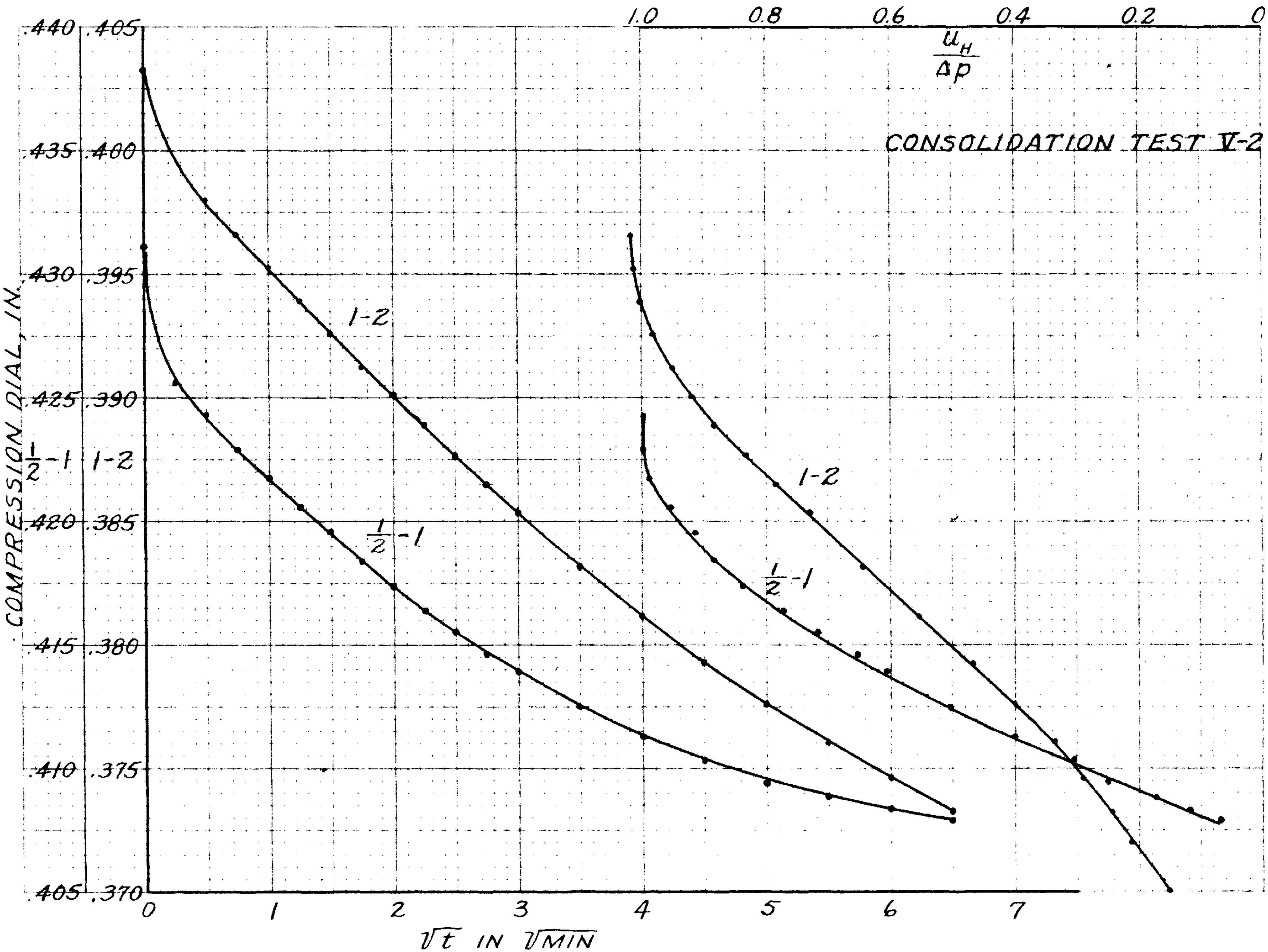
1 kg. per sq. cm. = 238 lbs. Scale Load, Tare = 31.2 lbs.

Increment Duration: (See table)

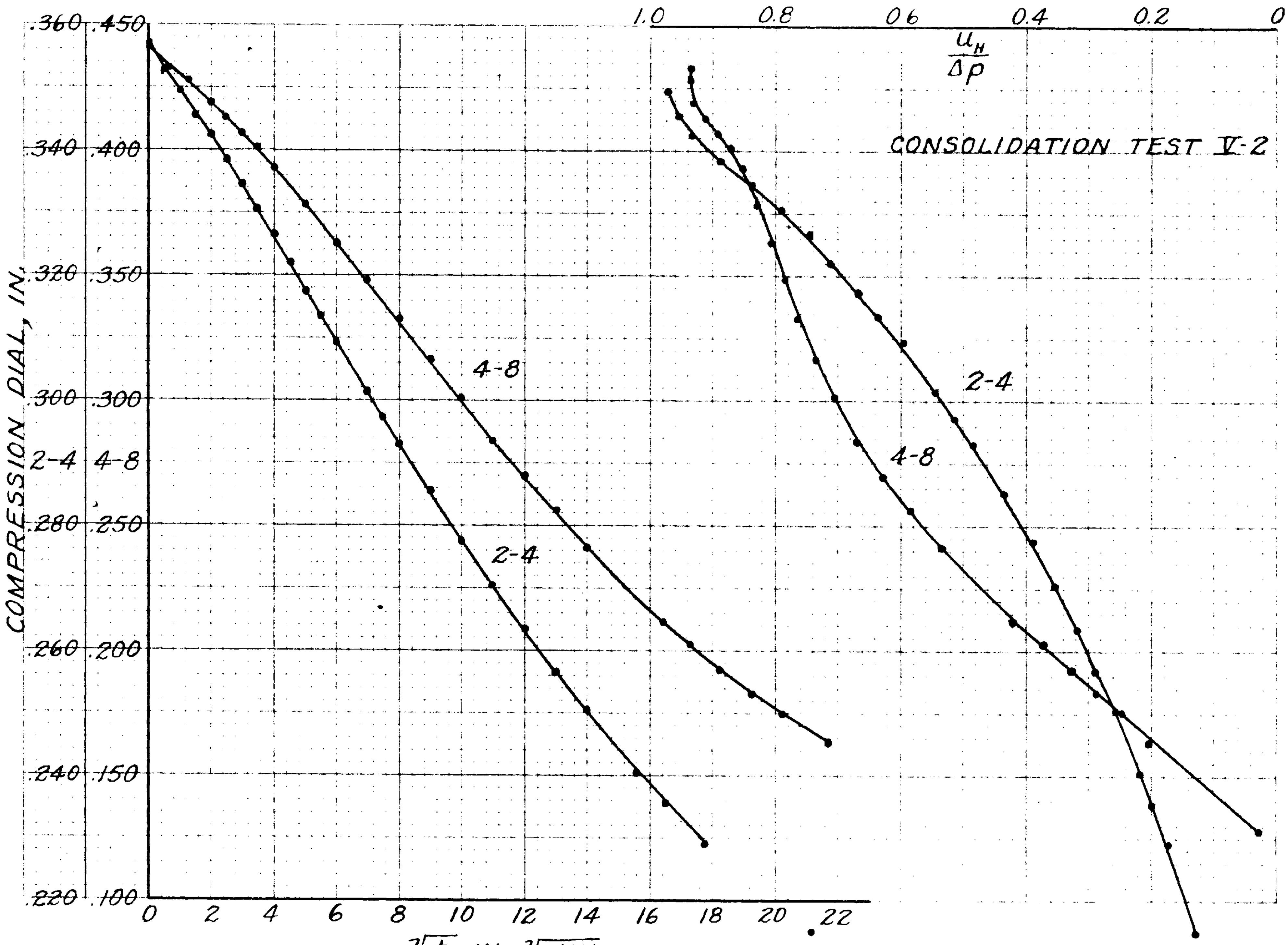
Date	Time	Applied Load kg./sq. cm.	Final Dial in.	Void Ratio
-	-	0.058	.46619	1.272
1/24/51	1:30 P.M.	0.25	.44752	1.262
1/25/51	2:43 P.M.	0.5	.43112	1.254
1/26/51	2:19 P.M.	1	.40338	1.237
1/27/51	2:17 P.M.	2	.35724	1.211
1/28/51	2:21 P.M.	4	[.17750 .4416	1.108
1/29/51	2:31 P.M.	8	.1240	0.926
1/30/51	2:32 P.M.	4	.14245	0.937
2/1/51	2:19 P.M.	5	.13900	0.935
2/2/51	2:10 P.M.	6	.13408	0.932

Remarks:

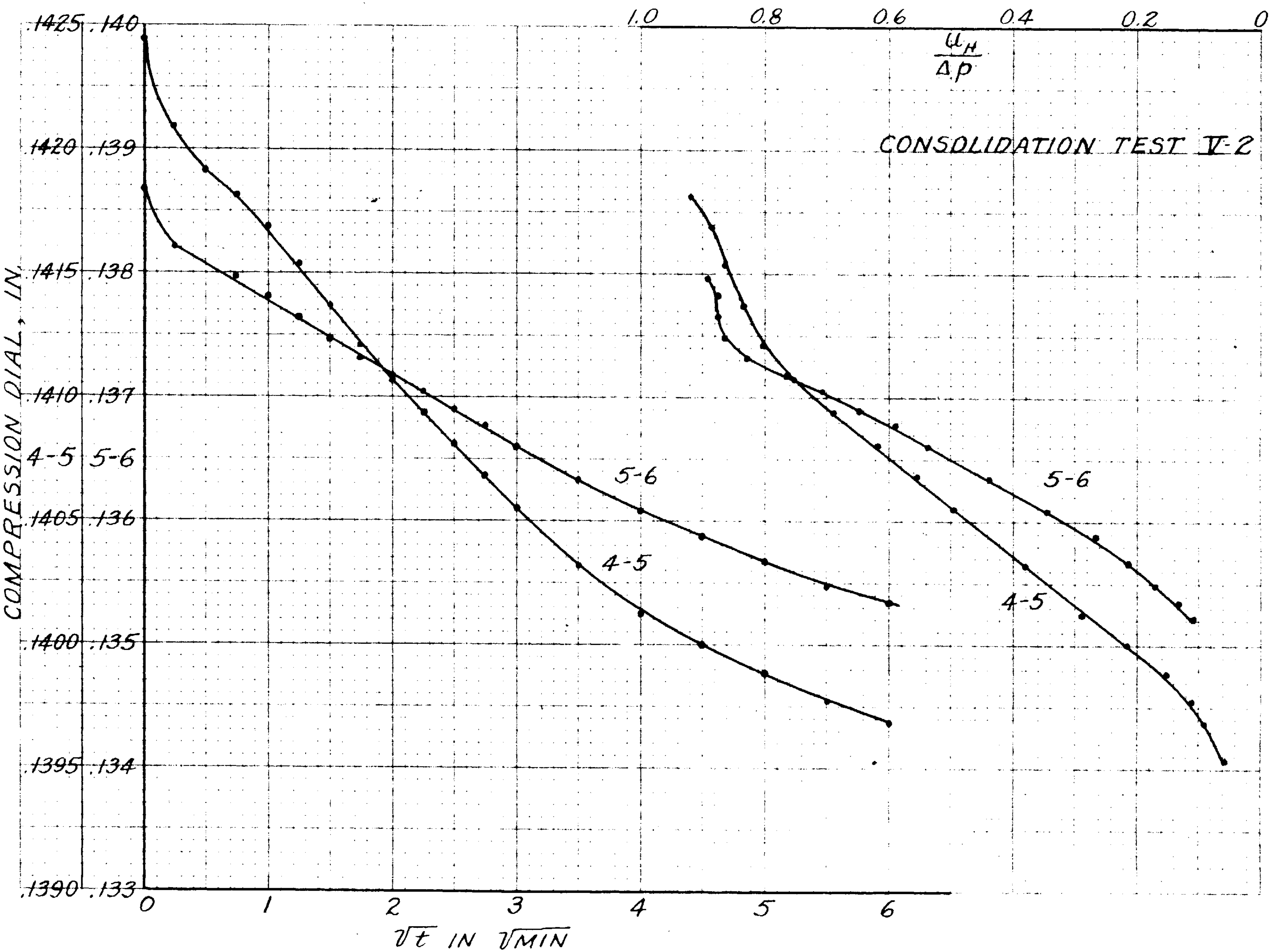
z_3 = (-)2.831 in at Dial = .1240 in.



CONSOLIDATION TEST V-2
COMPRESSION DIAL, IN.
$\frac{1}{2}$-1 1-2
.440 .405
.435 .400
.430 .395
.425 .390
.420 .385
.415 .380
.410 .375
.405 .370
0
1
2
3
4
5
6
7
$\sqrt{t}$ IN $\sqrt{MIN}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
1-2
$\frac{1}{2}$-1
1-2
$\frac{1}{2}$-1



CONSOLIDATION TEST V-2
COMPRESSION DIAL, IN.
2-4 4-8
.360 .450
.340 .400
.320 .350
.300 .300
.280 .250
.260 .200
.240 .150
.220 .100
0 2 4 6 8 10 12 14 16 18 20 22
√t IN √MIN.
1.0 0.8 0.6 0.4 0.2 0
$\frac{u_H}{\Delta p}$
4-8
2-4
2-4
4-8



CONSOLIDATION TEST V-2
COMPRESSION DIAL, IN.
.1425 .140
.1420 .139
.1415 .138
.1410 .137
4-5 5-6
.1405 .136
.1400 .135
.1395 .134
.1390 .133
0 1 2 3 4 5 6
√t IN √MIN
1.0 0.8 0.6 0.4 0.2 0
$\frac{u_H}{\Delta p}$
5-6
4-5

CONSOLIDATION TEST V-2

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$ - 1 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
64	.41700
85	.41640
168	.41552
298	.41494
528	.41432
1134	.41366
1440	.40338

Load Increment 1 - 2 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
49	.37200
64	.37002
124	.36579
440	.36081
1229	.35778
1440	.35724

Load Increment 2 - 4 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
447	.2148
506	.2100
780	.1953
1119	.1840
1440	.1775

Load Increment 4 - 8 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
499	.1597
774	.1400
1160	.1277
1378	.1247
1441	.1240

Load Increment 4 - 5 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
75	.13942
139	.13927
333	.13912
430	.13911
1091	.13900
1440	.13900 =.13867 ?

Load Increment 5 - 6 kg./sq. cm.

Elapsed Time min.	Comp. Dial in.
42	.13521
97	.13477
132	.13461
278	.13420
320	.13408

CONSOLIDATION TEST V-3

Feb. 5, 1951

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78

Average Natural Water Content = 47.2%
(See Figure for Distribution)

Approximate Initial Thickness = 4.00 in.

Weight Soil Solids = 5566 g

Height Soil Solids = 1.712 in.

Apparatus:

Barrel: Height, z_1 = 7.465 in., Diameter = 9.55 in.

Thickness of Rubber Membrane = 0.01 in.

Average Thickness Top Porous Stone and First Cover Plate = 1.281 in.

Loading Unit No. V with Special Leverage System

Loading:

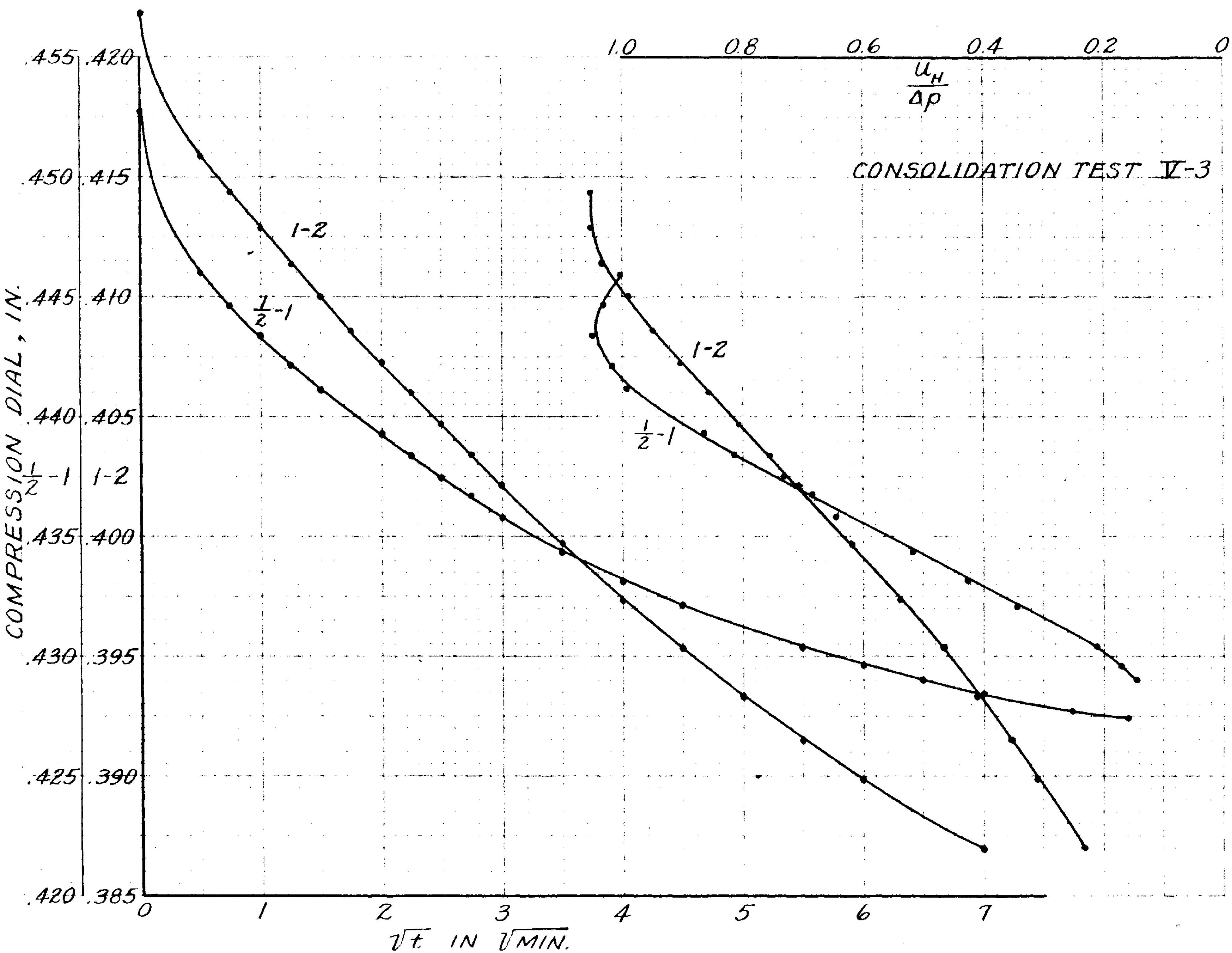
1 kg. per sq.cm. = 238 lbs. Scale Load, Tare = 31.2 lbs.

Increment Duration: (See table)

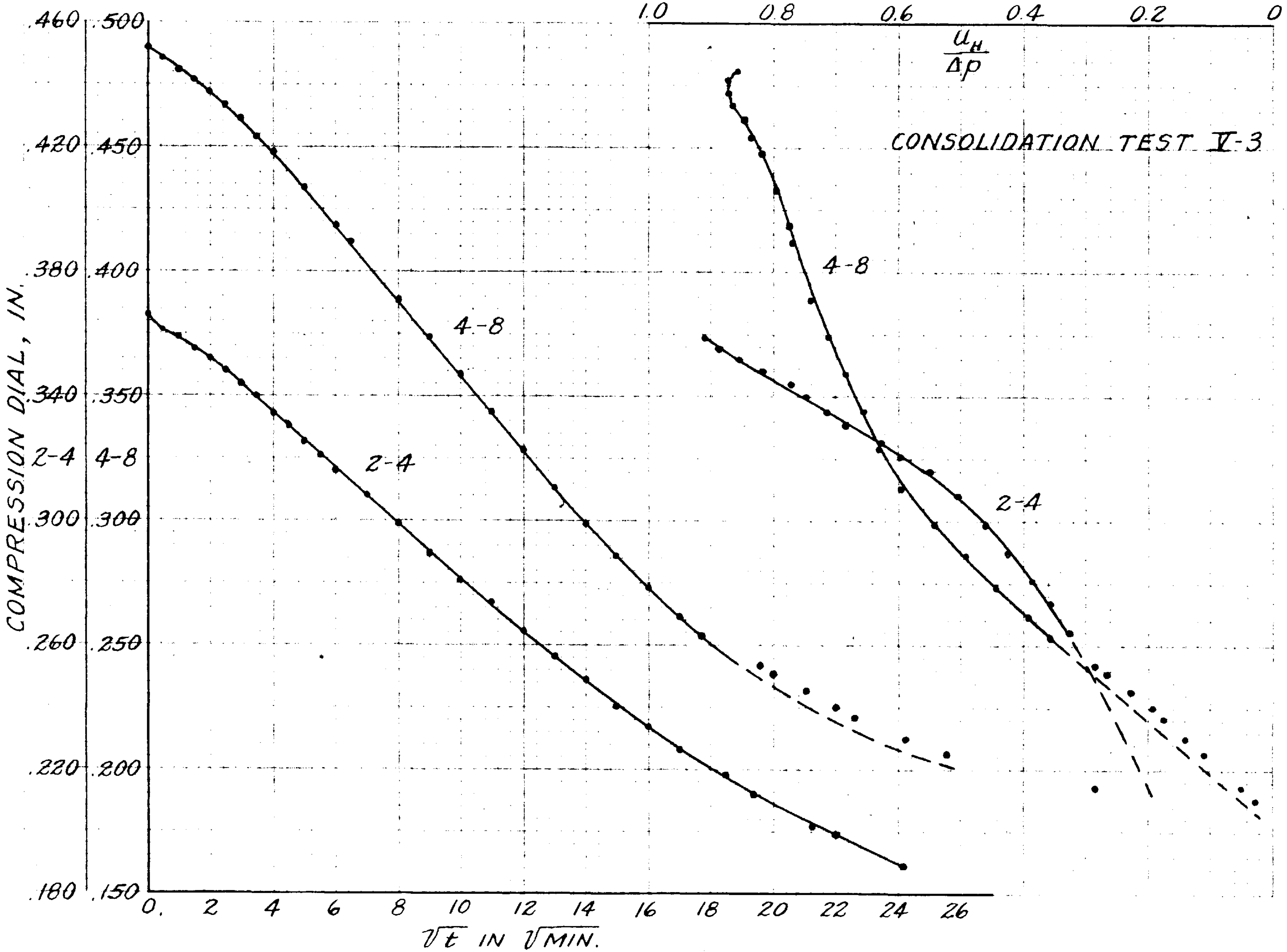
Date	Time	Applied Load kg./sq.cm.	Final Void in.	Void Ratio
2/5/51	10:30 P.M.	0.058	.49026	
2/6/51	12:56 P.M.	0.25	.47018	1.293
2/7/51	1:10 P.M.	0.5	.45269	1.286
2/8/51	1:04 P.M.	1	.42205	1.267
2/9/51	1:08 P.M.	2	.36645	1.235
2/10/51	1:04 P.M.	4	.1538 [=.4902	1.111
2/11/51	1:07 P.M.	8	.1823	0.931
2/12/51	2:42 P.M.	4	.20040	0.941
2/14/51	2:20 P.M.	5	.19709	0.939
2/15/51	2:48 P.M.	6	.19171	0.937

Remarks:

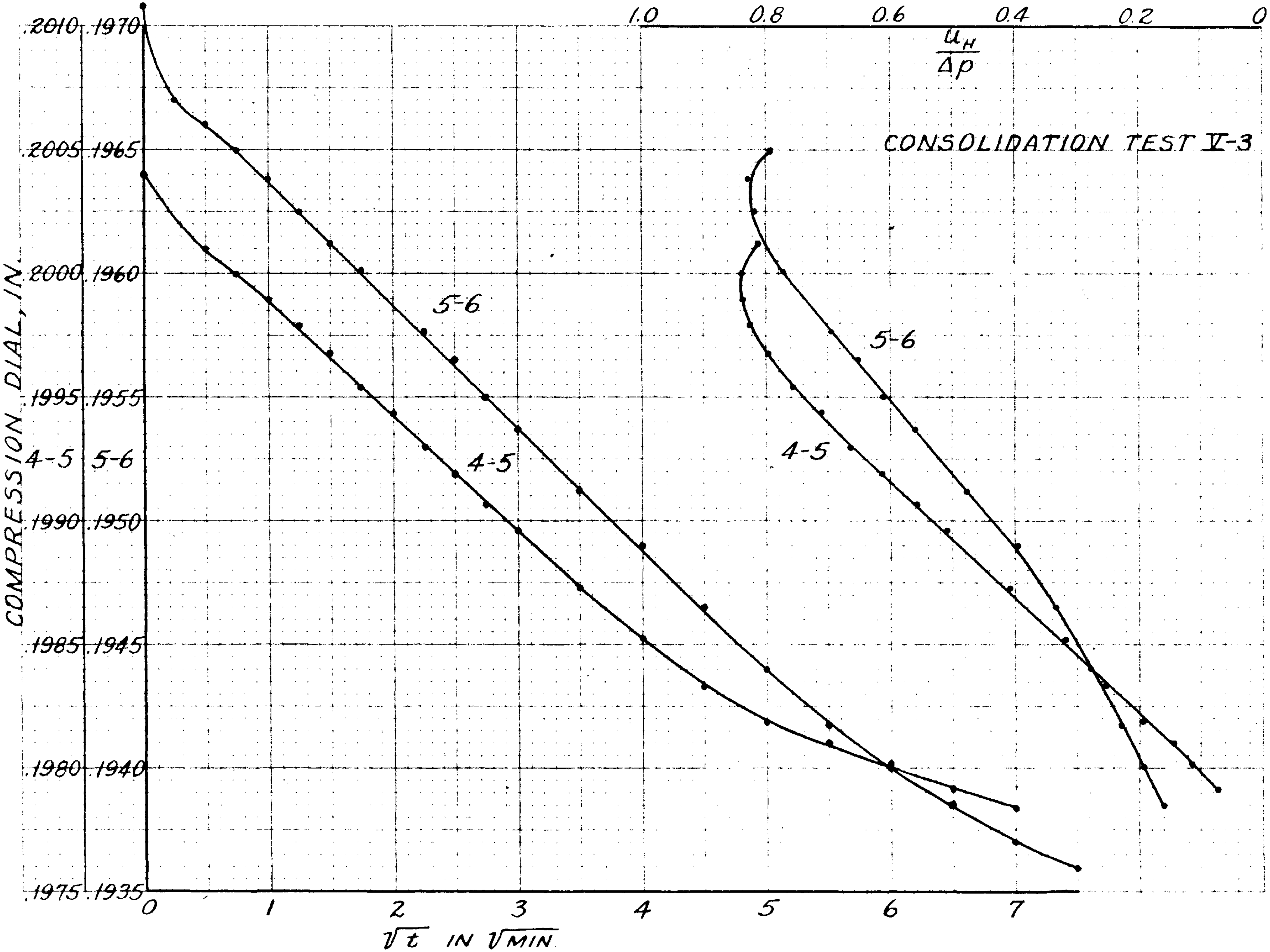
z_3 = (-)2.358 in. at Dial = .36645 in.



CONSOLIDATION TEST V-3
COMPRESSION DIAL, IN.
$\frac{1}{2}$-1
1-2
.455
.450
.445
.440
.435
.430
.425
.420
.420
.415
.410
.405
.400
.395
.390
.385
0
1
2
3
4
5
6
7
$\sqrt{t}$ IN $\sqrt{MIN.}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
1-2
$\frac{1}{2}$-1
1-2
$\frac{1}{2}$-1



CONSOLIDATION TEST V-3
COMPRESSION DIAL, IN.
2-4
4-8
.460
.420
.380
.340
.300
.260
.220
.180
.500
.450
.400
.350
.300
.250
.200
.150
0.
2
4
6
8
10
12
14
16
18
20
22
24
26
$\sqrt{t}$ IN $\sqrt{MIN.}$
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
4-8
2-4
4-8
2-4


CONSOLIDATION TEST V-3
COMPRESSION DIAL, IN.
4-5
5-6
.2010
.1970
.2005
.1965
.2000
.1960
.1995
.1955
.1990
.1950
.1985
.1945
.1980
.1940
.1975
.1935
0
1
2
3
4
5
6
7
√t IN √MIN
1.0
0.8
0.6
0.4
0.2
0
$\frac{u_H}{\Delta p}$
5-6
4-5
5-6
4-5

CONSOLIDATION TEST V-3

Additional Compression Data Used to Plot Log t Curves

Load Increment $\frac{1}{2}$- 1 kg./sq. cm.

Elapsed Time	Comp. Dial
87	.42679
175	.42520
235	.42466
326	.42431
500	.42409 ?
1148	.42232
1431	.42205

Load Increment 1- 2 kg./sq.cm.

Elapsed Time	Comp. Dial
64	.38448
95	.38106
203	.37565
242	.37446
1142	.36760
1227	.36713
1337	.36675
1427	.36645

Load Increment 2- 4 kg./sq.cm.

Elapsed Time	Comp. Dial
733	.1727
1120	.1612
1291	.1571
1440	.1538

Load Increment 4- 8 kg./sq.cm.

Elapsed Time	Comp. Dial
920	.1932
1353	.1841
1535	.1823

Load Increment 4- 5 kg./sq. cm.

Elapsed Time	Comp. Dial
111	.19761
165	.19753
372	.19739
488	.19732
1073	.19718
1246	.19713
1455	.19709

Load Increment 5- 6 kg./sq. cm.

Elapsed Time	Comp. Dial
56	.19359
64	.19348
138	.19297
267	.19263
407	.19242
518	.19231
1042	.19192
1440	.19171

VII SERIES F CONSOLIDATION TEST DATA

CONSOLIDATION TEST F-1

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 35.1% Middle = 37.6%
Bottom = 37.0%
Weight Soil Solids = 395.5 g
Height Soil Solids = .600 in.

Apparatus:

Barrel Height, z_1 = 1.250 in.
Barrel Diameter = 4.289 in.
Thickness of Membrane (if used) = x in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. I

Loading:

1 kg.per sq.cm. = 205.3 lbs.Scale
Tare = 9.5 lbs.

Increment Duration = 12 hrs. (except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Dec. 14, 1950

z_3 = 0.659 in. at Dial = .09114 in.

Total Compression From
0.1-0.5 kg.per sq.cm. = .01388 in.
0.5-1 kg.per sq.cm. = .00916 in.

COMPRESSION DATA

Load Increment	1-2		2-4		4-8		8-7		7-4		4-5		5-6	
Date	Dec.15,'50		Dec.16,'50		Dec.16,'50		Dec.17,'50		Dec.17,'50		Jan.6,'51		Jan.6,'51	
Time	10:19 P.M.		10:21 A.M.		10:15 P.M.		10:22 A.M.		2:42 P.M.		9:27 A.M.		12:00 A.M.	
Initial Void Ratio	1.007		0.979		0.916		0.796		0.796		0.804		0.802	
Final Void Ratio	0.979		0.916		0.796		0.796		0.804		0.802		0.799	
	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
	0	.21696	0	.20040	0	.16349	0	.09114	0	.09121	0	.09540	0	.09412
	:04	.21348	:04	.19800	:04	.16020	:04	.09112	:04	.09210	:04	.09515	:04	.09380
	:15	.21210	:15	.19590	:15	.15720	:15	.09119	:15	.09257	:08.5	.09500	:08.5	.09370
	:34	.21075	:34	.19368	:34	.15360	:34	.09125	:34	.09316	:15	.09490	:15	.09361
	1:00	.20951	1:00	.19175	1:00	.15015	1:00	.09129	1:00	.09357	:23.5	.09480	:23.5	.09357
	1:34	.20838	1:34	.18969	1:34	.14620	1:34	.09132	1:34	.09384	:34	.09469	:34	.09349
	2:15	.20742	2:15	.18770	2:15	.14235	2:15	.09132+	2:15	.09417	:46	.09462	1:00	.09335
	3:04	.20656	3:04	.18586	3:04	.13868	3:04	.09133-	3:04	.09438	1:00	.09454	1:34	.09324
	4:00	.20586	4:00	.18411	4:00	.13492	4:00	.09133-	4:00	.09453	1:34	.09442	2:15	.09316
	5:04	.20543	5:04	.18270	5:04	.13158	5:04	.09133-	5:04	.09457	2:15	.09436	3:04	.09310
	6:15	.20479	6:15	.18152	6:15	.12815	6:15	.09133-	6:15	.09461	3:04	.09432	4:00	.09306
	7:34	.20446	7:34	.18036	7:34	.12498	7:34	.09133-	7:34	.09466	4:00	.09429	5:04	.09303
	9:00	.20422	9:00	.17933	9:00	.12222	9:00	.09133	9:00	.09467	5:04	.09427	6:15	.09301
	12:15	.20379	12:15	.17753	12:15	.11738	12:15	.09133	12:15	.09489	6:15	.09426	201	.09267
	16:00	.20349	16:00	.17606	16:00	.11338	16:00	.09133	16:00	.09502	7:34	-		
	72	.20211	25:00	.17379	20:15	.11037	20:15	.09133	27:00	.09512	9:00	.09424		
	722	.20040	36	.17240	26:00	.10758	25:00	.09133	39	.09515	153	.09412		
			67	.17035	36	.10439	30	.09133	337	.09529				
			104	.16896	49	.10219	36	.09133	523	.09532				
			224	.16659	64	.10045	49	.09133-	1043	.09541				
			586	.16399	83	.09907	64	.09132	2511	.09545				
			714	.16349	92	.09851	74	.09131	7263	.09547				
					727	.09114	97	.09129	28458	.09540				
							198	.09125						
							246	.09121						

CONSOLIDATION TEST F-2

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 38.1% Middle = 34.9%
Bottom = 36.4%
Weight Soil Solids = 392.4 g
Height Soil Solids = 0.601 in.

Apparatus:

Barrel Height, z_1 = 1.260 in.
Barrel Diameter = 4.290 in.
Thickness of Membrane (if used) = 0.01 in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. II

Loading:

1 kg.per sq.cm. = 203.5 lbs.Scale
Tare = 30.0 lbs.

Increment Duration = 12 hrs. (except as shown below)

Initial Applies Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Dec. 14, 1950
Sample in Rubber Membrane

z_3 = 0.657 in. at Dial = .09583 in.
Total Compression From
0.1-0.5 kg.per sq.cm. = .01466 in.
0.5-1 kg.per sq.cm. = .00670 in.

COMPRESSION DATA

Load Increment	1-2		2-4		4-8		8-7		7-4		4-5		5-6	
Date	Dec.15,'50		Dec.16,'50		Dec.16,'50		Dec.17,'50		Dec.17,'50		Jan.6,'51		Jan.6,'51	
Time	10:10 P.M.		10:12 A.M.		10:05 P.M.		10:11 A.M.		2:32 P.M.		9:19 A.M.		11:55 A.M.	
Initial Void Ratio	1.012		0.995		0.933		0.808		0.809		0.817		0.816	
Final Void Ratio	0.995		0.933		0.808		0.809		0.817		0.816		0.814	
	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
	0	.21864	0	.20771	0	.17121	0	.09583	0	.09657	0	.10133	0	.10023
	:04	.21646	:04	.20560	:04	.16880	:04	.09597	:04	.09735	:04	.10115	:04	.10002
	:15	.21543	:15	.20390	:15	.16650	:15	.09610	:15	.09785	:08.5	.10105	:08.5	.09991
	:34	.21443	:34	.20210	:34	.16340	:34	.09625	:34	.09855	:15	.10094	:15	.09982
	1:00	.21345	1:00	.20030	1:00	.16010	1:00	.09635	1:00	.09897	:23.5	.10084	:23.5	.09973
	1:34	.21260	1:34	.19870	1:34	.15671	1:34	.09641	1:34	.09947	:34	.10077	:34	.09965
	2:15	.21188	2:15	.19702	2:15	.15288	2:15	.09645	2:15	.09977	:46	.10071	:46	.09957
	3:04	.21139	3:04	.19542	3:04	.14935	3:04	.09647	3:04	.10003	1:00	.10066	1:00	.09949
	4:00	.21105	4:00	.19385	4:00	.14569	4:00	.09649	4:00	.10025	1:34	.10056	1:34	.09934
	5:04	.21078	5:04	.19248	5:04	.14078	5:04	.09650	5:04	.10040	2:15	.10049	2:15	.09923
	6:15	.21057	6:15	.19115	6:15	.13842	6:15	.09650	6:15	.10049	3:04	.10047	3:04	.09916
	7:34	.21039	7:34	.19013	7:34	.13473	7:34	.09650+	7:34	.10057	4:00	.10044	4:00	.09910
	9:00	.21023	9:00	.18916	9:00	.13128	9:00	.09651-	9:00	.10064	5:04	.10043	6:15	.09902
	12:15	.20999	12:15	.18739	12:15	.12560	12:15	.09651	13:00	.10075	6:15	.10042	9:00	.09897
	16:00	.20980	16:00	.18580	16:00	.12078	16:00	.09651+	17:00	.10077	7:34	.10041	206	.09868
	20:15	.20966	20:15	.18449	20:15	.11731	20:25	.09652	20:15	.10078	12:15	.10039		
	25:00	-	25:00	.18332	25:00	.11431	25:00	.09653	25:00	.10079	156	.10023		
	33:00	.20937	36	.18140	30:15	.11181	30:15	.09654	37	.10084				
	81	.20883	76	.17844	36:00	.11006	36:00	.09654+	49	.10086				
	722	.20771	113	.17687	49	.10740	49	.09655-	347	.10107				
			233	.17439	64	.10542	64	.09655	533	.10108				
			585	.17171	83	.10369	83	.09656	1053	.10121				
			713	.17121	102	.10251	108	.09657	2521	.10131+				
					726	.09583	219	.09657-	7273	.10139				
							257	.09657	28468	.10133				

C O N S O L I D A T I O N   T E S T   F-3

Soil Sample: Undisturbed
Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 31.9% Middle = 32.5%
Bottom = 40.7%
Weight Soil Solids = 424.1 g
Height Soil Solids = 0.645 in.

Apparatus:

Barrel Height, z_1 = 1.250 in.
Barrel Diameter = 4.289 in.
Thickness of Membrane (if used) = x in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. II

Loading:

1 kg.per sq.cm. = 205.3 lbs.Scale
Tare = 29.0 lbs.

Increment Duration = 12 hrs.
(except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 13, 1951

z_3 = 0.689 in. at Dial = .08850 in.

Total Compression From
0.1-0.5 kg.per sq.cm. = .01265 in.
0.5-1 kg.per sq.cm. = .00783 in.

COMPRESSION DATA

Load Increment	1-2		2-4		4-8		8-7		7-4		4-5		5-6	
Date	Jan.14,'51		Jan.15,'51		Jan.15,'51		Jan.16,'51		Jan.16,'51		Jan.20,'51		Jan.20,'51	
Time	10:19 P.M.		10:27 A.M.		10:18 P.M.		10:58 A.M.		2:59 P.M.		11:03 A.M.		1:33 P.M.	
Initial Void Ratio	0.921		0.899		0.834		0.718		0.719		0.725		0.723	
Final Void Radio	0.899		0.834		0.718		0.719		0.725		0.723		0.721	
	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
	0	.21952	0	.20572	0	.16365	0	.08850	0	.08886	0	.09276	0	.09182
	:04	.21700	:04	.20240	:04	.15950	:04	.08867	:04	.08992	:04	.09242	:04	.09143
	:15	.21575	:15	.20015	:15	.15610	:08.5	.08871	:08.5	.09028	:08.5	.09233	:08.5	.09132
	:34	.21445	:34	.19780	:34	.15145	:15	.08875	:15	.09055	:15	.09226	:15	.09122
	1:00	.21330	1:00	.19528	1:00	.14682	:23.5	.08879	:23.5	.09078	:23.5	.09218	:23.5	.09111
	1:34	.21225	1:34	.19268	1:34	.14150	:34	.08882	:34	.09103	:34	.09210	:34	.09102
	2:15	.21145	2:15	.19038	2:15	.13685	:46	.08882	:46	.09128	:46	.09207	:46	.09092
	3:04	.21090	3:04	.18835	3:04	.13203	1:00	.08882	1:00	.09144	1:00	.09203	1:00	.09087
	4:00	.21050	4:00	.18636	4:00	.12790	1:34	.08882	1:34	.09174	1:34	.09200	1:34	.09079
	5:04	.21014	5:04	.18448	5:04	.12390	2:15	.08883	2:15	.09183	2:15	.09199	2:15	.09077
	6:15	.20985	6:15	.18280	6:15	.12048	3:04	.08883	3:04	.09198	6:15	.09194	3:04	.09073
	7:34	.20965	7:34	.18137	7:34	.11750	4:00	.08883	4:00	.09206	7:34	.09193	4:00	.09072
	9:00	.20946	9:00	.18029	9:00	.11458	5:04	.08883	5:04	.09213	9:00	.09193	7:34	.09067
	12:15	.20911	12:15	.17839	13:00	.10928	6:15	.08883	6:15	.09218	30	.09188	12:15	.09063
	19:00	.20871	17:15	.17636	16:00	.10675	7:34	.08883	9:00	.09277	147	.09182	25:00	.09059
	31	.20827	20:15	.17551	21:00	.10388	9:00	.08884	15:00	.09233			35:00	.09058
	52	.20779	25:00	.17441	25:00	.10234	29	.08885	20:00	.09237				
	60	.20761	30:15	.17349	30:15	.10076	56	.08885	30:00	.09243				
	586	.20586	40	.17223	40	.09900	117	.08886	80	.09254				
	720	.20572	60	.17067	50	.09778	146	.08886	120	.09260				
			85	.16955	572	.08915	196	.08886	446	.09275				
			148	.16779	618	.08897	233	.08886	2542	.09276				
			214	.16674	751	.08853			5521	.09276				
			305	.16569	760	.08850								
			371	.16520										
			413	.16492										
			517	.16440										
			702	.16365										

C O N S O L I D A T I O N   T E S T   F-4

Soil Sample: Undisturbed Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 31.1% Middle = 38.6%
Bottom = 31.5%
Weight Soil Solids = 417.5 g
Height Soil Solids = 0.640 in.

Apparatus:

Barrel Height, z_1 = 1.260 in.
Barrel Diameter = 4.290 in.
Thickness of Membrane (if used) = 0.01 in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. I

Loading:

1 kg.per sq.cm. = 203.5 lbs.Scale
Tare = 5.5 lbs.

Increment Duration = 12 hrs. (except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 13, 1951
Sample in Rubber Membrane

z_3 = 0.681 in. at Dial = .09382 in.
Total Compression From
0.1-0.5 kg.per sq.cm. = .01327 in.
0.5-1 kg.per sq.cm. = .00831 in.

COMPRESSION DATA

Load Increment	1-2	2-4	4-8	8-7	7-4	4-5	5-6
Date	Jan.14,'51	Jan.15,'51	Jan.15,'51	Jan.16,'51	Jan.16,'51	Jan.20,'51	Jan.20,'51
Time	10:29 P.M.	10:37 A.M.	10:28 P.M.	11:08 A.M.	3:07 P.M.	11:09 A.M.	1:38 P.M.
Initial Void Ratio	0.930	0.906	0.846	0.735(-)	0.735(+)	0.741	0.740
Final Void Ratio	0.906	0.846	0.735(-)	0.735(+)	0.741	0.740	0.738

1-2 Elapsed Time Min:Sec.	1-2 Comp. Dial (in.)	2-4 Elapsed Time Min:Sec.	2-4 Comp. Dial (in.)	4-8 Elapsed Time Min:Sec.	4-8 Comp. Dial (in.)	8-7 Elapsed Time Min:Sec.	8-7 Comp. Dial (in.)	7-4 Elapsed Time Min:Sec.	7-4 Comp. Dial (in.)	4-5 Elapsed Time Min:Sec.	4-5 Comp. Dial (in.)	5-6 Elapsed Time Min:Sec.	5-6 Comp. Dial (in.)
0	.21842	0	.20355	0	.16500	0	.09382	0	.09410	0	.09847	0	.09749
:04	.21550	:04	.20130	:04	.16220	:04	.09392	:04	.09516	:04	.09828	:04	.09717
:15	.21400	:15	.19940	:15	.15920	:08.5	.09396	:08.5	.09547	:08.5	.09815	:08.5	.09703
:34	.21275	:34	-	:34	.15530	:15	.09403	:15	.09570	:15	.09805	:15	.09693
1:00	.21178	1:00	.19480	1:00	.15140	:23.5	.09409	:23.5	.09613	:23.5	.09796	:23.5	.09680
1:34	.21090	1:34	.19278	1:34	.14680	:34	.09412	:34	.09645	:34	.09788	:34	.09669
2:15	-	2:15	.19069	2:15	.14240	:46	.09414	:46	-	:46	.09779	:46	.09660
3:04	.21955	3:04	.18869	3:04	.13810	1:00	.09417	1:00	.09692	1:00	.09773	1:00	.09653
4:00	.21909	4:00	.18690	4:00	.13380	1:34	.09419	1:34	.09728	1:34	.09767	1:34	.09643
5:04	.21862	5:04	.18522	5:04	.13000	2:15	.09421	2:15	.09755	2:15	.09761	2:15	.09638
6:15	.21829	6:15	.18375	6:15	.12640	3:04	.09422	3:04	-	3:04	.09760	3:04	.09633
7:34	.21798	7:34	.18263	7:34	.12322	4:00	.09422	4:00	.09765	4:00	.09759	4:00	.09631
9:00	.21775	9:00	.18150	9:30	.11963	5:04	-	5:04	.09779	6:15	.09758	8:00	.09624
21	.21670	12:15	.17969	12:15	.11587	6:15	-	6:15	.09786	9:00	.09757	20:00	.09616
42	.21593	16:00	.17818	16:00	.11255	19:00	.09422	9:00	.09795	25:00	.09752	30:00	.09612
55	.21563	20:15	.17690	20:15	.11000	46	.09419	20:00	.09810	142	.09749		
576	.21375	25:00	.17588	25:00	.10810	107	.09417	72	.09825				
720	.20355	30:15	.17491	30:15	.10658	136	.09415	112	.09829				
		52	.17280	40	.10450	186	.09413	438	.09839				
		75	.17152	562	.09452	223	.09410	2534	.09846				
		138	.16949	608	.09433			5513	.09847				
		204	.16834	741	.09382								
		295	.16717										
		361	.16665										
		403	.16637										
		507	.16580										
		696	.16500										

CONSOLIDATION TEST F-5

Soil Sample: Undisturbed
Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 41.0% Middle = 37.3%
Bottom = 35.0%
Weight Soil Solids = 397.1 g
Height Soil Solids = 0.603 in.

Apparatus:

Barrel Height, z_1 = 1.250 in.
Barrel Diameter = 4.289 in.
Thickness of Membrane (if used) = x in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. II

Loading:

1 kg.per sq.cm. = 205.3 lbs.Scale
Tare = 29.0 lbs.

Increment Duration = 24 hrs.
(except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 20, 1951

z_3 = 0.660 in. at Dial = .06100 in.

Total Compression From
0.1-0.5 kg.per sq.cm. = .01118 in.
0.5-1 kg.per sq.cm. = .00682 in.

COMPRESSION DATA

Load Increment	1-2		2-4		4-8		8-7		7-4		4-2		2-1		1-2		2-3	
Date	Jan.23,'51		Jan.24,'51		Jan.25,'51		Jan.26,'51		Jan.26,'51		Jan.26,'51		Jan.26,'51		Jan.26,'51		Jan.27,'51	
Time	12:44 P.M.		12:44 P.M.		12:43 P.M.		12:53 P.M.		1:53 P.M.		4:53 P.M.		8:53 P.M.		9:53 P.M.		9:53 P.M.	
Initial Void Ratio	1.056		1.032		0.958		0.789		0.790		0.798		0.813		0.831		0.821	
Final Void Ratio	1.032		0.958		0.789		0.790		0.798		0.813		0.831		0.821		0.813	
	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
	0	.22200	0	.20742	0	.16217	0	.06100	0	.06144	0	.06600	0	.07537	0	.08582	0	.08016
	:04	.21960	:04	.20390	:04	.15765	:04	.06119	:04	.06251	:04	.06770	:04	.07663	:04	.08482	:04	.07940
	:15	.21845	:15	.20165	:15	.15370	:08.5	.06121	:08.5	.06274	:08.5	.06796	:08.5	.07682	:08.5	.08462	:08.5	.07923
	:34	.21723	:34	.19940	:34	.14920	:15	.06125	:15	.06296	:15	.06836	:15	.07709	:15	.08438	:15	.07905
	1:00	.21613	1:00	.19720	1:00	.14260	:23.5	.06129	:23.5	.06332	:23.5	.06870	:23.5	.07735	:23.5	.08412	:23.5	.07889
	1:34	.21512	1:34	.19485	1:34	.13660	:34	.06132	:34	.06361	:34	.06902	:34	.07765	:34	.08385	:34	.07872
	2:15	.21433	2:15	.19270	2:15	.13090	:46	.06135	:46	.06380	:46	.06935	:46	.07785	:46	-	:46	.07853
	3:04	.21369	3:04	.19068	3:04	.12575	1:00	.06136	1:00	.06409	1:00	.06968	1:00	.07819	1:00	.08339	1:00	.07838
	4:00	.21311	4:15	.18857	4:00	.12070	1:34	.06139	1:34	.06456	1:34	.07030	1:34	.07875	1:34	.08288	1:34	.07798
	5:04	.21270	5:04	.18730	5:04	.11600	2:15	.06139	2:15	.06479	2:15	.07083	2:15	.07928	2:15	.08247	2:15	.07769
	6:15	.21234	6:15	.18572	6:15	.11100	3:04	.06141	3:04	.06503	3:04	.07157	3:04	.07976	3:04	.08207	3:04	.07739
	7:34	.21203	7:34	.18430	7:34	.10700	4:00	.06141	4:00	.06523	4:00	.07206	4:00	.07031	4:00	.08173	4:00	.07711
	9:00	.21178	9:00	.18294	9:00	.10280	9:00	.06142	5:04	.06537	5:04	.07257	5:04	.08078	5:04	.08144	5:04	.07691
	12:15	.21142	12:15	.18087	12:15	.09567	20:00	.06142	7:34	.06552	6:15	-	6:15	.08129	6:15	.08121	6:15	.07673
	16:00	.21118	16:00	.17928	16:00	.08965	60	.06144	11:00	.06562	9:00	.07352	7:34	.08177	7:34	.08102	11:00	.07639
	20:15	.21092	20:00	.17802	20:00	.08530			16:00	.06570	17:00	.07426	9:00	.08231	15	.08067	22:00	.07609
	25:00	.21074	37	.17461	30	.07915			32	.06579	143	.07525	12:15	.08303	32	.08051	37	.07593
	40	.21036	71	.17152	54	.07329			72	.06585	240	.07537	16:00	.08374	60	.08041	69	.07578
	58	.21003	110	.16985	96	.06959			180	.06600			20:15	.08427	77	.08038	93	.07572
	81	.20978	240	.16724	143	.06774							25:00	.08470	680	.08021	773	.07531
	116	.20948	485	.16501	229	.06579							40	.08546	907	.08019		
	200	.20898	1192	.16261	556	.06287							60	.08582	1440	.08016		
	256	.20879	1431	.16217	1224	.06133												
	381	.20843			1440	.06100												
	642	.20802																
	1186	.20747																
	1436	.20742																

CONSOLIDATION TEST F-6

Soil Sample: Undisturbed
Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 39.5% Middle = 37.2%
Bottom = 35.2%
Weight Soil Solids = 393.1 g
Height Soil Solids = 0.602 in.

Apparatus:

Barrel Height, z_1 = 1.260 in.
Barrel Diameter = 4.290 in.
Thickness of Membrane (if used) = 0.01 in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. I

Loading:

1 kg.per sq.cm. = 203.5 lbs.Scale
Tare = 5.5 lbs.

Increment Duration = 24 hrs.
(except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 20, 1951
Sample in Rubber Membrane

z_3 = 0.657 in. at Dial = .05937 in.
Total Compression From
0.1-0.5 kg.per sq.cm. = .01546 in.
0.1-1 kg.per sq.cm. = .00822 in.

COMPRESSION DATA

Load Increment	1-2	2-4	4-8	8-7	7-4	4-2	2-1	1-2	2-3
Date	Jan.23,'51	Jan.24,'51	Jan.25,'51	Jan.26,'51	Jan.26,'51	Jan.26,'51	Jan.26,'51	Jan.26,'51	Jan.27,'51
Time	12:50 P.M.	12:49 P.M.	12:53 P.M.	12:59 P.M.	1:59 P.M.	4:59 P.M.	8:59 P.M.	9:59 P.M.	9:59 P.M.
Initial Void Ratio	1.063	1.037	0.943	0.805	0.806	0.814	0.827	0.844	0.835
Final Void Ratio	1.037	0.943	0.805	0.806	0.814	0.827	0.844	0.835	0.824

Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
0	.21632	0	.19987	0	.14305	0	.05937	0	.05999	0	.06482	0	.07324	0	.08317	0	.07792
:04	.21385	:04	.19700	:04	.14020	:04	.05952	:04	.06110	:04	.06581	:04	.07381	:04	.08272	:04	.07756
:15	.21272	:15	.19510	:15	.13750	:08.5	.05954	:08.5	.06140	:08.5	.06620	:08.5	.07405	:08.5	.08250	:08.5	.07740
:34	.21158	:34	.19302	:34	.13380	:15	.05959	:15	.06160	:15	.06655	:15	.07437	:15	.08228	:15	.07721
1:00	.21045	1:00	.19075	1:00	.13020	:23.5	.05967	:23.5	.06192	:23.5	.06681	:23.5	.07462	:23.5	.08203	:23.5	.07700
1:34	.20940	1:34	.18840	1:34	.12610	:34	.05972	:34	.06229	:34	.06719	:34	.07495	:34	.08182	:34	.07681
2:15	-	2:15	.18602	2:15	.12190	:46	.05974	:46	.06257	:46	.06752	:46	.07527	:46	.08158	:46	.07665
3:04	.20776	3:04	.18370	3:04	.11770	1:00	.05979	1:00	.06278	1:00	.06776	1:00	.07559	1:00	.08140	1:00	.07649
4:00	.20708	4:00	.18170	4:00	.11350	1:34	.05988	1:34	.06323	1:34	.06854	1:34	.07611	1:34	.08091	1:34	.07610
5:30	.20643	5:04	.17965	5:04	.10950	2:15	.05991	2:15	.06357	2:15	.06914	2:15	.07663	2:15	.08051	2:15	.07575
6:15	.20617	6:15	.17765	6:15	.10560	3:04	.05993	3:04	.06376	3:04	.06964	3:04	.07727	3:04	.08011	3:04	.07549
7:34	.20575	7:34	.17575	7:34	.10180	4:00	.05995	4:00	.06397	4:00	.07016	4:00	.07777	4:00	.07970	4:00	.07520
9:00	.20546	9:00	.17400	9:00	.09830	9:00	.05997	5:04	.06412	5:04	.07057	5:04	.07835	5:04	.07945	5:04	.07494
12:15	.20490	12:15	.17118	12:15	.09205	24:00	.05998	7:34	.06436	7:34	.07130	6:15	-	6:15	.07921	6:15	.07475
16:00	.20453	16:00	.16865	16:00	.08710	60	.05999	10:00	.06444	12:15	.07189	7:34	.07922	7:34	.07900	7:34	.07459
20:15	-	23:00	.16512	20:15	.08288			26	.06458	137	.07312	9:00	.07962	10:00	.07875	15:00	.07418
23:00	.20404	31	.16262	30	.07730			65	.06473	240	.07324	12:15	.08051	23:00	.07842	31	.07391
33	.20360	66	.15703	45	.07300			150	.06482			16:00	.08114	46	.07819	62	.07374
52	.20315	105	.15390	86	.06852							20:00	.08159	70	.07812	86	.07369
75	.20272	235	.14997	133	.06629							33	.08252	674	.07797	767	.07330
110	.20234	480	.14678	219	.06411							60	.08317	901	.07794		
194	.20175	1187	.14359	546	.06153									1440	.07792		
250	.20147	1436	.14305	1214	.05968												
375	.20107			1440	.05937												
636	.20059																
1180	.20000																
1430	.19987																

C O N S O L I D A T I O N   T E S T   F-7

Soil Sample: Undisturbed
Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 44.6% Middle = 43.1%
Bottom = 41.8%
Weight Soil Solids = 367.0 g
Height Soil Solids = 0.557 in.

Apparatus:

Barrel Height, z_1 = 1.250 in.
Barrel Diameter = 4.289 in.
Thickness of Membrane (if used) = x in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. I

Loading:

1 kg.per sq.cm. = 205.3 lbs.Scale
Tare = 7.6 lbs.

Increment Duration = 24 hrs.
(except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 28, 1951

z_3 = 0.722 in. at Dial = .13173 in.

Total Compression From
0.1-0.5 kg.per sq.cm. = .01268 in.
0.5-1 kg.per sq.cm. = .00682 in.

COMPRESSION DATA

Load Increment	1-2	2-4	4-8	8-7	7-4	4-2	2-1	1-2	2-3
Date	Jan.30,'51	Jan.31,'51	Feb.1,'51	Feb.2,'51	Feb.2,'51	Feb.2,'51	Feb.2,'51	Mar.2,'51	Mar.3,'51
Time	2:59 P.M.	3:00 P.M.	3:00 P.M.	2:54 P.M.	3:54 P.M.	6:54 P.M.	10:54 P.M.	8:18 P.M.	7:37 P.M.
Initial Void Ratio	1.208	1.180	1.048	0.872	0.872	0.883	0.898	0.921	0.914
Final Void Ratio	1.180	1.048	0.872	0.872	0.883	0.898	0.921	0.914	0.902

Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
0	.22050	0	.20507	0	.13173	0	.03368	0	.03391	0	.03921	0	.04860	0	.06227	0	.05697
:04	.21810	:04	.20172	:15	.12460	:04	.03371	:04	.03495	:04	.04020	:15	.04975	:04	.06172	:04	.05639
:08.5	.21740	:15	.19930	1:00	.11745	:08.5	.03372	:08.5	.03527	:08.5	.04048	1:00	.05078	:08.5	.06155	:15	.05598
:15	.21687	:34	.19672	2:15	.10955	:15	.03374	:15	-	:15	.04070	2:15	.05179	:15	.06140	:34	.05556
:23.5	.21628	1:00	.19435	4:00	.10155	:23.5	.03377	:23.5	.03579	:23.5	.04107	4:00	.05279	:23.5	.06119	1:00	.05510
:34	.21570	1:34	.19195	6:15	.09358	:34	.03378	:34	.03615	:34	.04142	6:15	.05381	:34	.06096	1:34	.05472
:46	.21518	2:15	.18945	9:00	.08645	:46	.03380	:46	.03642	:46	.04167	9:00	.05481	:46	.06075	2:15	.05432
1:00	.21459	3:04	.18705	12:15	.07965	1:00	.03382	1:00	.03657	1:00	.04205	12:15	.05572	1:00	.06057	3:04	.05392
1:34	.21383	4:00	.18455	16:00	.07330	1:34	.03386	1:34	.03712	1:34	.04259	16:00	.05658	1:34	.06018	4:00	.05360
2:15	.21322	5:04	.18237	21:00	.06737	2:15	.03388	2:15	.03749	2:15	.04322	20:15	.05729	2:15	.05982	5:04	.05333
3:04	.21271	6:15	.18005	25:00	.06363	3:04	.03389	3:04	.03770	3:04	.04371	25:00	.05775	3:04	.05953	6:15	.05305
4:00	.21223	7:34	.17778	30:15	.06010	4:00	.03389+	4:00	.03800	4:00	.04428	30:15	.05829	4:00	.05927	7:34	.05281
5:04	.21183	9:00	.17553	51	.05075	6:15	.03390	5:04	.03822	5:04	.04472	36	.05858	5:04	.05901	9:00	.05260
6:15	.21151	12:15	.17157	92	.04583	9:00	.03391	6:15	.03836	6:15	.04525	673	.06142	6:15	.05879	12:15	.05231
9:00	.21103	16:00	.16805	293	.03897	16:00	.03391	7:34	.03846	7:34	.04561	908	.06153	7:34	.05864	18:00	.05200
15:00	.21034	20:15	.16468	385	.03775	39	.03391	9:00	.03854	9:00	.04600	2191	.06172	9:00	.05851	37:00	.05157
30	.20951	25:00	.16210	1050	.03446			16:00	.03868	10:34	.04628	10836	.06204	19:00	.05801	80	.05096
84	.20839	40	.15638	1440	.03368			23:00	.03881	12:15	-	26496	.06219	80	.05761	825	.05033
556	.20614	93	.14787					40	.03897	14:04	.04669	40146	.06227	730	.05713	1102	.05024
1123	.20537	142	.14439					89	.03912	16:00	.04692			1092	.05703	1440	.05017
1440	.20507	274	.14001					180	.03921	27	.04761			1392	.05697		
		460	.13685							52	.04806						
		1123	.13267							90	.04833						
		1440	.13173							141	.04851						
										182	.04859						
										240	.04860						

C O N S O L I D A T I O N   T E S T   F-8

Soil Sample: Undisturbed
Boston Blue Clay

Specific Gravity = 2.78
Natural Water Content:
Top = 42.4% Middle = 41.4%
Bottom = 43.3%
Weight Soil Solids = 366.7 g
Height Soil Solids = 0.561 in.

Apparatus:

Barrel Height, z_1 = 1.260 in.
Barrel Diameter = 4.290 in.
Thickness of Membrane (if used) = 0.01 in.
Thickness Upper Stone and Brass Cover, z_2 = 0.831 in.
Loading Unit No. II

Loading:

1 kg.per sq.cm. = 203.5 lbs.Scale
Tare = 27.5 lbs.

Increment Duration = 24 hrs.
(except as shown below)

Initial Applied Load = 0.1 kg. per sq.cm.

Remarks:

Test Begun Jan. 28, 1951

z_3 = 0.724 in. at Dial = .13019 in.

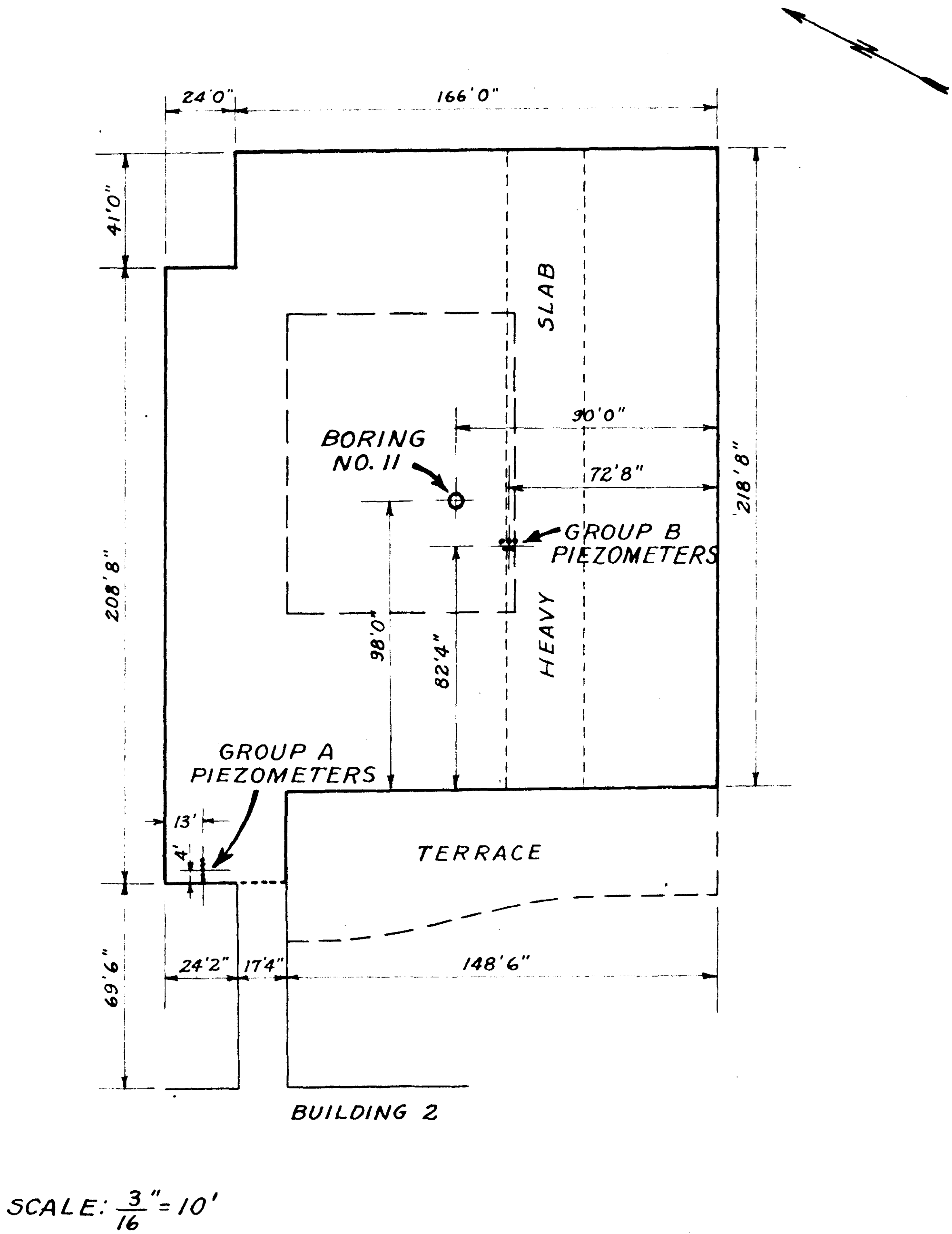
Total Compression From
0.1-0.5 kg.per sq.cm. = .01478 in.
0.5-1 kg.per sq.cm. = .00805 in.

COMPRESSION DATA

Load Increment	1-2		2-4		4-8		8-7		7-4		4-2		2-1		1-2		2-3	
Date	Jan.30,'51		Jan.31,'51		Feb.1,'51		Feb.2,'51		Feb.2,'51		Feb.2,'51		Feb.2,'51		Mar.2,'51		Mar.3,'51	
Time	2:49 P.M.		2:50 P.M.		2:56 P.M.		2:49 P.M.		3:49 P.M.		6:49 P.M.		10:49 P.M.		8:09 P.M.		7:32 P.M.	
Initial Void Ratio	1.211		1.181		1.056		0.882		0.883		0.894		0.911		0.934		0.927	
Final Void Ratio	1.181		1.056		0.882		0.883		0.894		0.911		0.934		0.927		0.915	
	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)	Elapsed Time Min:Sec.	Comp. Dial (in.)
	0	.21717	0	.19998	0	.13019	0	.03280	0	.03362	0	.03917	0	.04879	0	.06265	0	.05749
	:04	.21470	:04	.19715	:15	.12450	:04	.03291	:04	-	:04	.04000	:15	.09987	:04	.06222	:04	..5700
	:08.5	.21415	:15	.19518	1:00	.11875	:08.5	.03299	:08.5	.03472	:08.5	.04030	1:00	.05083	:08.5	.06207	:15	.05666
	:15	.21359	:34	.19303	2:15	.11135	:15	.03307	:15	.03498	:15	.04055	2:15	.05178	:15	.06190	:34	.05628
	:23.5	.21300	1:00	.19085	4:00	.10365	:23.5	.03313	:23.5	.03532	:23.5	.04076	4:00	.05278	:23.5	.06178	1:00	.05585
	:34	.21244	1:34	.18880	6:15	.09545	:34	.03321	:34	.03562	:34	.04107	6:15	.05382	:34	.06162	1:34	.05550
	:46	.21188	2:15	.18660	9:00	.08740	:46	.03327	:46	.03588	:46	.04137	9:00	.05485	:46	.06147	2:15	.05507
	1:00	.21143	3:04	.18425	13:15	.07800	1:00	.03333	1:00	.03619	1:00	.04166	12:15	.05581	1:00	.06131	3:04	.05469
	1:34	.21068	4:00	.18185	16:00	.07325	1:34	.03341	1:34	.03672	1:34	.04218	16:00	.05670	1:34	.06096	4:00	.05432
	2:15	.20992	5:04	.17982	20:15	.06752	2:15	.03347	2:15	.03715	2:15	.04271	20:15	.05742	2:15	.06065	5:04	.05397
	3:04	.20931	6:15	.17775	25:00	.06270	3:04	.03350	3:04	.03752	3:04	.04335	25:00	.05802	3:04	.06037	6:15	.05367
	4:00	.20872	7:34	.17560	30:15	.05915	4:00	.03352	4:00	.03778	4:00	.04382	30:15	.05858	4:00	.06007	7:34	.05341
	5:04	.20821	9:00	.17345	45	.05335	6:15	.03355	5:04	.03800	5:04	.04447	39	.05912	5:04	.05981	9:00	.05317
	6:15	.20777	13:15	.16862	95	.04452	11	.03357	7:34	.03837	6:15	.04492	678	.06161	6:15	.05961	12:15	.05275
	9:00	.20700	16:00	.16613	297	.03819	20	.03359	9:00	.03848	7:34	.04543	913	.06172	7:34	.05943	20:00	.05232
	15:00	.20603	20:15	.16288	389	.03699	43	.03361	16:00	.03872	9:00	.04575	2196	.06186	9:00	.05926	40	.05188
	30:00	.20489	25:00	.16017	1054	.03365	60	.03362	28:00	.03881	10:34	.04616	10841	.06235	12:15	.05898	81	.05162
	94	.20330	30:15	.15790	1440	.03280			45:00	.03891	12:15	.04651	26501	.06260	16:00	.05881	830	.05084
	566	.20092	36:00	.15575					94	.03910	14:04	.04673	40151	.06265	29:00	.05852	1097	.05076
	1133	.20023	49	.15213					180	.03917	16:00	.04691			89	.05818	1440	.05069
	1440	.19998	103	.14531							32	.04778			739	.05768		
			151	.14216							57	.04826			1101	.05756		
			284	.13818							95	.04851			1401	.05749		
			470	.13521							145	.04869						
			1133	.13090							187	.04877						
			1440	.13019							240	.04879						

LIST OF FIGURES

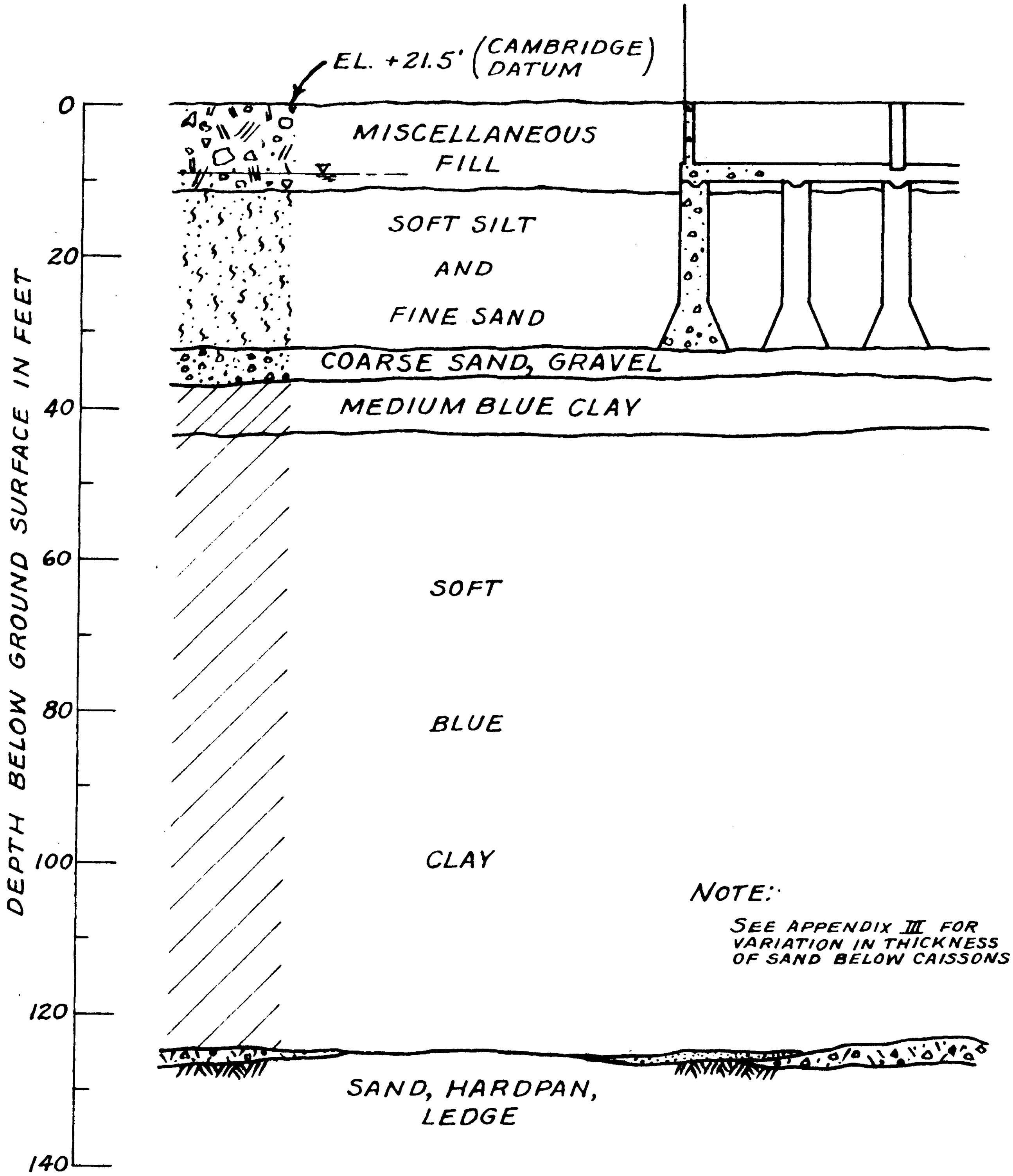
47. Primary Compression Ratio Study



FOUNDATION PLAN
SHOWING BORING NO. 11 AND
PIEZOMETER LOCATIONS

Slide 2

FIG. 1



GEOLOGIC SECTION OF FOUNDATION SOIL

FIG. 2

FIG. 3

SUMMARY OF SOIL PROPERTIES PLOTTED AGAINST DEPTH
NEW LIBRARY BUILDING
BORING NO. 11

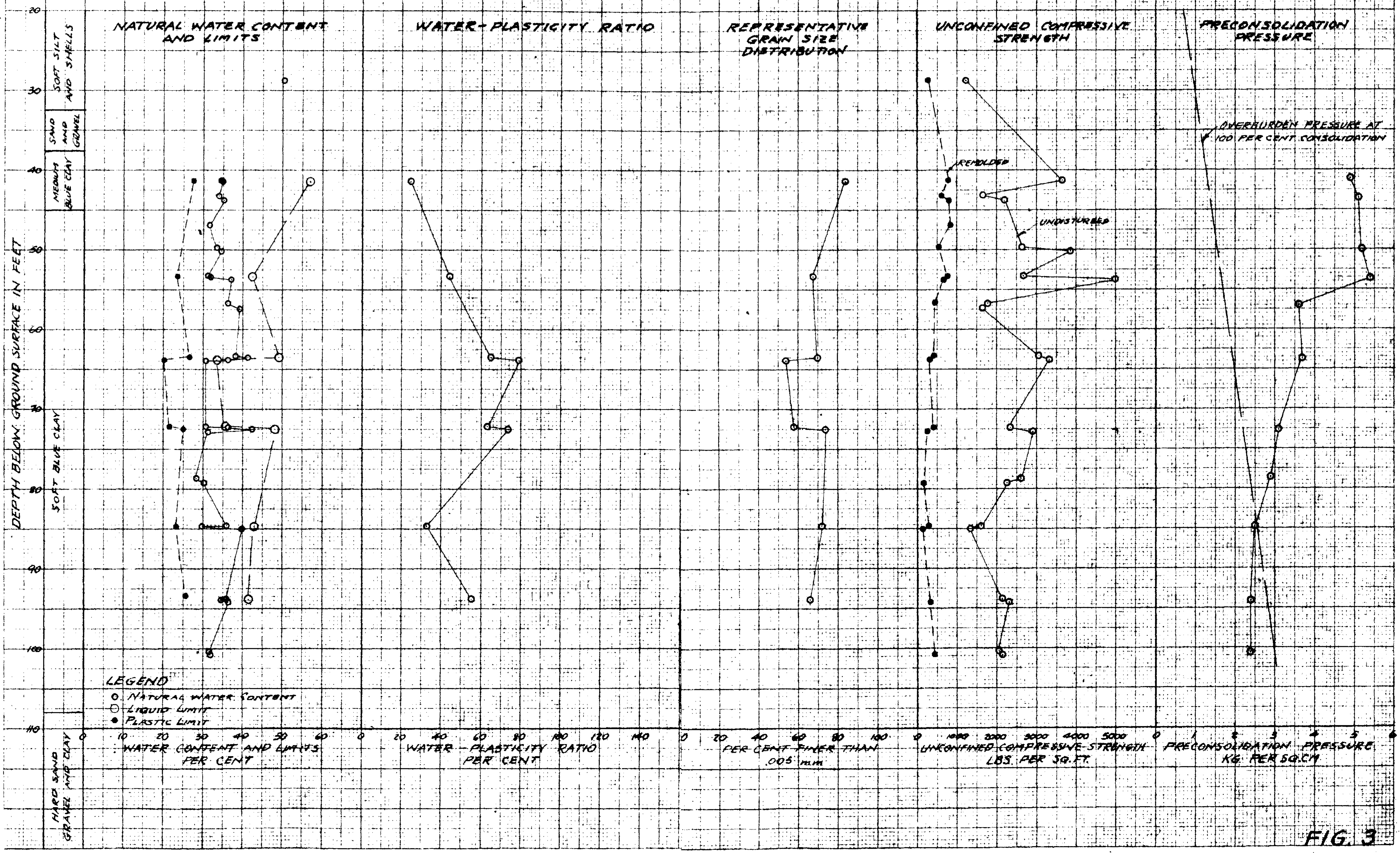


FIG. 3

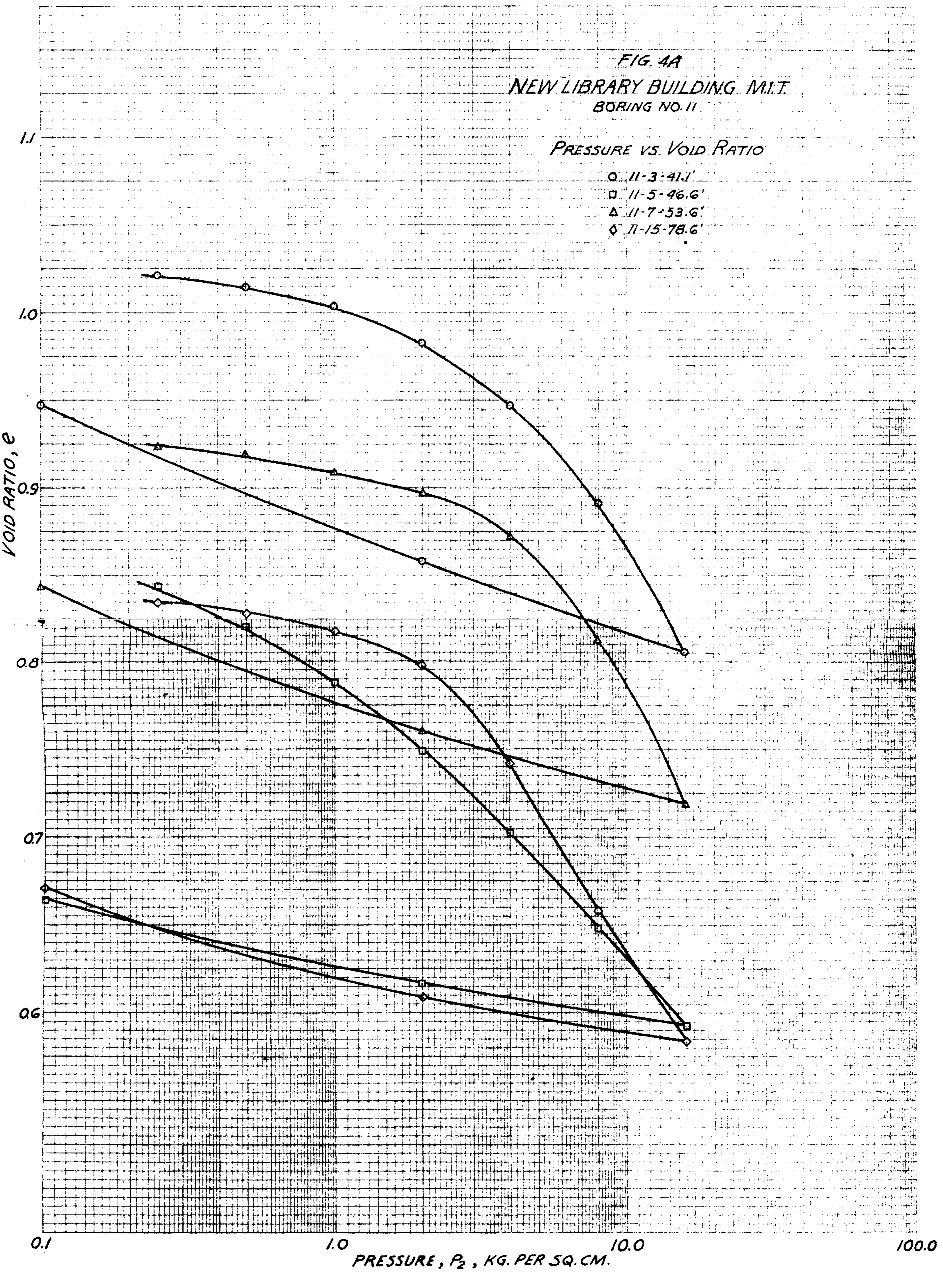


FIG. 4A
NEW LIBRARY BUILDING M.I.T.
BORING NO. 11
PRESSURE VS. VOID RATIO
11-3-41.1'
11-5-46.6'
11-7-53.6'
11-15-78.6'
VOID RATIO, e
1.1
1.0
0.9
0.8
0.7
0.6
0.1
1.0
10.0
100.0
PRESSURE, P_2, KG. PER SQ. CM.

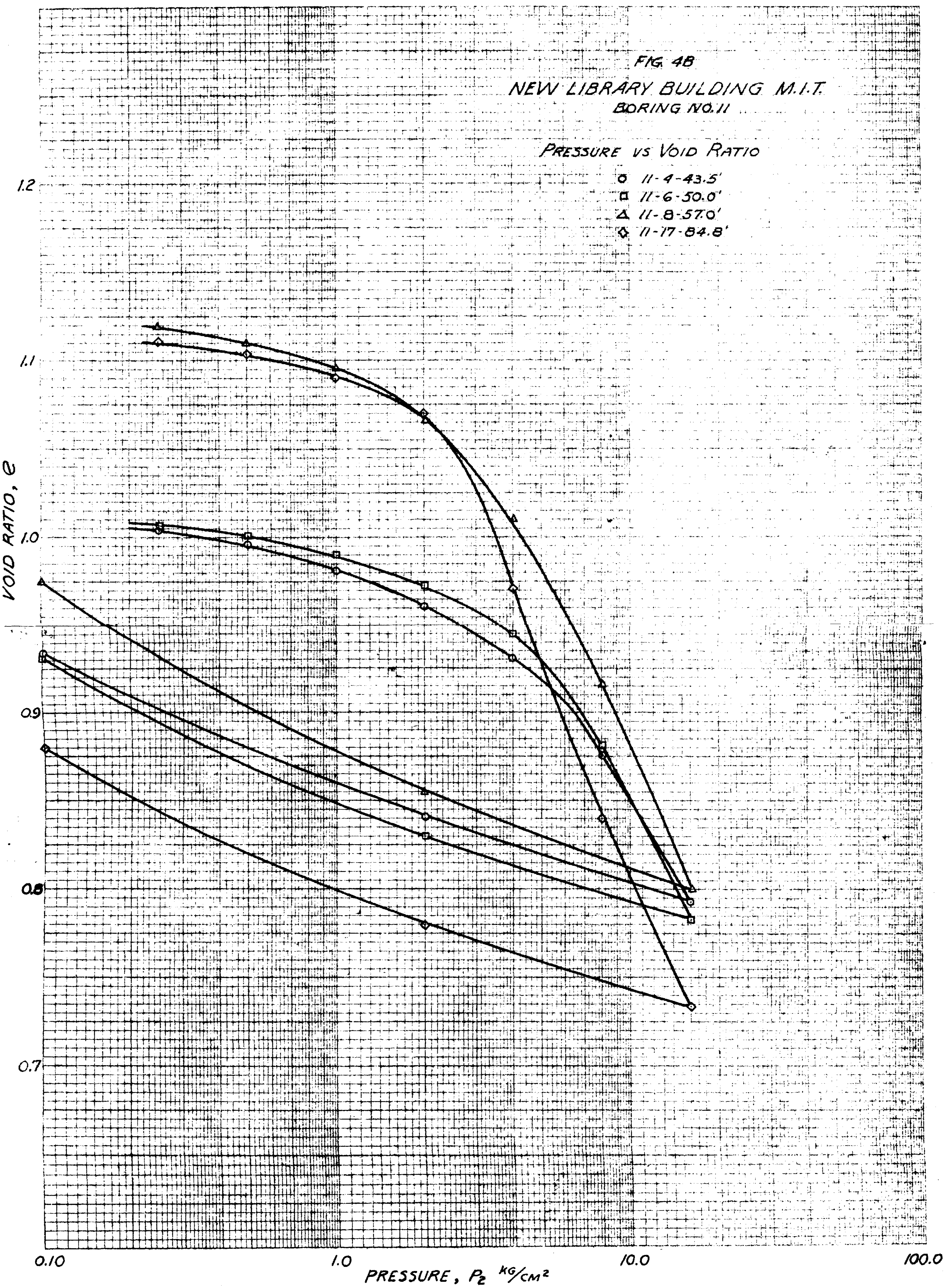


FIG. 4B
NEW LIBRARY BUILDING M.I.T.
BORING NO. 11
PRESSURE VS VOID RATIO
○ 11-4-43.5'
□ 11-6-50.0'
△ 11-8-57.0'
◇ 11-17-84.8'
1.2
1.1
1.0
0.9
0.8
0.7
VOID RATIO, e
0.10
1.0
10.0
100.0
PRESSURE, P_2 KG/CM²

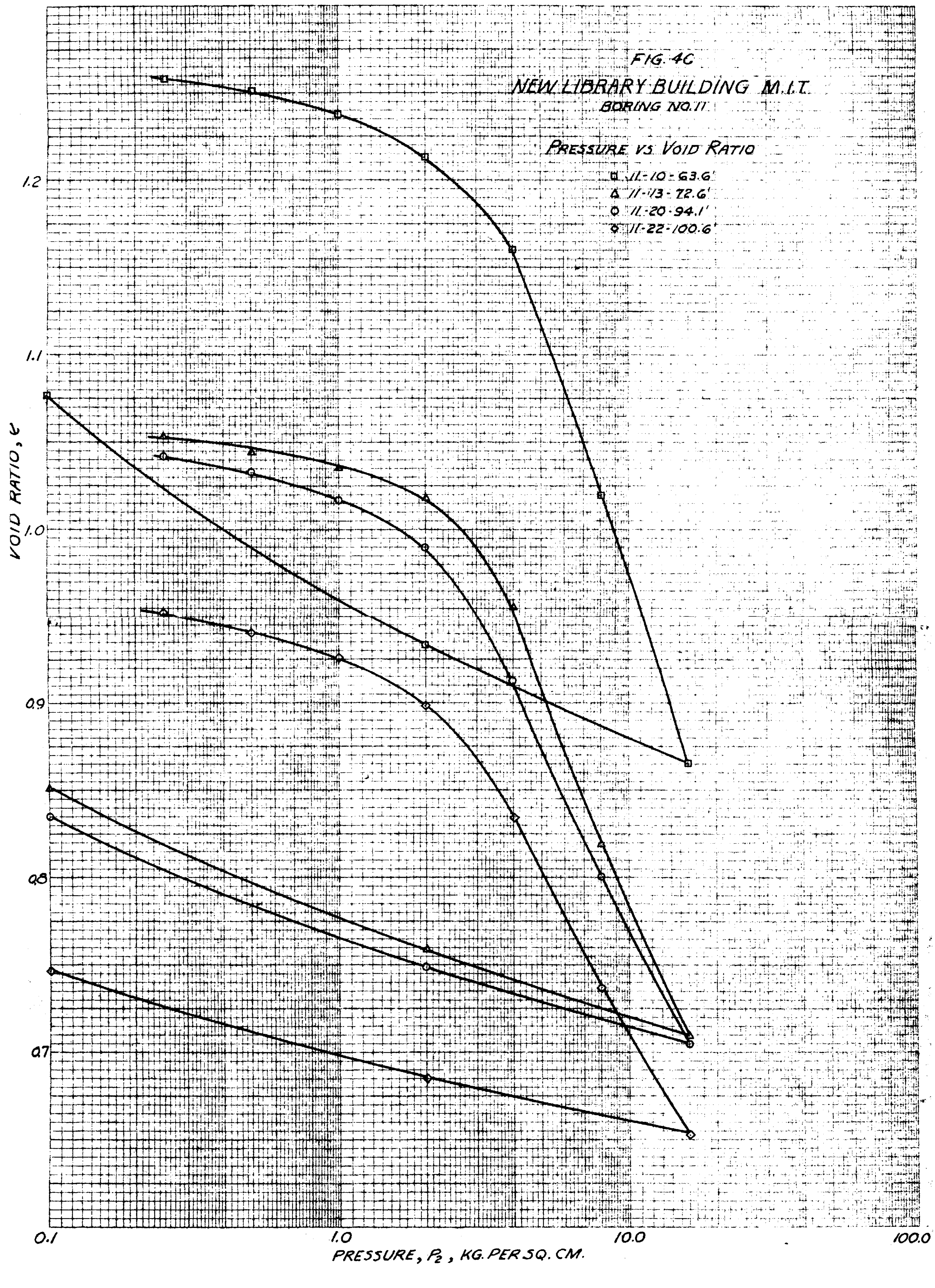


FIG. 4C
NEW LIBRARY BUILDING M.I.T.
BORING NO. 11
PRESSURE VS VOID RATIO
11-10-63.6'
11-13-72.6'
11-20-94.1'
11-22-100.6'
VOID RATIO, e
1.2
1.1
1.0
0.9
0.8
0.7
0.1
1.0
10.0
100.0
PRESSURE, P_2, KG. PER SQ. CM.

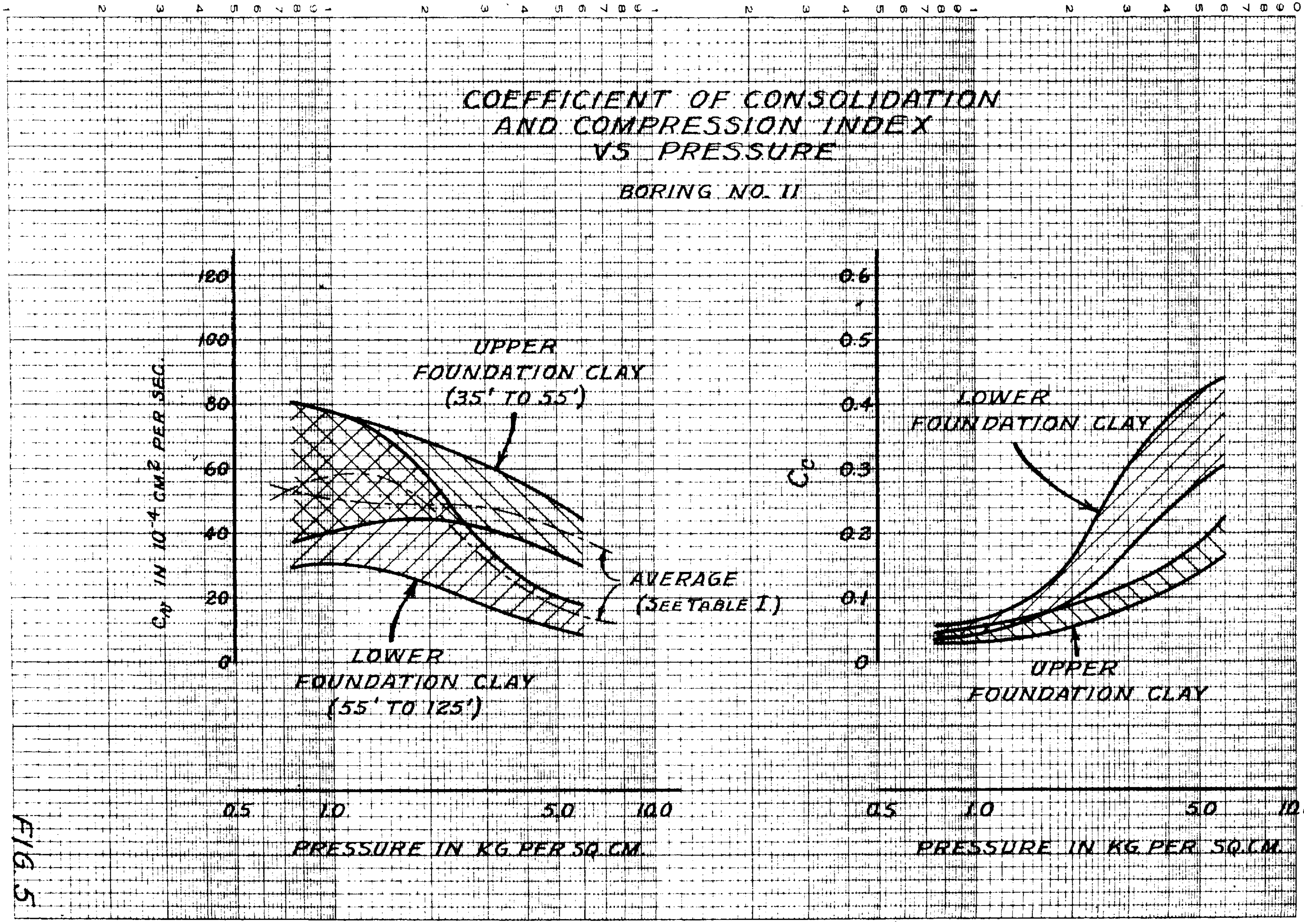
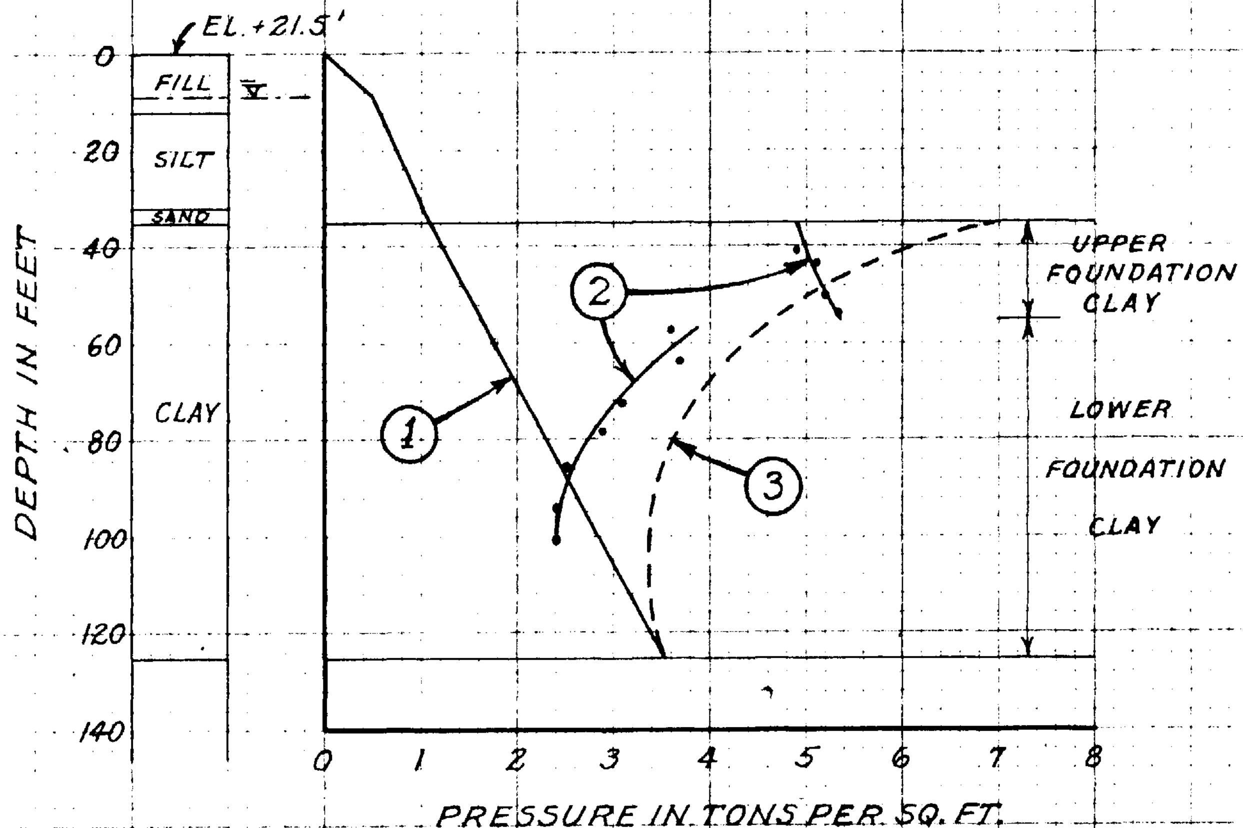

COEFFICIENT OF CONSOLIDATION
AND COMPRESSION INDEX
VS PRESSURE
BORING NO. II
UPPER
FOUNDATION CLAY
(35' TO 55')
LOWER
FOUNDATION CLAY
(55' TO 125')
AVERAGE
(SEE TABLE I.)
C_v IN 10^{-4} CM^2 PER SEC.
PRESSURE IN KG PER SQ.CM.
C_c
LOWER
FOUNDATION CLAY
UPPER
FOUNDATION CLAY
PRESSURE IN KG PER SQ.CM.

FIG. 5



KEY:

(1) OVERBURDEN INTERGRANULAR PRESSURE

(2) MAXIMUM PAST PRESSURE FROM CONSOLIDATION TESTS

(3) ADJUSTED MAXIMUM PAST PRESSURE

OVERBURDEN PRESSURE AND MAXIMUM PAST PRESSURE VS DEPTH

FIG. 6

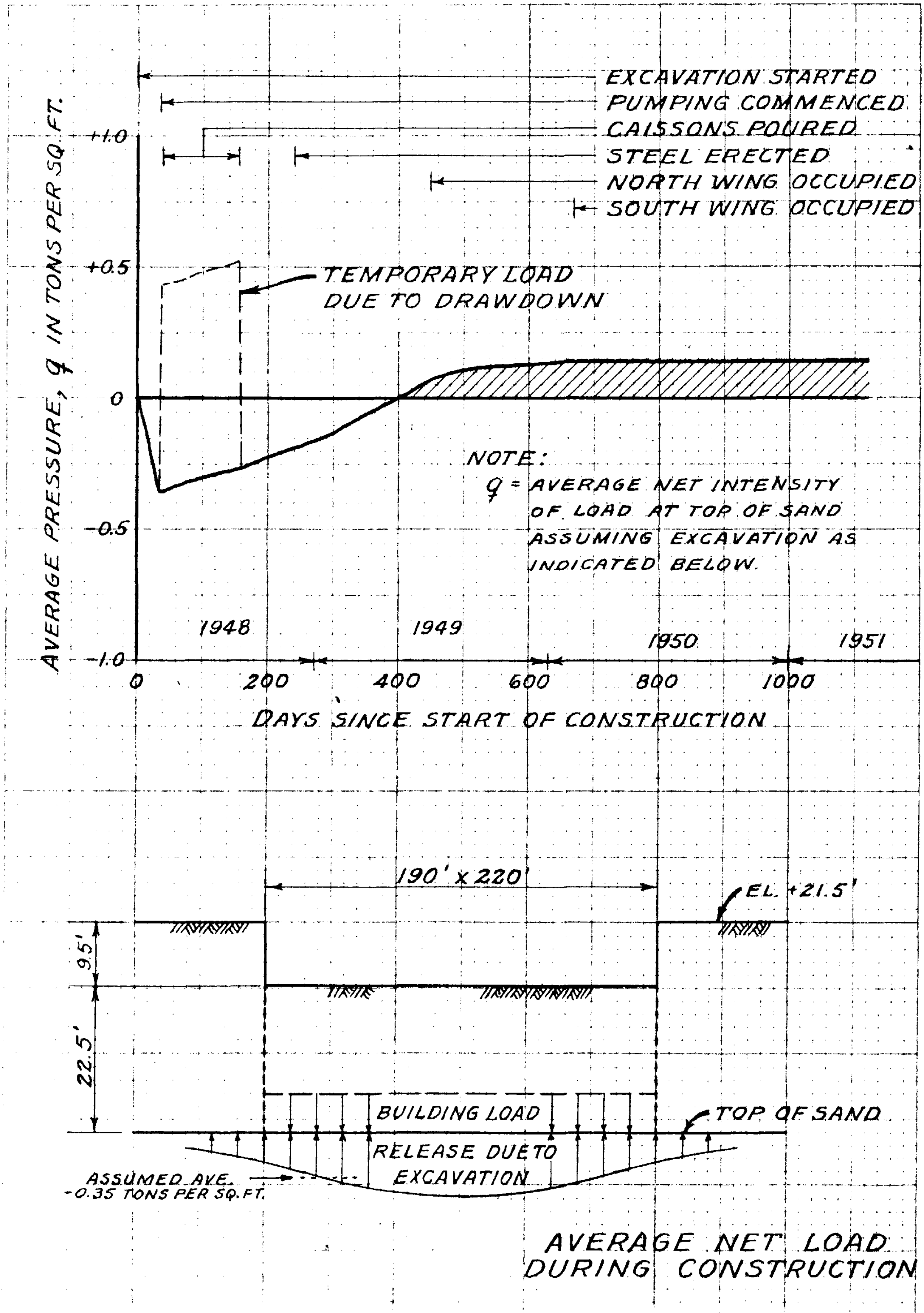
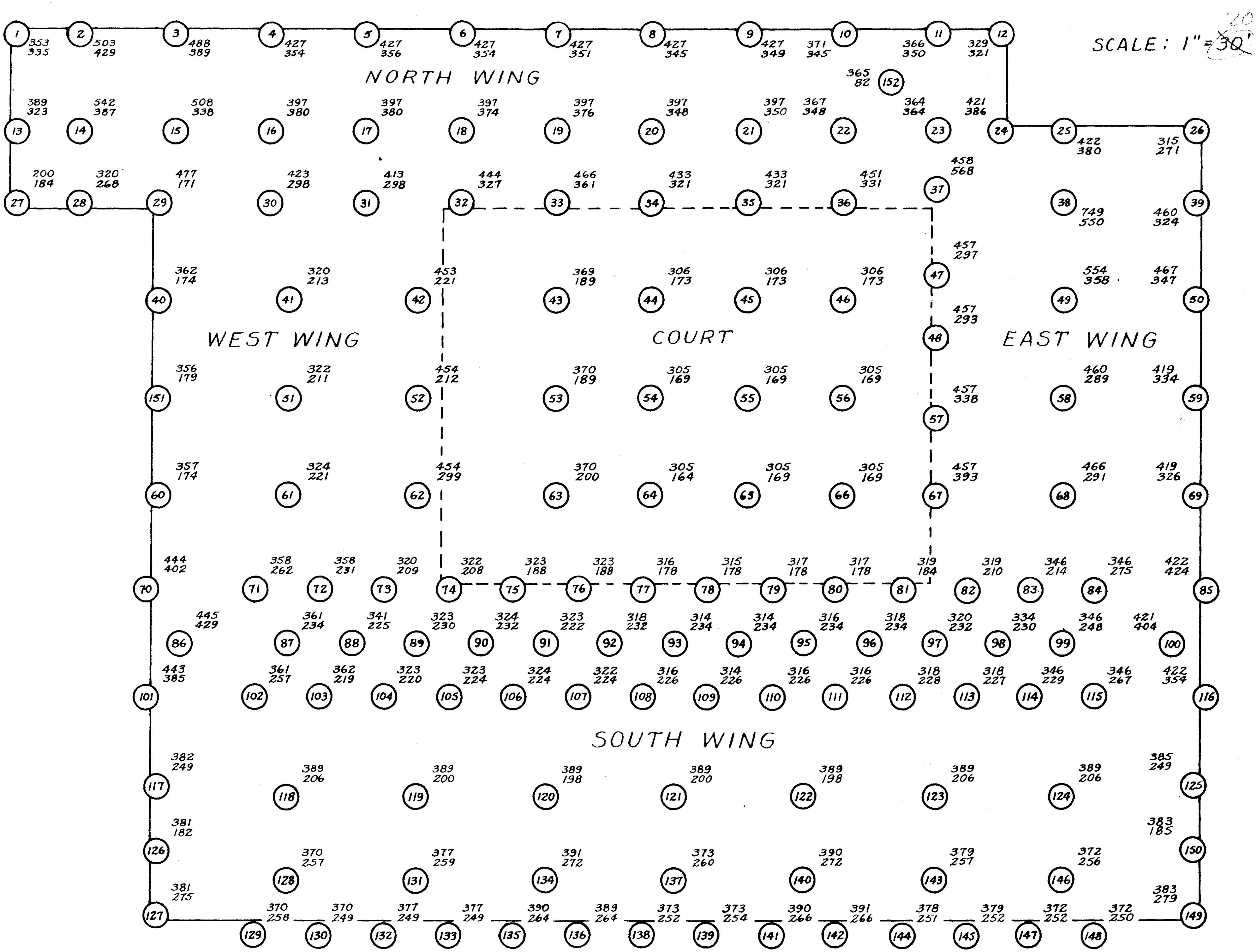

EXCAVATION STARTED
PUMPING COMMENCED
CAISSONS POURED
STEEL ERECTED
NORTH WING OCCUPIED
SOUTH WING OCCUPIED
TEMPORARY LOAD DUE TO DRAWDOWN
AVERAGE PRESSURE, q IN TONS PER SQ. FT.
+1.0
+0.5
0
-0.5
-1.0
NOTE:
q = AVERAGE NET INTENSITY OF LOAD AT TOP OF SAND ASSUMING EXCAVATION AS INDICATED BELOW.
1948
1949
1950
1951
0
200
400
600
800
1000
DAYS SINCE START OF CONSTRUCTION
190' x 220'
EL. +21.5'
9.5'
22.5'
BUILDING LOAD
TOP OF SAND
RELEASE DUE TO EXCAVATION
ASSUMED AVE. -0.35 TONS PER SQ. FT.
AVERAGE NET LOAD DURING CONSTRUCTION

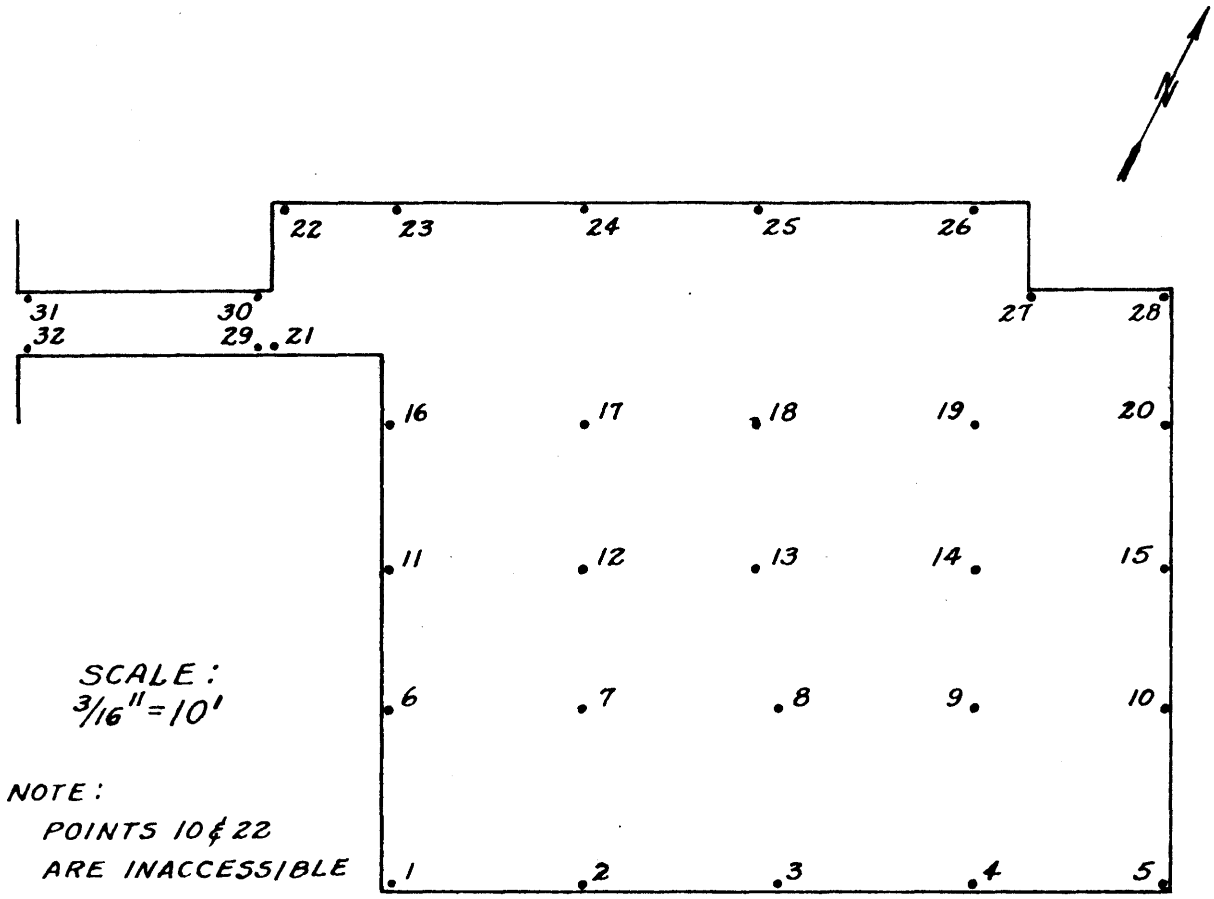
FIG. 7



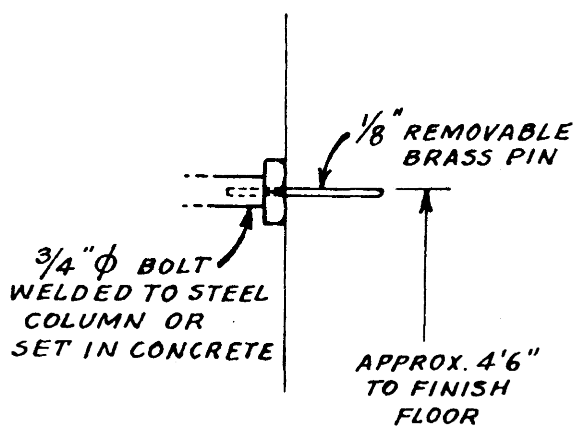
KEY:
TOP NUMBER - DESIGN LOAD IN KIPS
BOTTOM NUMBER - EXISTING LOAD IN KIPS

CAISSON PLAN

FIG 8



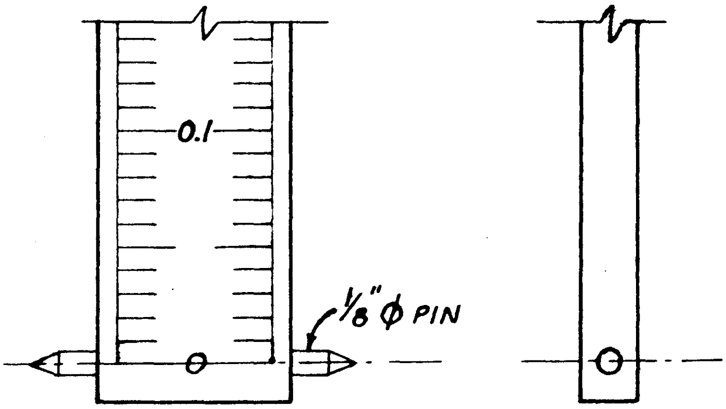
SETTLE-MENT POINT	BOLT IN USE
7	
8	
9	
12	
13	
14	
17	
18	
19	



DETAIL OF
OBSERVATION POINT

LOCATION OF
SETTLEMENT OBSERVATION
POINTS

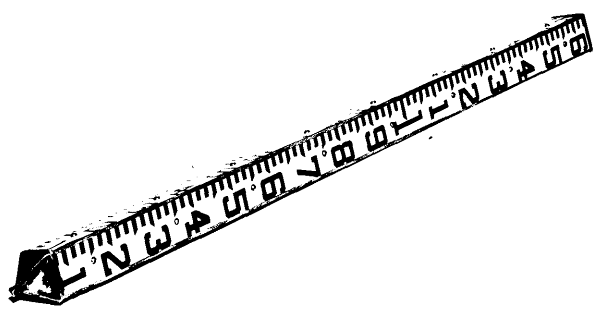
FIG. 9



FRONT
ELEVATION

SIDE
ELEVATION

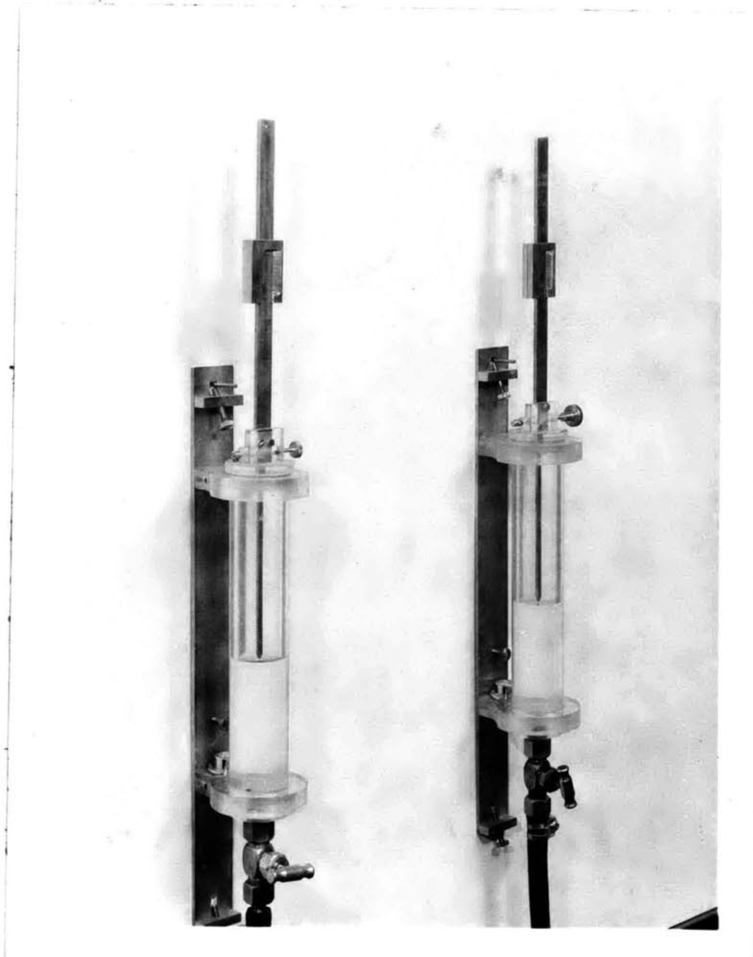
(a)



(b)

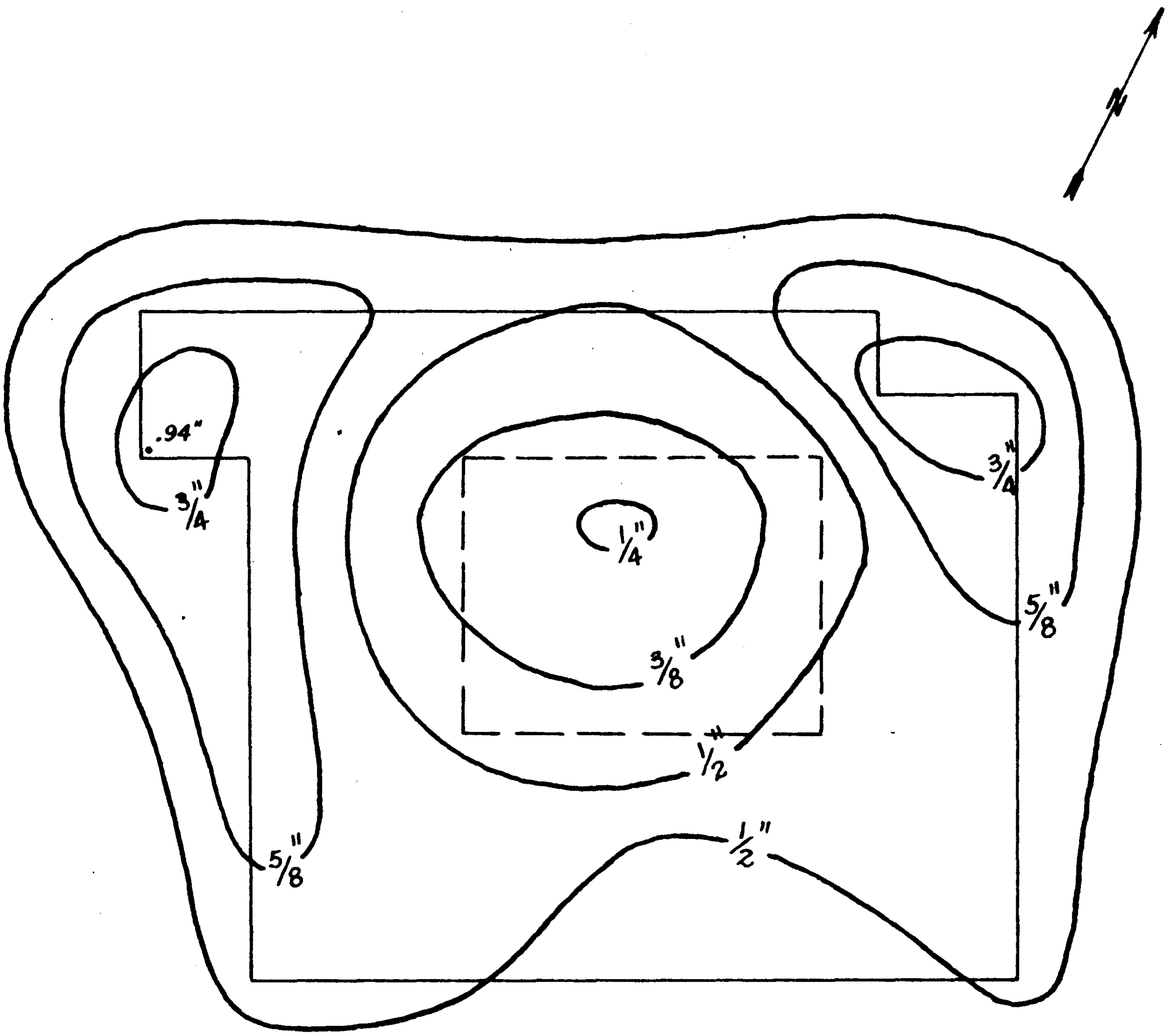
SPECIAL RODS
USED FOR
SETTLEMENT READINGS

FIG. 10



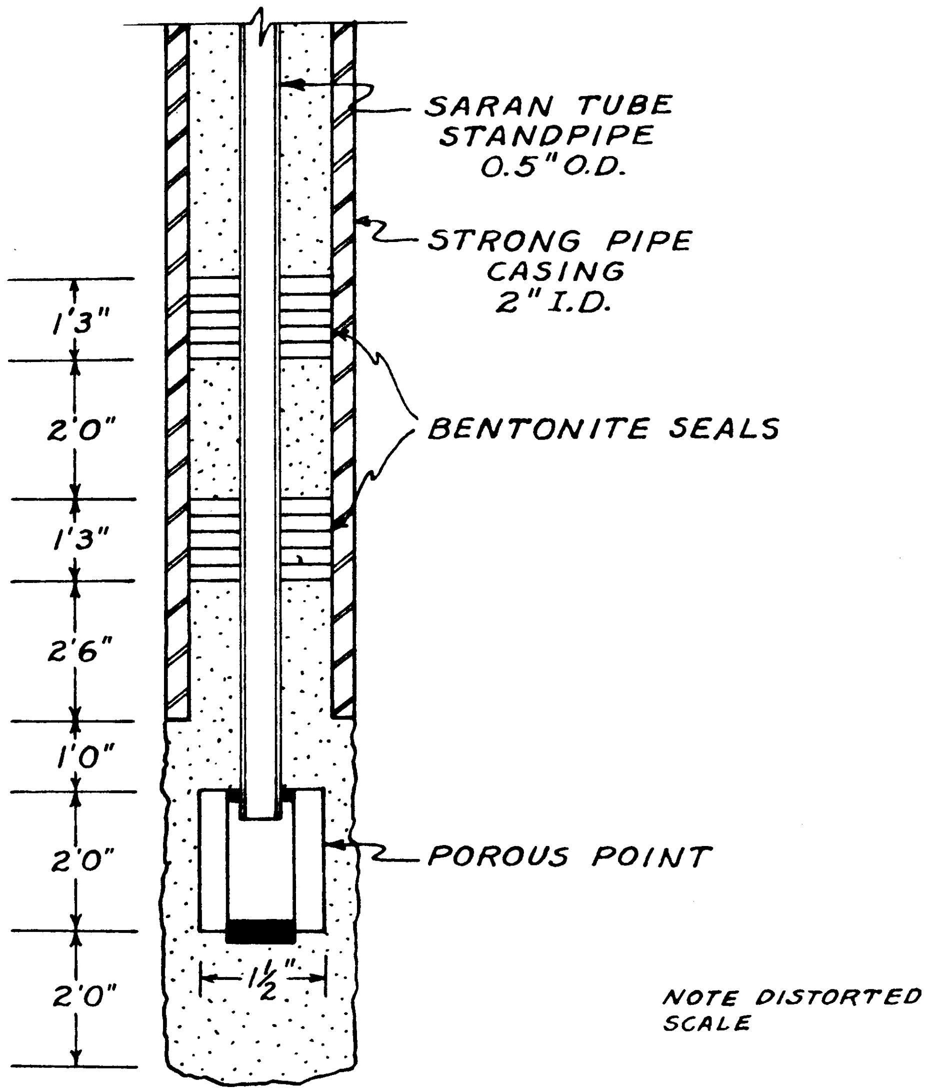
THE AQUALEVEL

FIG. 11



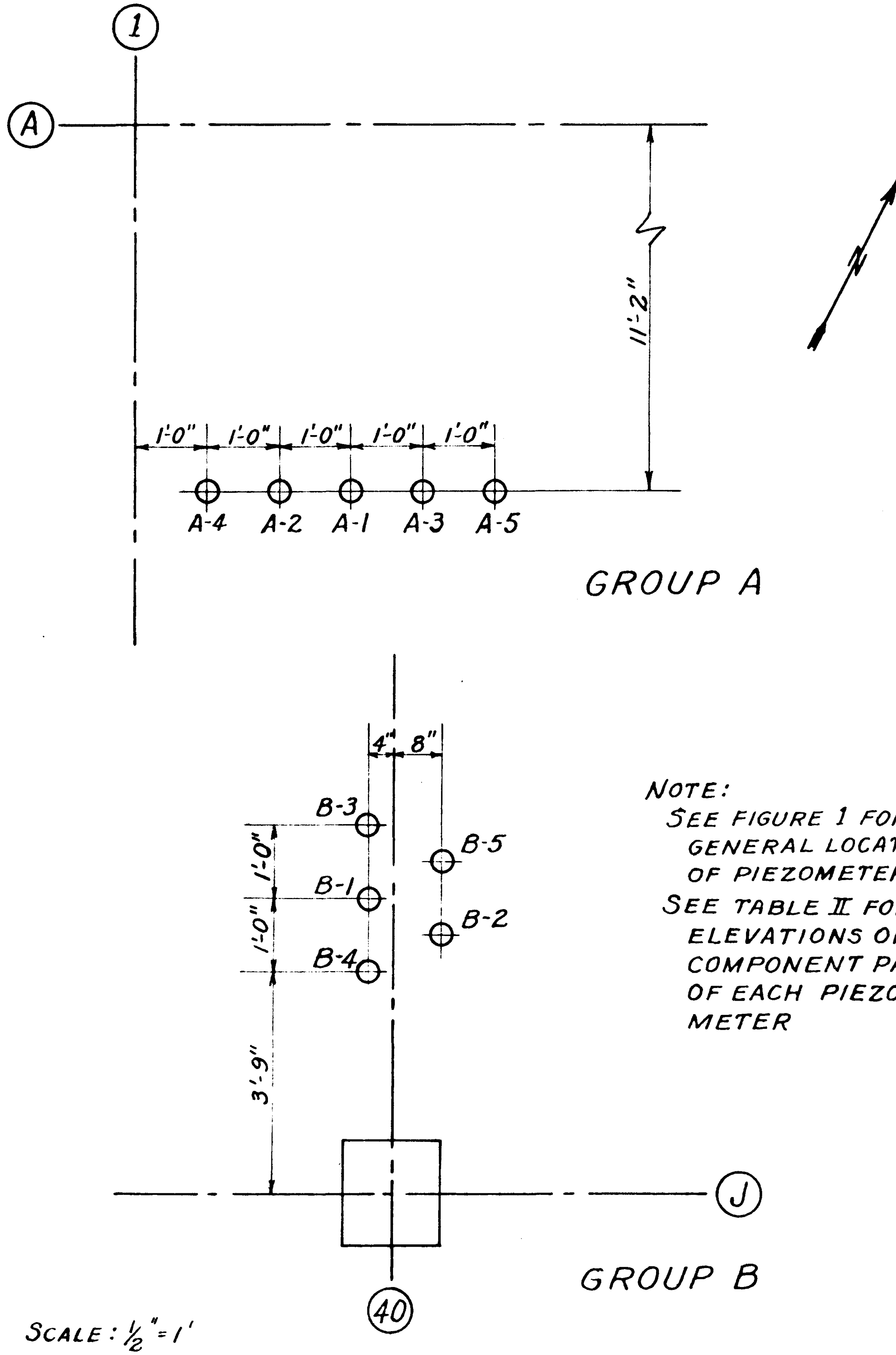
CONTOURS OF
EQUAL SETTLEMENT
JAN. 1, 1951

FIG. 12



SECTION THROUGH PIEZOMETER ASSEMBLY

FIG. 13

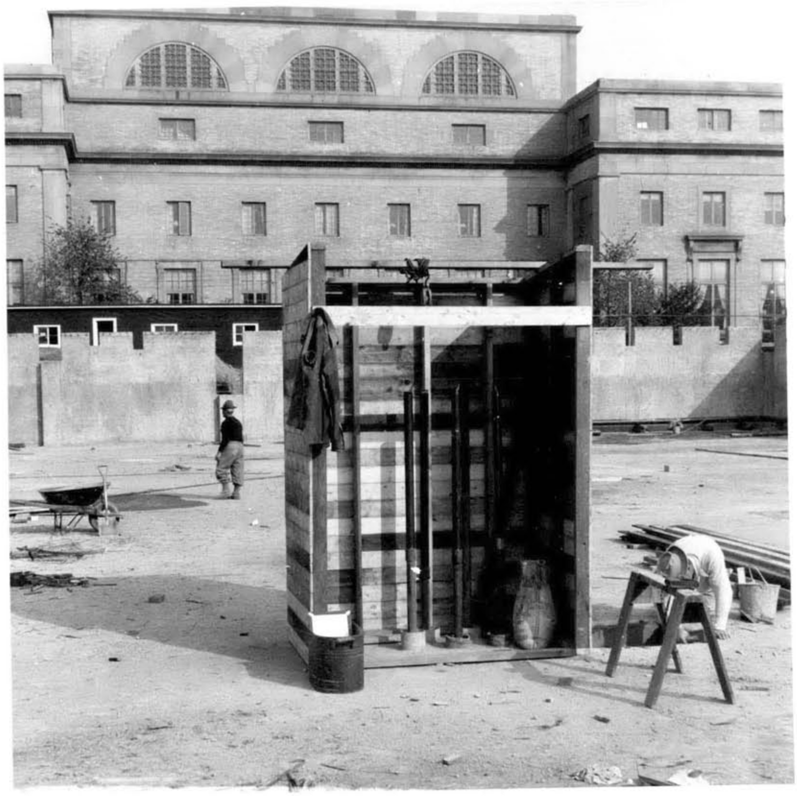


NOTE:
SEE FIGURE 1 FOR GENERAL LOCATION OF PIEZOMETERS
SEE TABLE II FOR ELEVATIONS OF COMPONENT PARTS OF EACH PIEZOMETER

SCALE: ½" = 1'

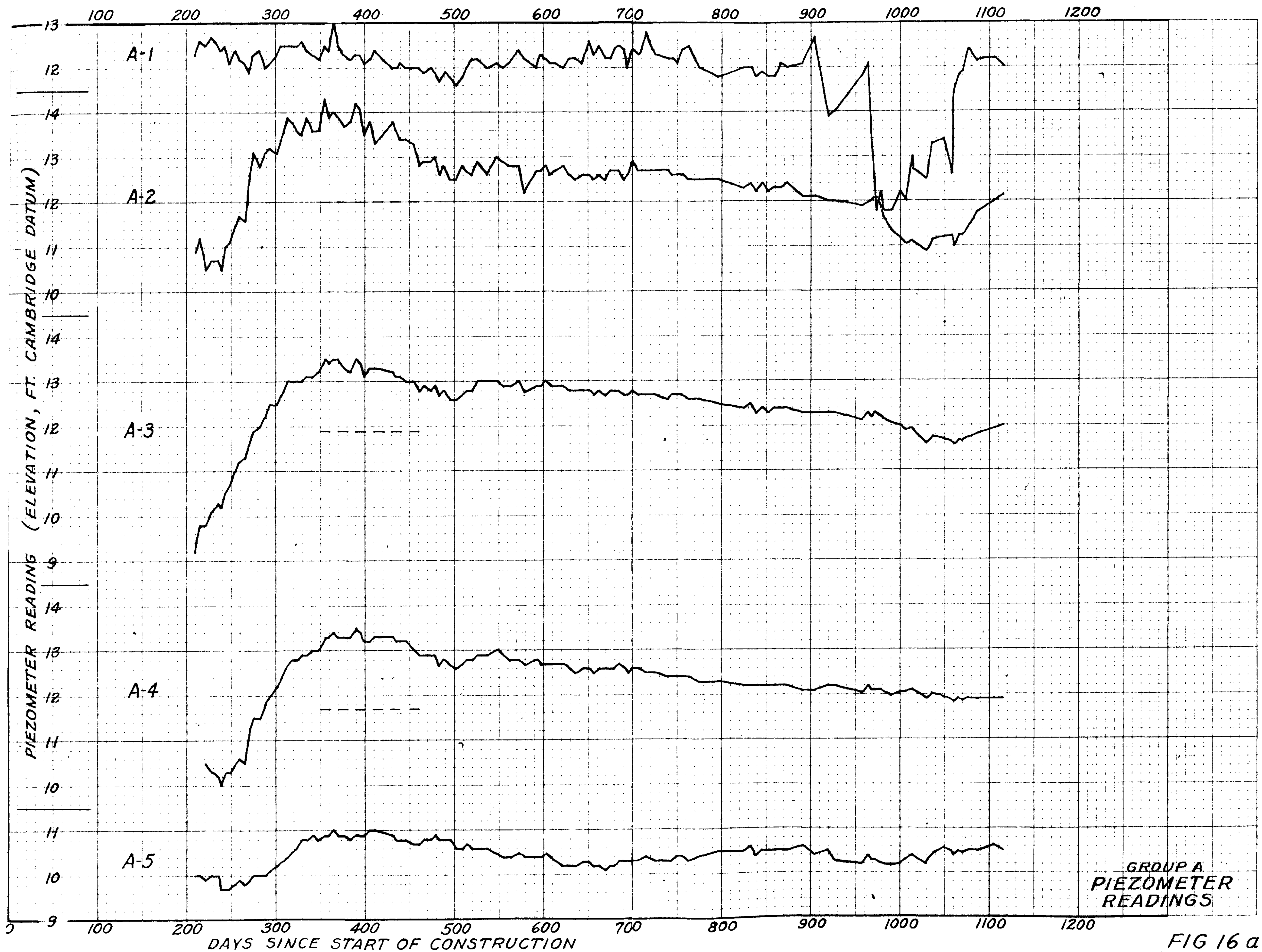
PIEZOMETER LOCATION
AND NUMBERING

FIG. 14



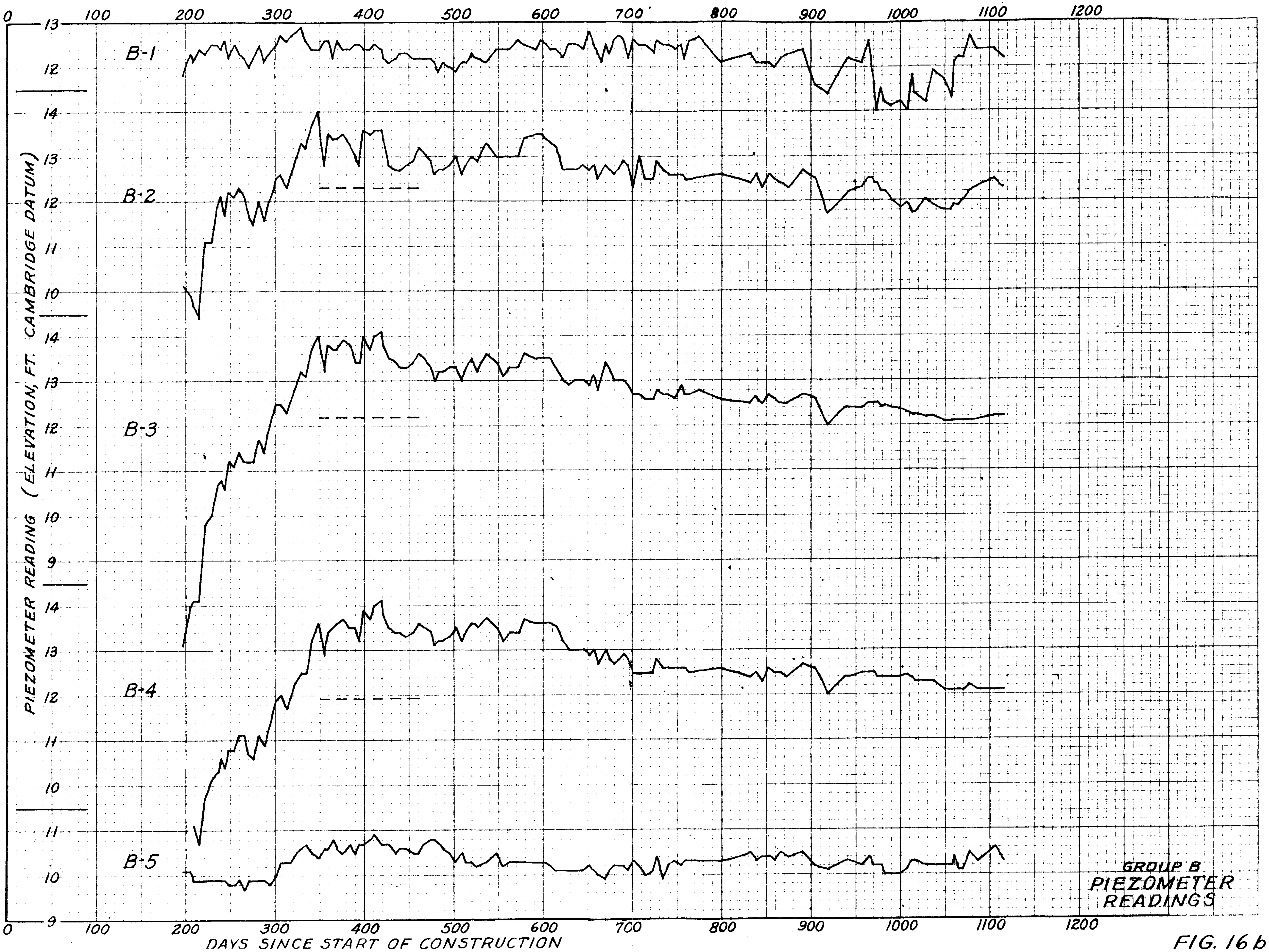
PHOTOGRAPH OF
GROUP B PIEZOMETERS

FIG. 15



A-1
A-2
A-3
A-4
A-5
PIEZOMETER READING (ELEVATION, FT. CAMBRIDGE DATUM)
DAYS SINCE START OF CONSTRUCTION
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
9
10
11
12
13
14
GROUP A
PIEZOMETER READINGS

FIG 16 a



GROUP B
PIEZOMETER
READINGS
B-1
B-2
B-3
B-4
B-5
PIEZOMETER READING (ELEVATION, FT. CAMBRIDGE DATUM)
DAYS SINCE START OF CONSTRUCTION
0
100
200
300
400
500
600
700
800
900
1000
1100
1200

FIG. 16 b

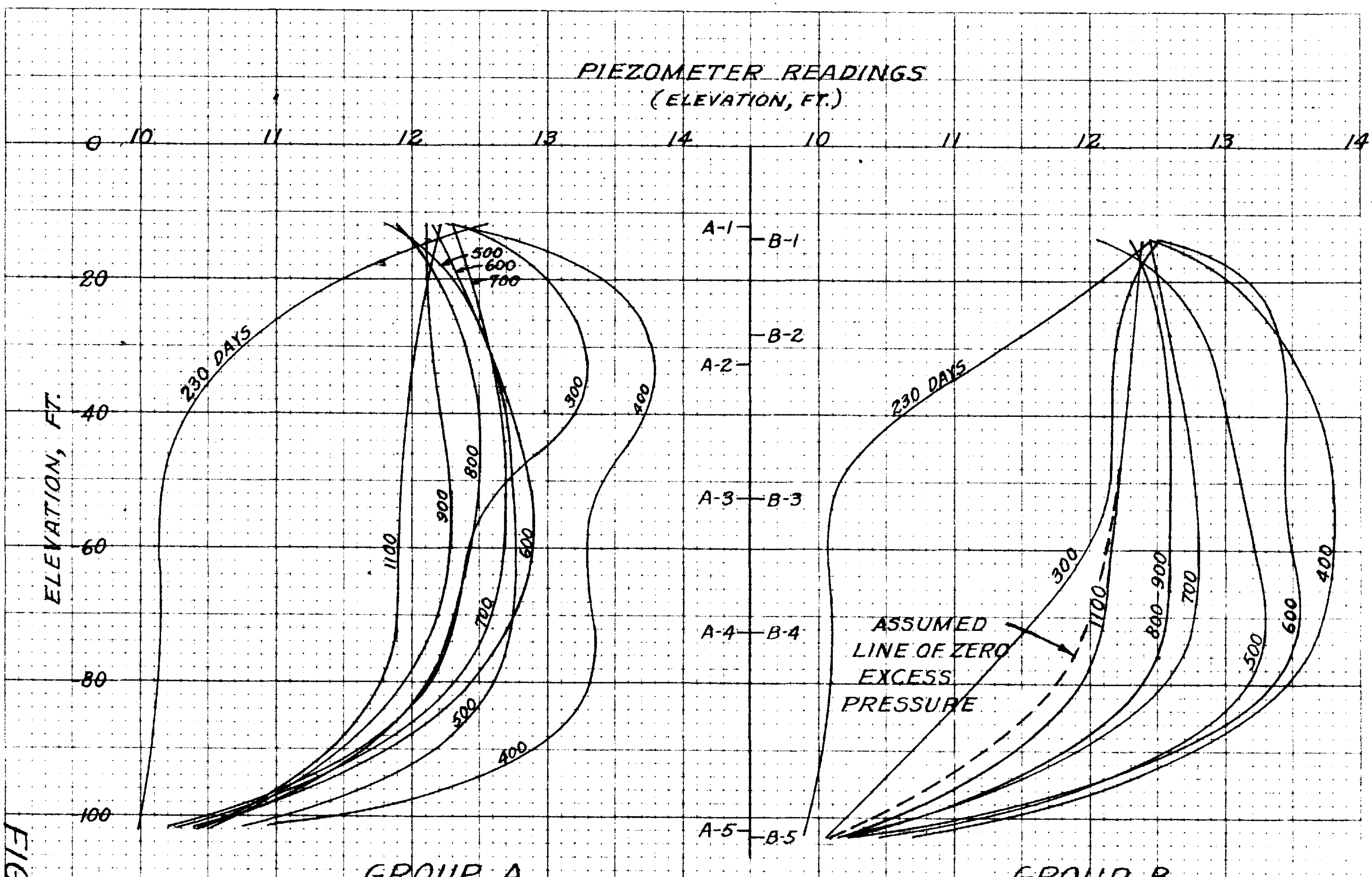


FIG. 17

PIEZOMETRIC ISOCHRONES

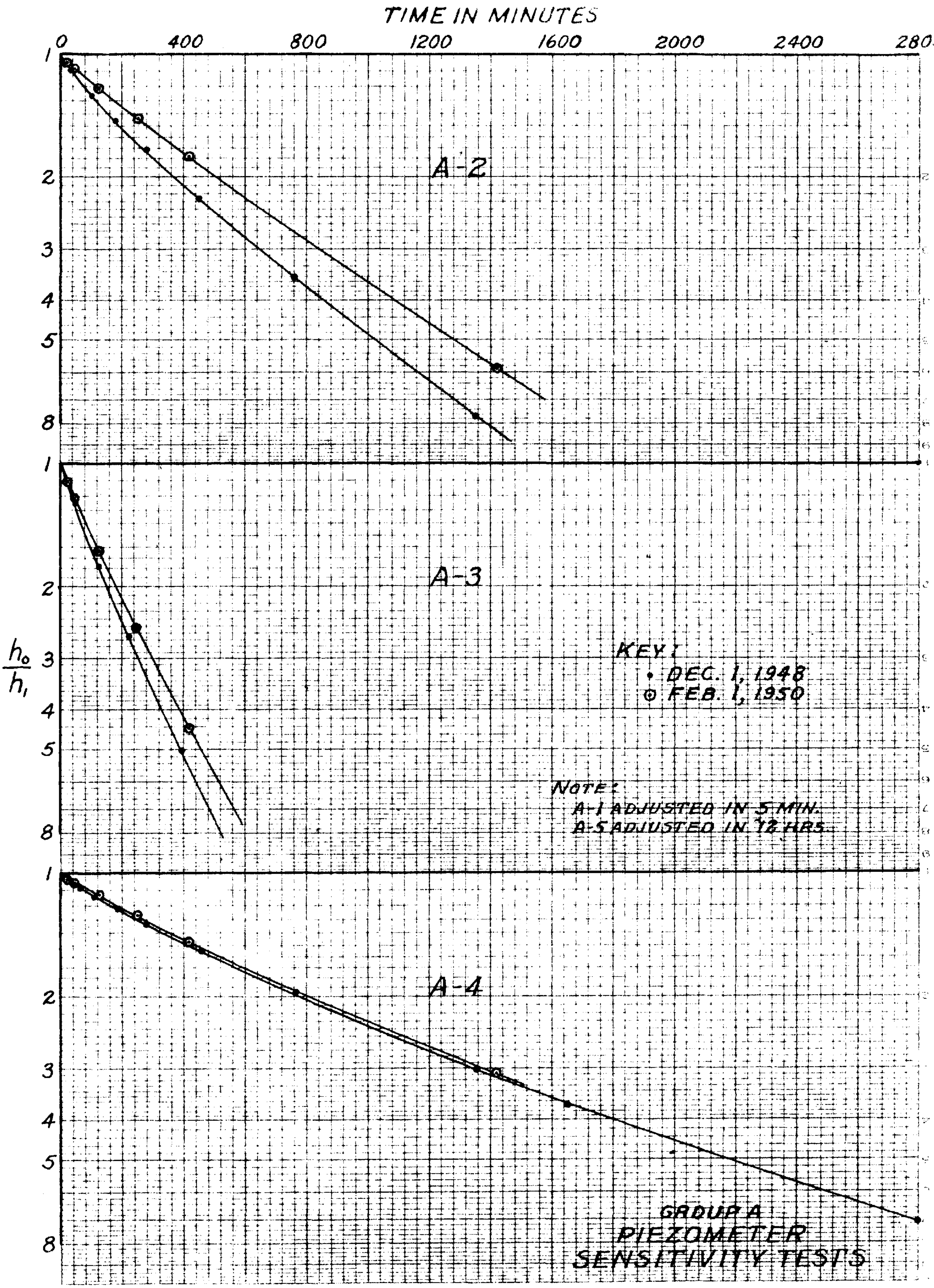
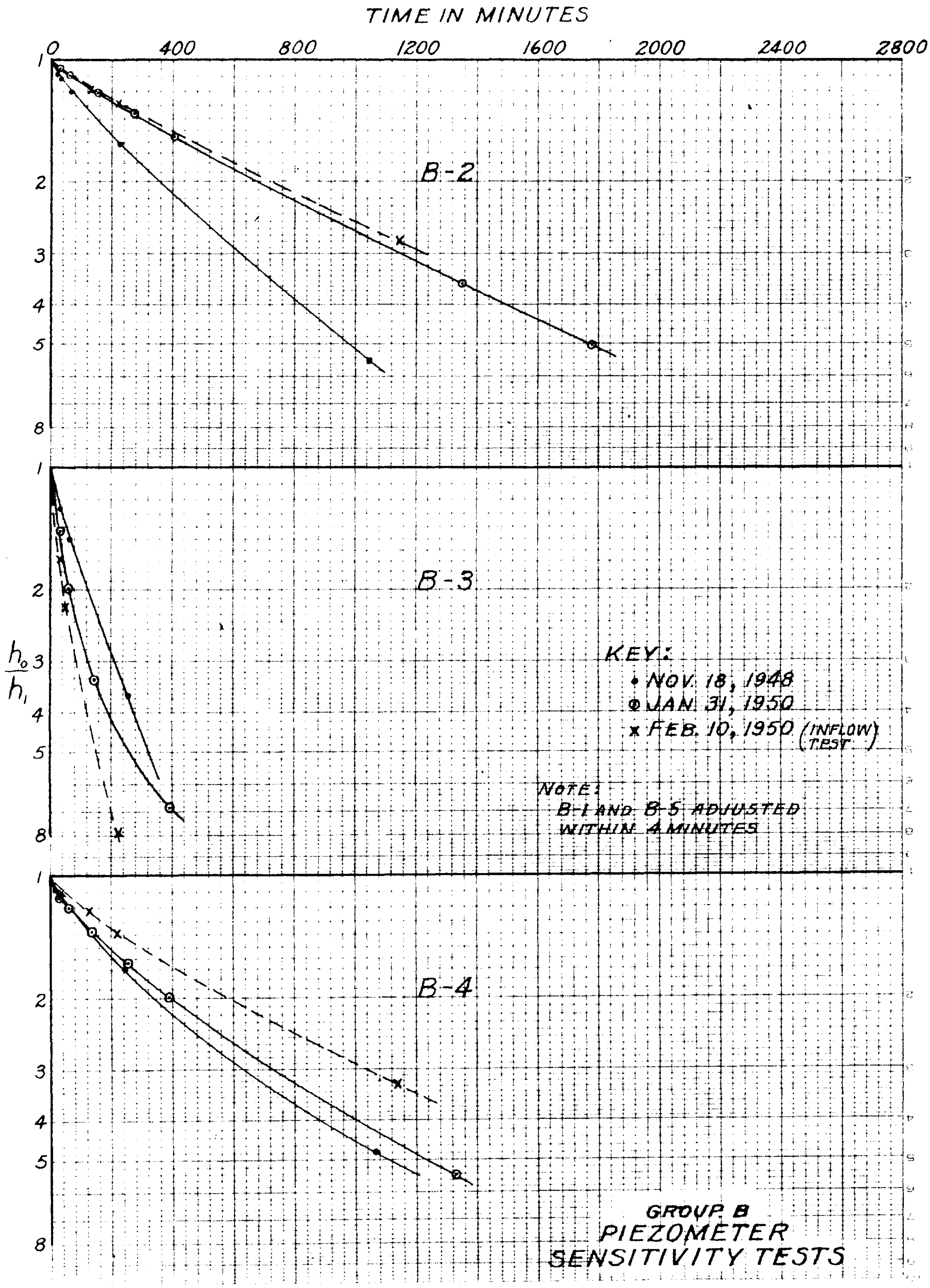

TIME IN MINUTES
0
400
800
1200
1600
2000
2400
280
1
2
3
4
5
8
A-2
A-3
A-4
$\frac{h_0}{h_1}$
KEY:
• DEC. 1, 1948
⊙ FEB. 1, 1950
NOTE:
A-1 ADJUSTED IN 5 MIN.
A-5 ADJUSTED IN 12 HRS.
GROUP A
PIEZOMETER
SENSITIVITY TESTS

FIG. 18 a



TIME IN MINUTES
0
400
800
1200
1600
2000
2400
2800
B-2
B-3
B-4
$\frac{h_0}{h_1}$
KEY:
• NOV. 18, 1948
⊙ JAN. 31, 1950
x FEB. 10, 1950 (INFLOW TEST)
NOTE:
B-1 AND B-5 ADJUSTED WITHIN 4 MINUTES
GROUP B
PIEZOMETER
SENSITIVITY TESTS

FIG 18b

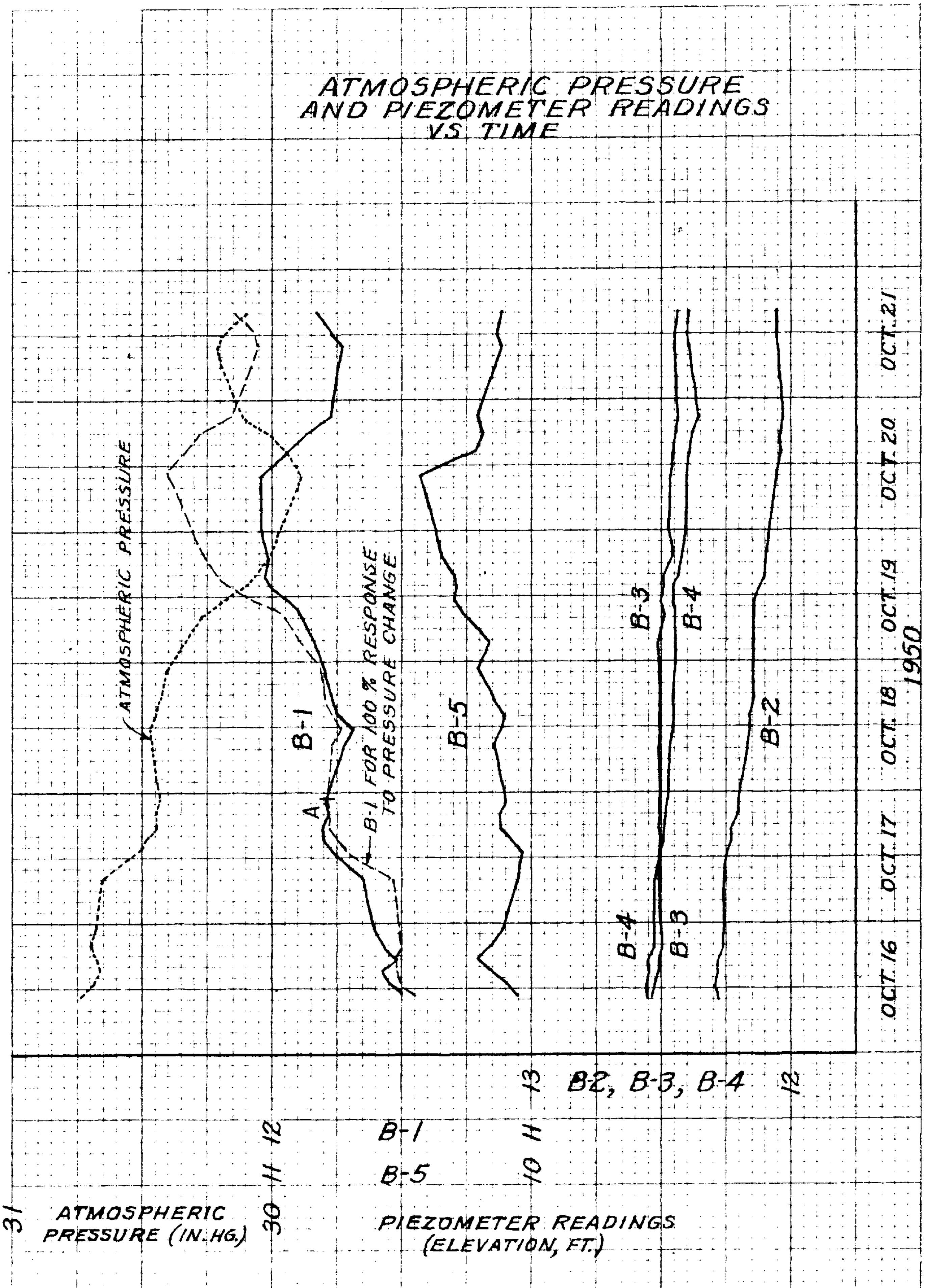
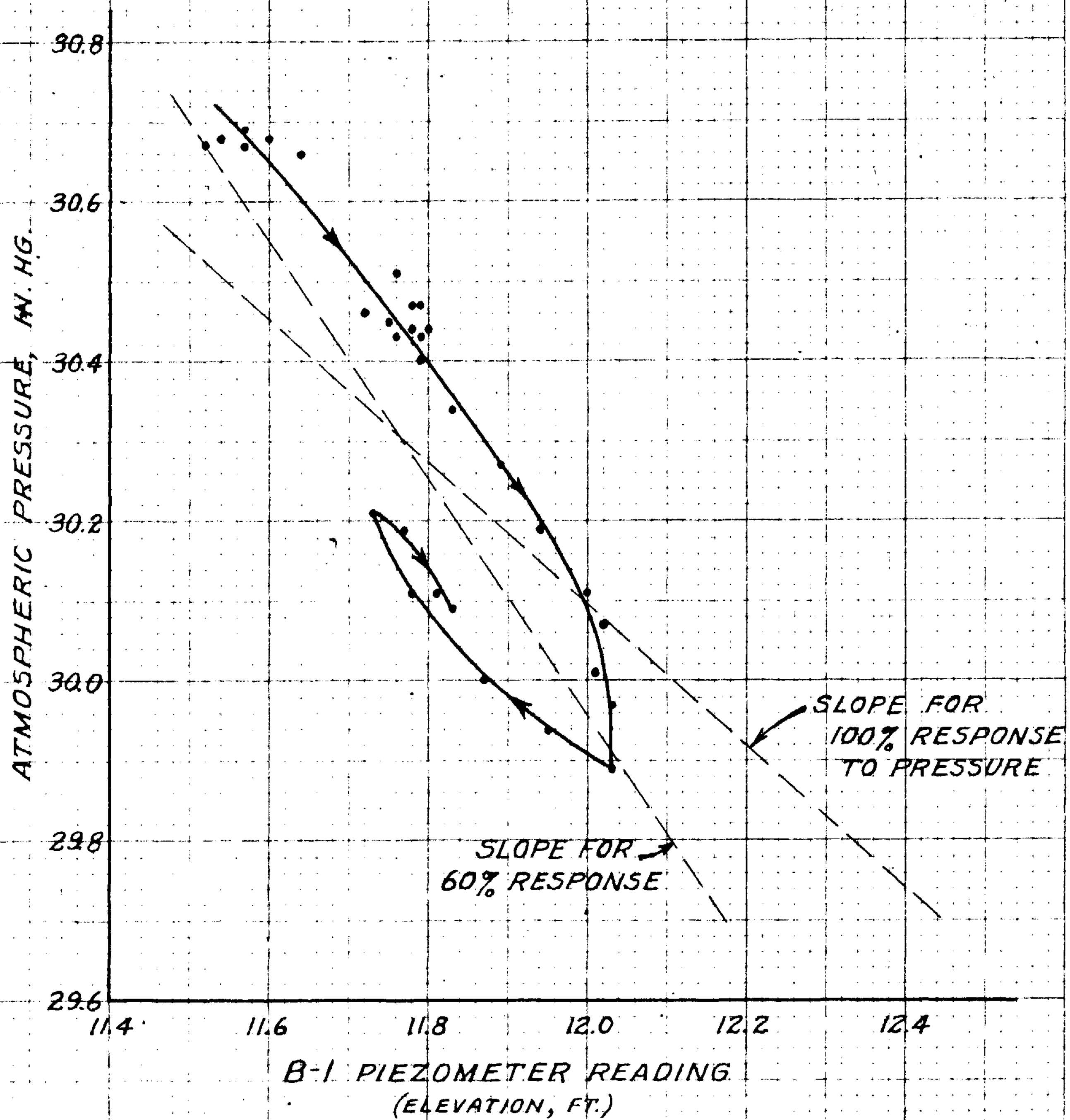

ATMOSPHERIC PRESSURE
AND PIEZOMETER READINGS
VS TIME
ATMOSPHERIC PRESSURE
B-1
A
B-1 FOR 100 % RESPONSE TO PRESSURE CHANGE
B-5
B-3
B-4
B-2
OCT. 16
OCT. 17
OCT. 18
OCT. 19
OCT. 20
OCT. 21
1950
13
B-2, B-3, B-4
12
B-1
B-5
10
11
12
31
ATMOSPHERIC
PRESSURE (IN. HG.)
30
PIEZOMETER READINGS
(ELEVATION, FT.)

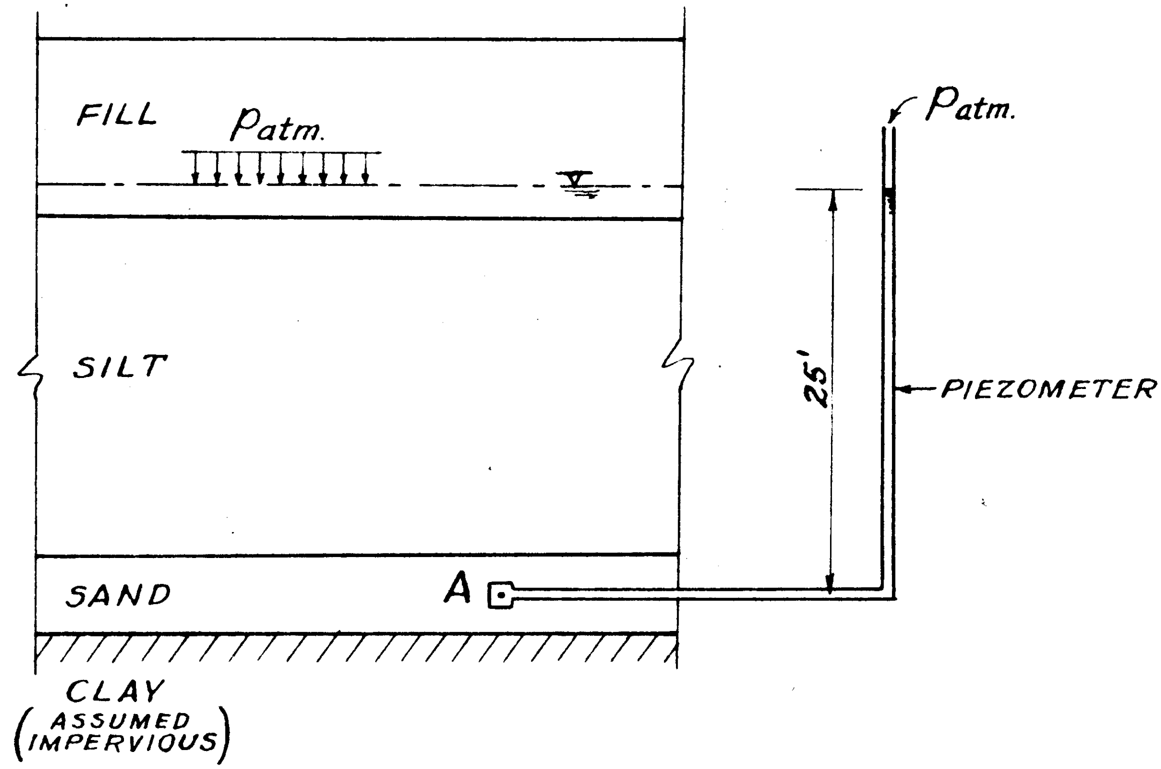
FIG. 19

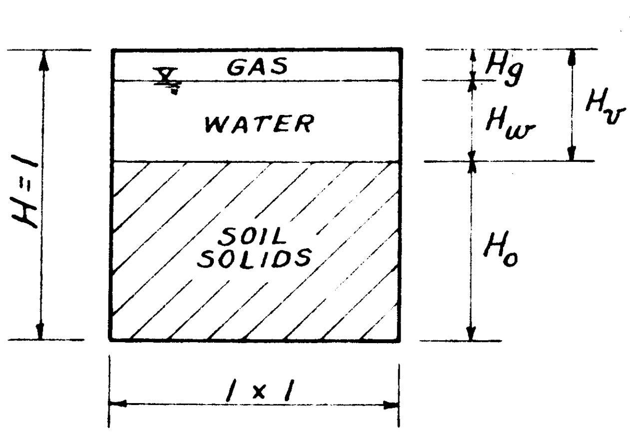


ATMOSPHERIC PRESSURE VS PIEZOMETER B-1

FIG. 20

PRESSURE CONDITIONS AT PIEZOMETER B-1





SCHEMATIC REPRESENTATION OF ELEMENT A

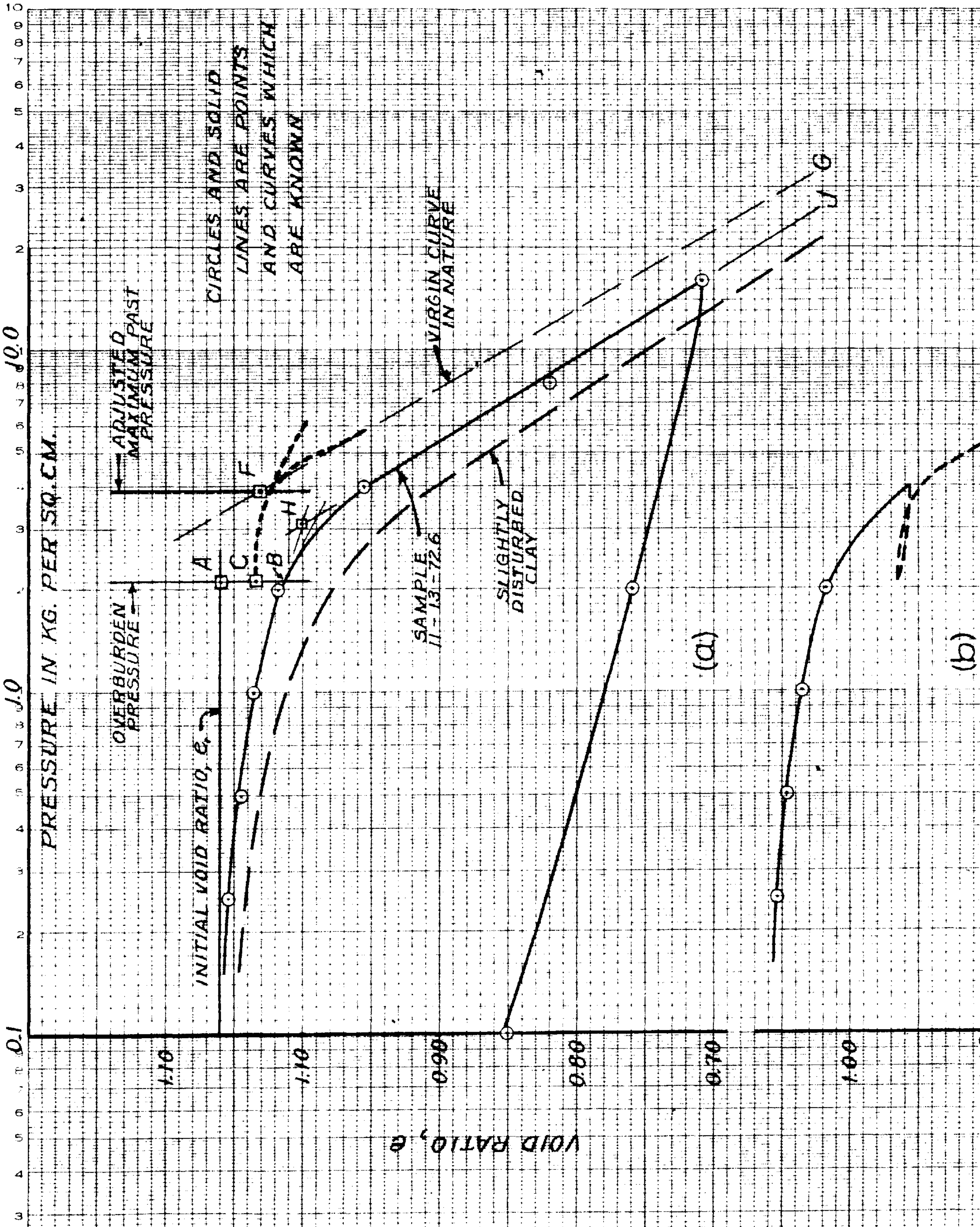
INITIAL PRESSURES AT POINT A:

INTERGRANULAR PRESSURE = $\bar{\sigma}_i$

WATER PRESSURE = u_i
$(25' \times \gamma_w)$

ABSOLUTE WATER PRESSURE = ABSOLUTE GAS PRESSURE
$= u_i + p_{atm.\,i}$

FIG. 21



LABORATORY vs FIELD COMPRESSION CURVES

FIG.22

TAYLOR BOND HYPOTHESIS

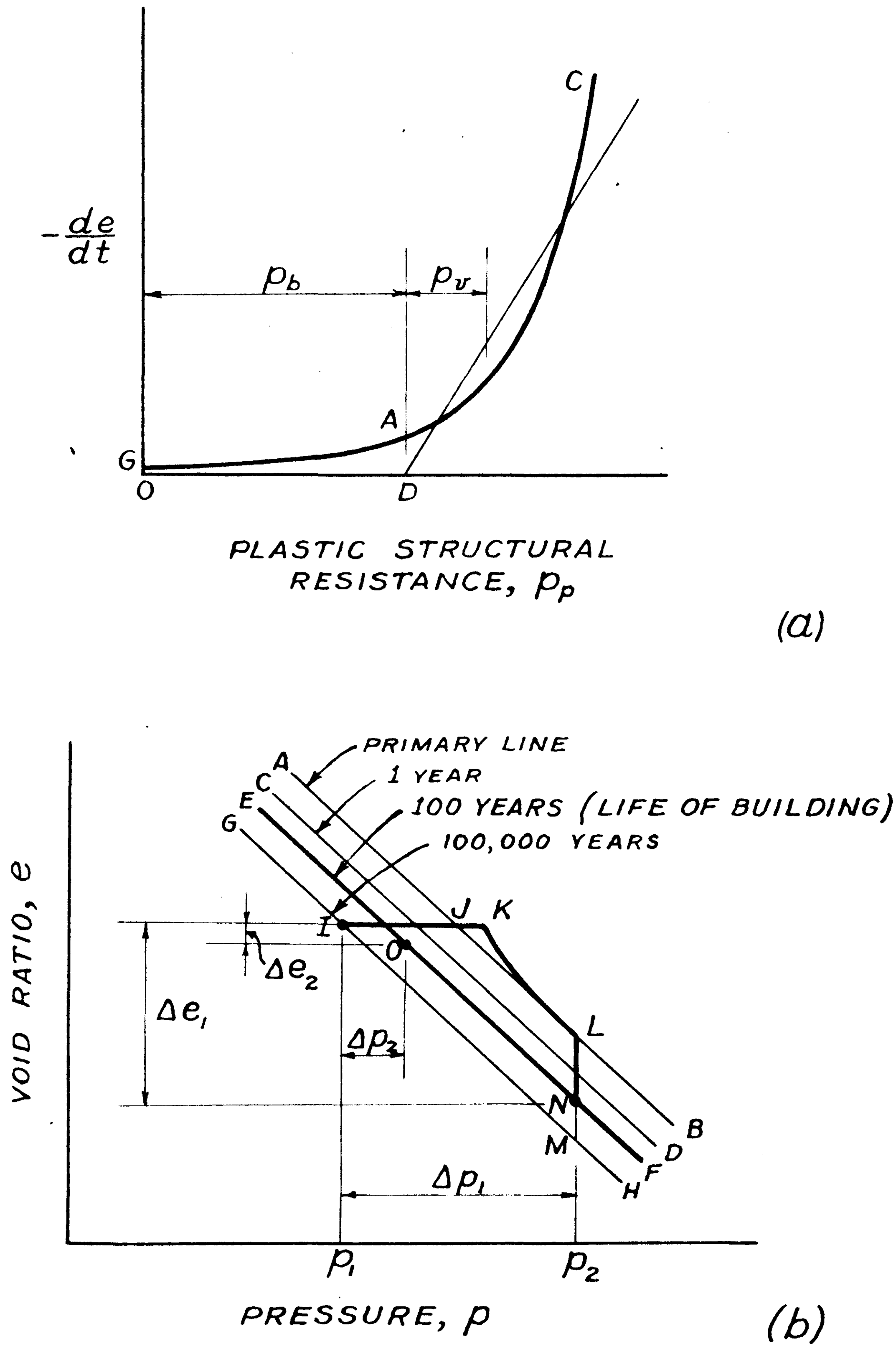
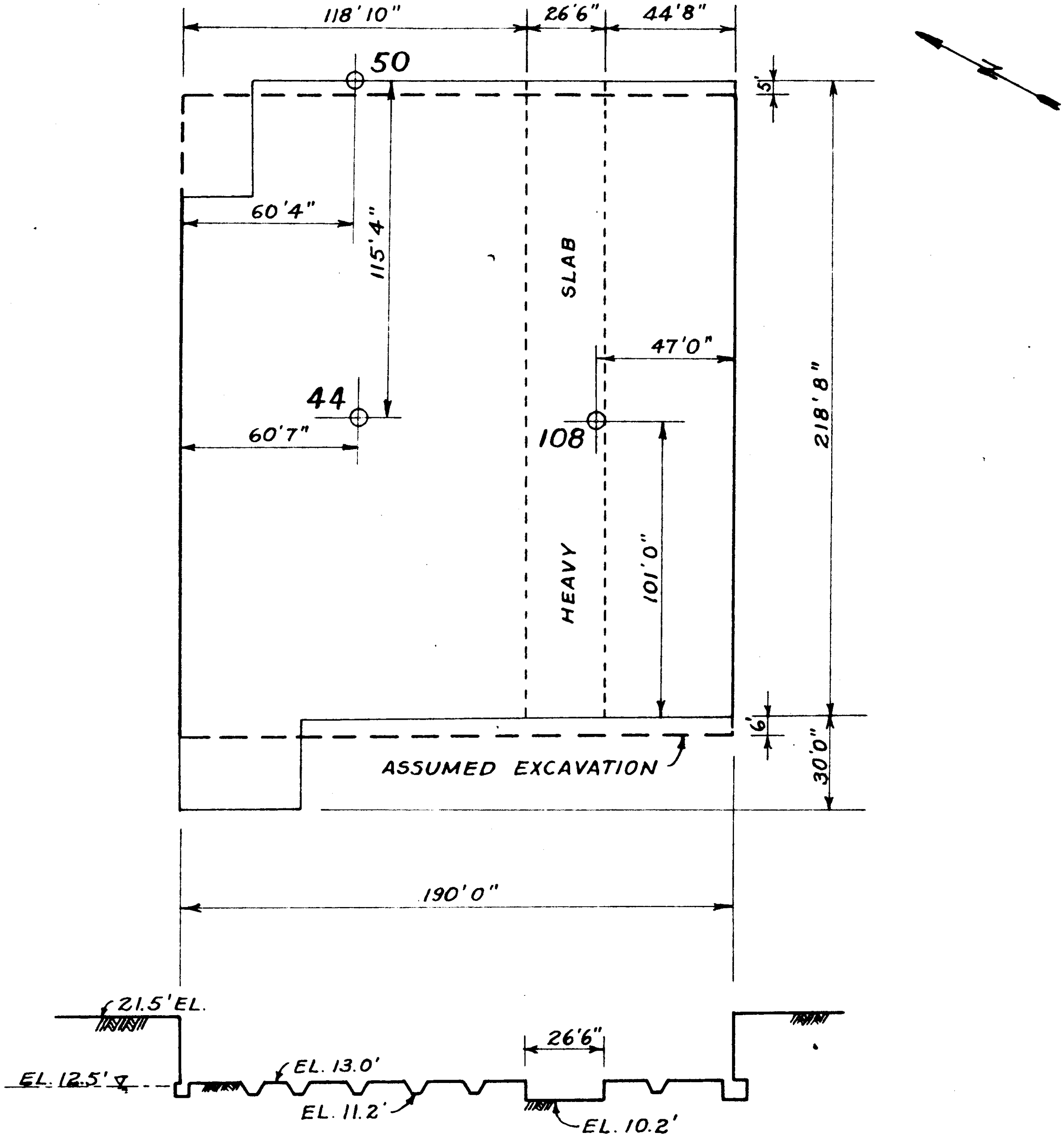


FIG. 23



HORIZ. SCALE: $\frac{3}{16}$" = 10'

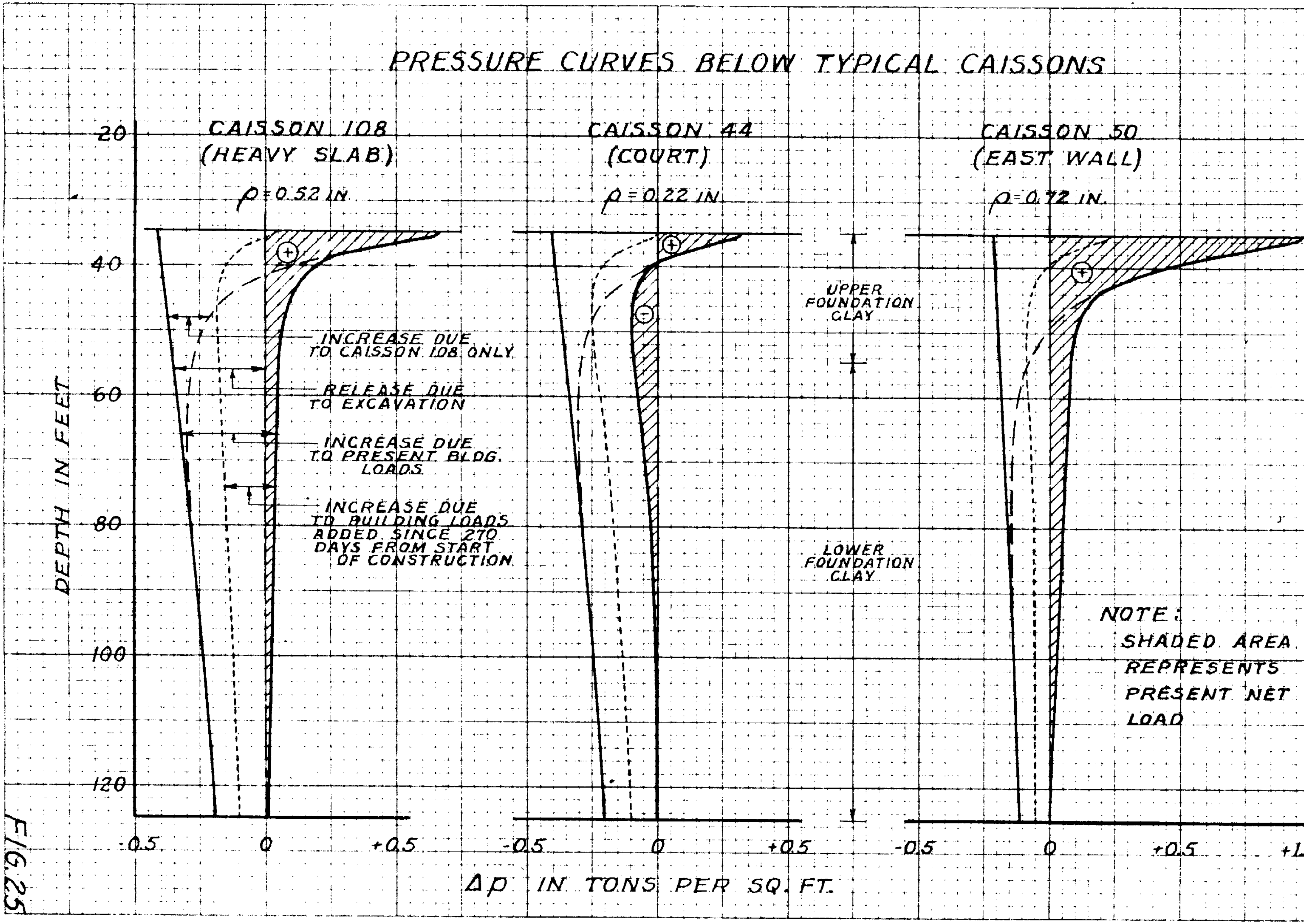
VERT. SCALE: $\frac{1}{2}$" = 10'

ASSUME AN AVERAGE NET EXCAVATION OVER THE ENTIRE BUILDING AREA OF 9.0 FT. @ 110 LBS PER CU FT.

ASSUME AN ADDITIONAL 2.3 FT. @ 60 LBS PER CU FT IN THE HEAVY SLAB ZONE.

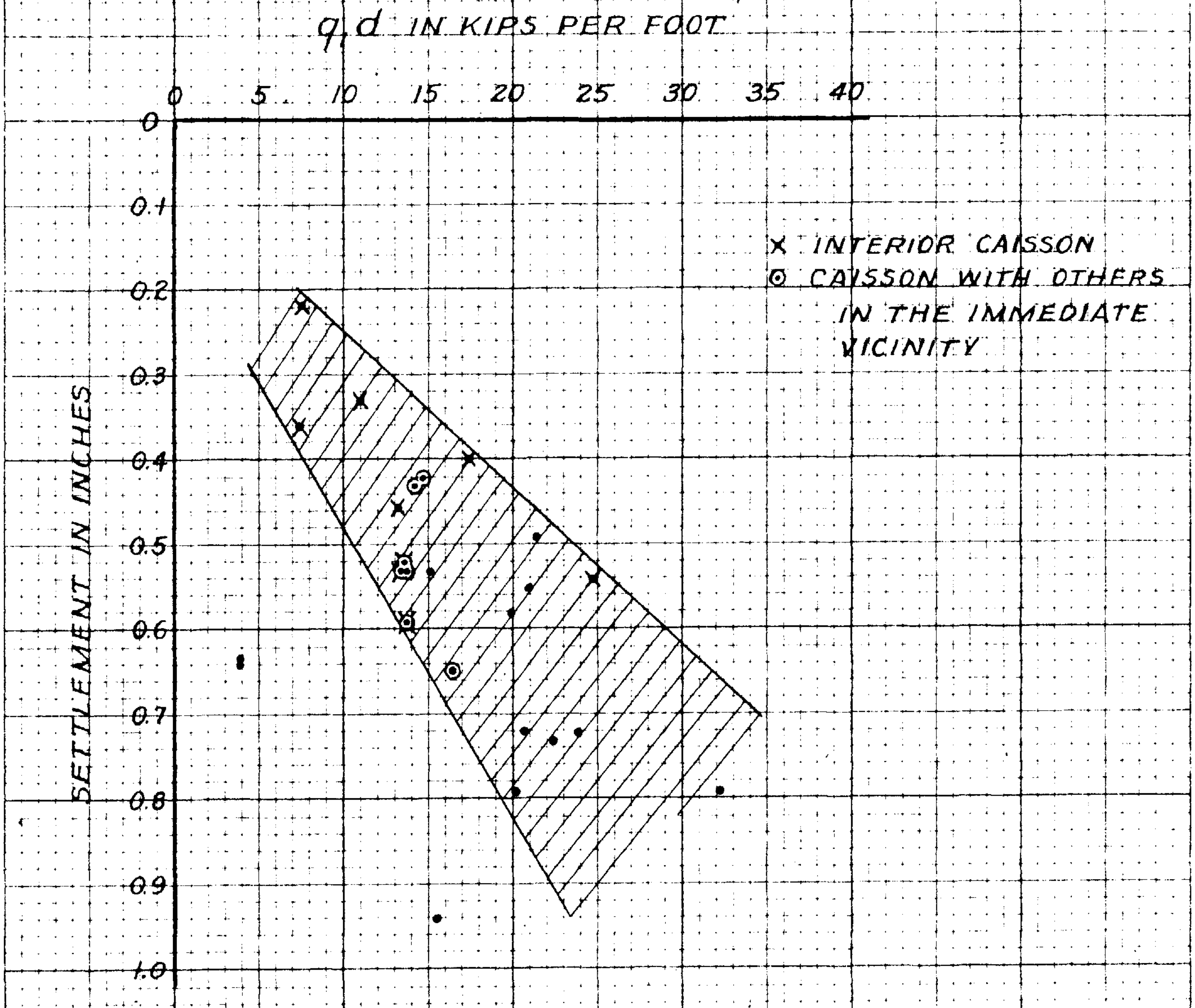
ASSUMED EXCAVATION AT LIBRARY

FIG. 24



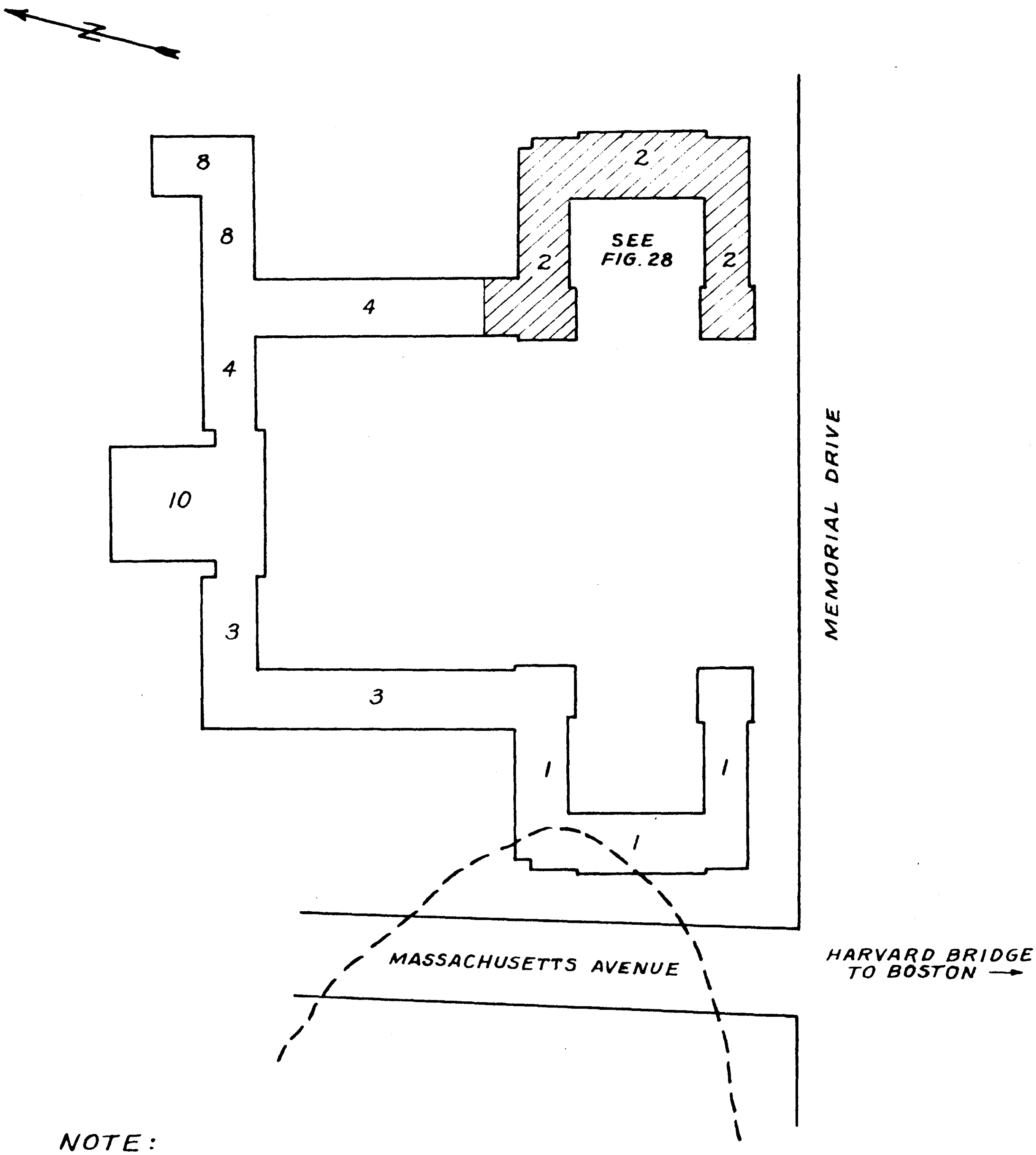
PRESSURE CURVES BELOW TYPICAL CAISSONS
CAISSON 108
(HEAVY SLAB)
ρ = 0.52 IN.
CAISSON 44
(COURT)
ρ = 0.22 IN.
CAISSON 50
(EAST WALL)
ρ = 0.72 IN.
INCREASE DUE TO CAISSON 108 ONLY
RELEASE DUE TO EXCAVATION
INCREASE DUE TO PRESENT BLDG. LOADS
INCREASE DUE TO BUILDING LOADS ADDED SINCE 270 DAYS FROM START OF CONSTRUCTION
UPPER FOUNDATION CLAY
LOWER FOUNDATION CLAY
NOTE: SHADED AREA REPRESENTS PRESENT NET LOAD
DEPTH IN FEET
20
40
60
80
100
120
-0.5
0
+0.5
+1.0
Δp IN TONS PER SQ. FT.

FIG. 25



RELATIONSHIP BETWEEN SETTLEMENT AND THE INDIVIDUAL CAISSON

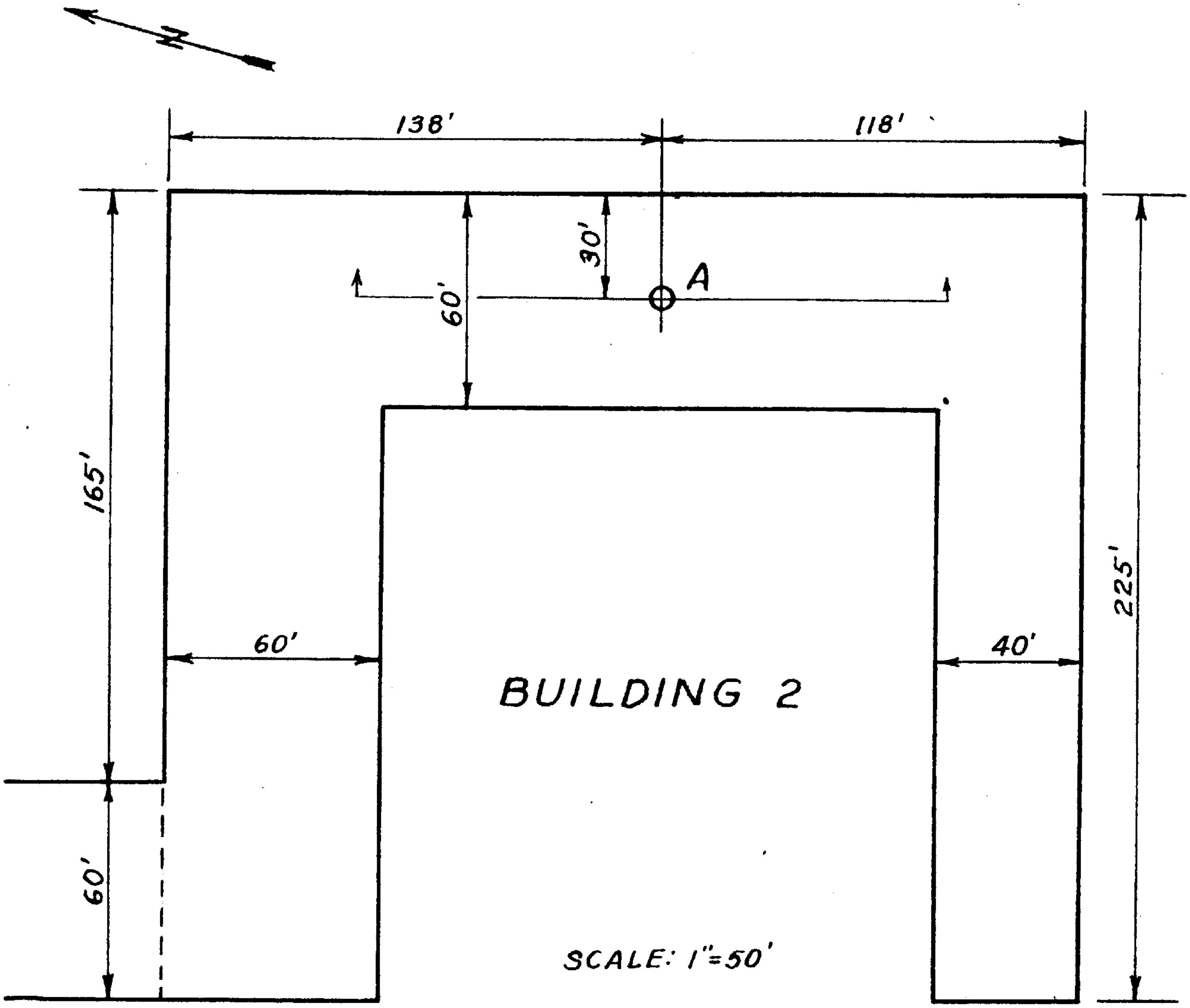
FIG. 26

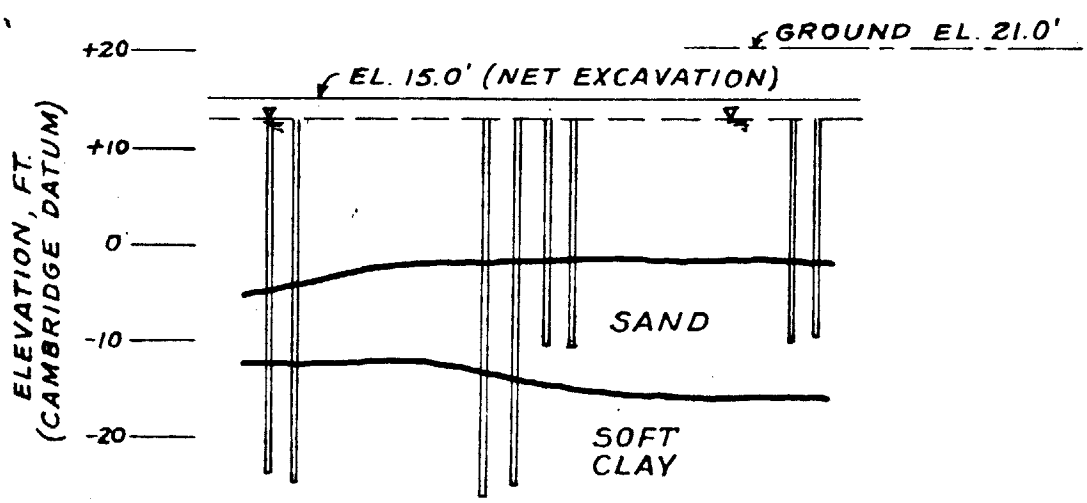


NOTE:
BUILDING NUMBERS ARE
THOSE CURRENTLY IN USE

MASS. INST. OF TECHNOLOGY
1916

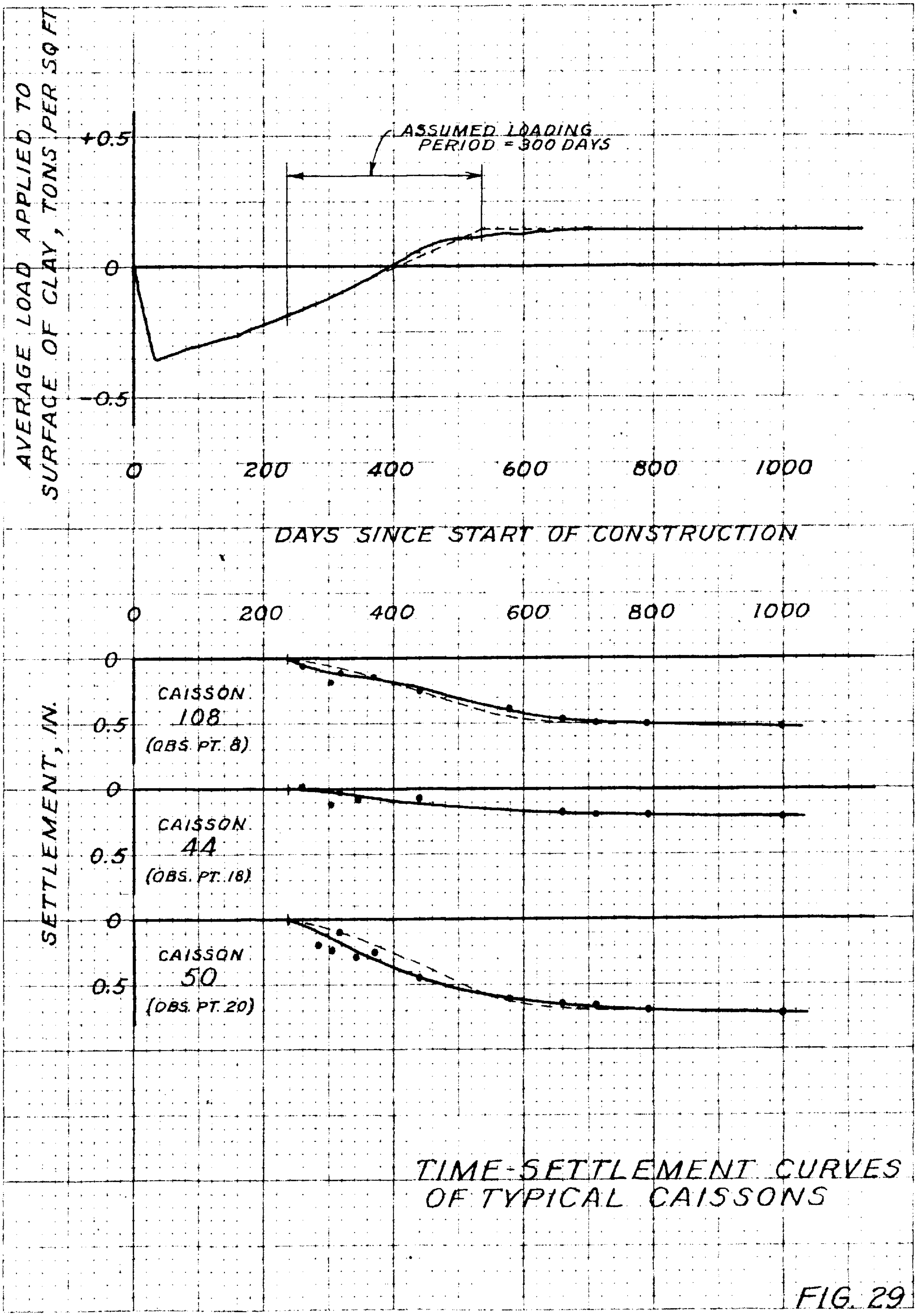
FIG. 27





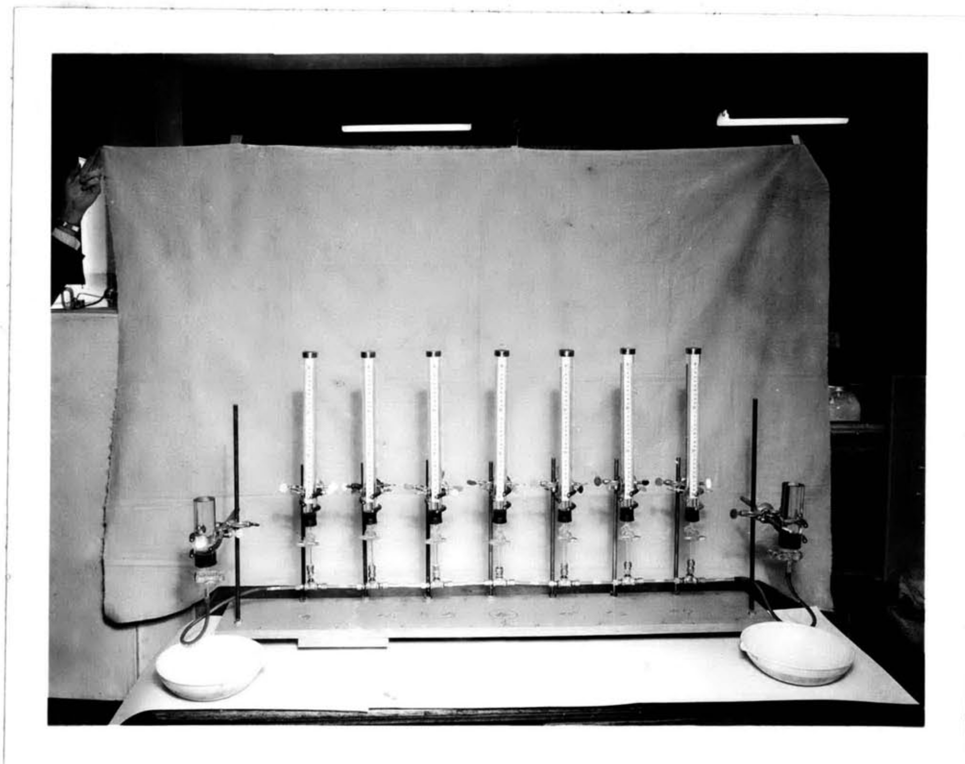
SECTION THROUGH POINT A

FIG. 28



TIME-SETTLEMENT CURVES OF TYPICAL CAISSONS

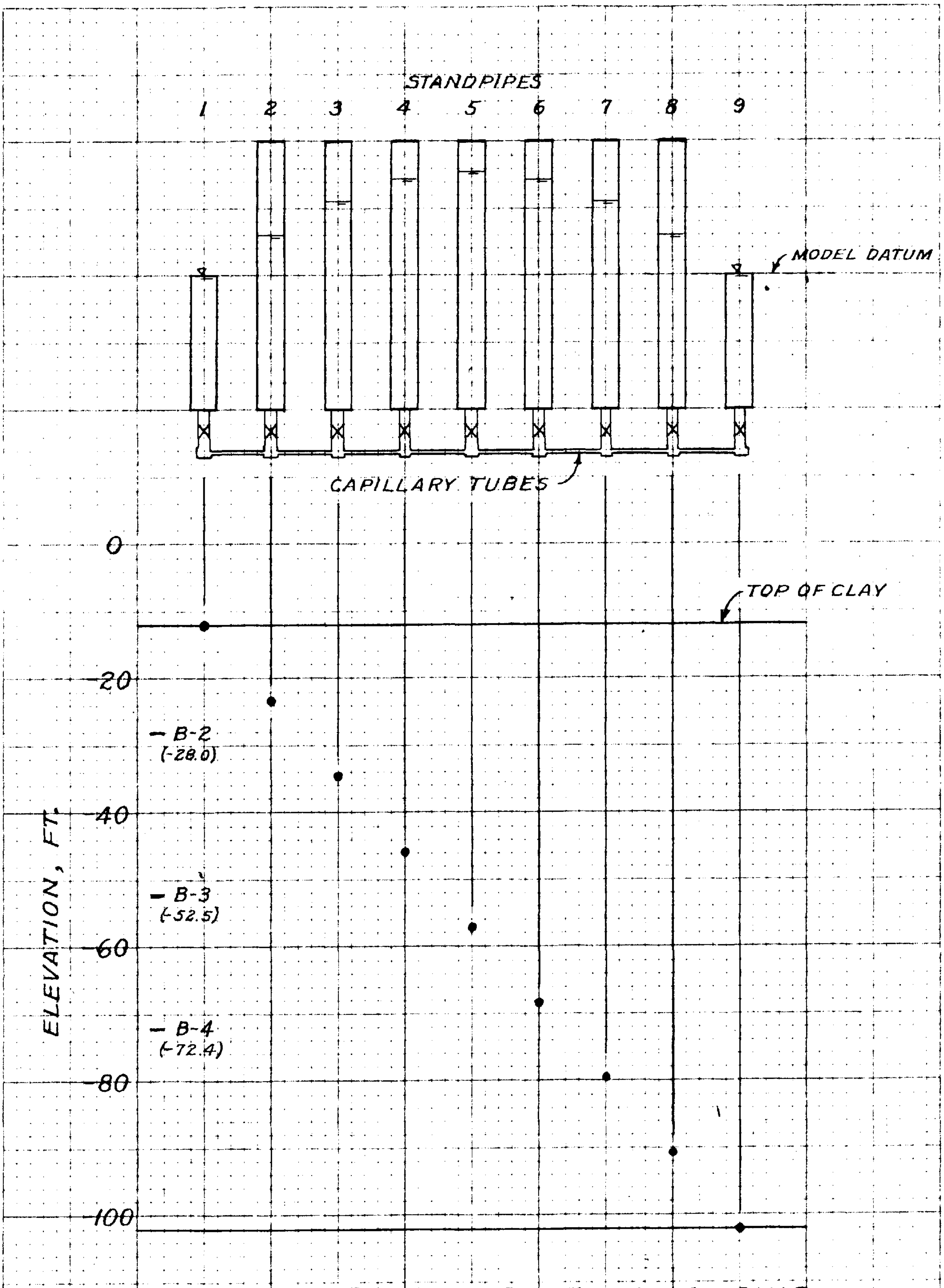
FIG. 29



CONSOLIDATION

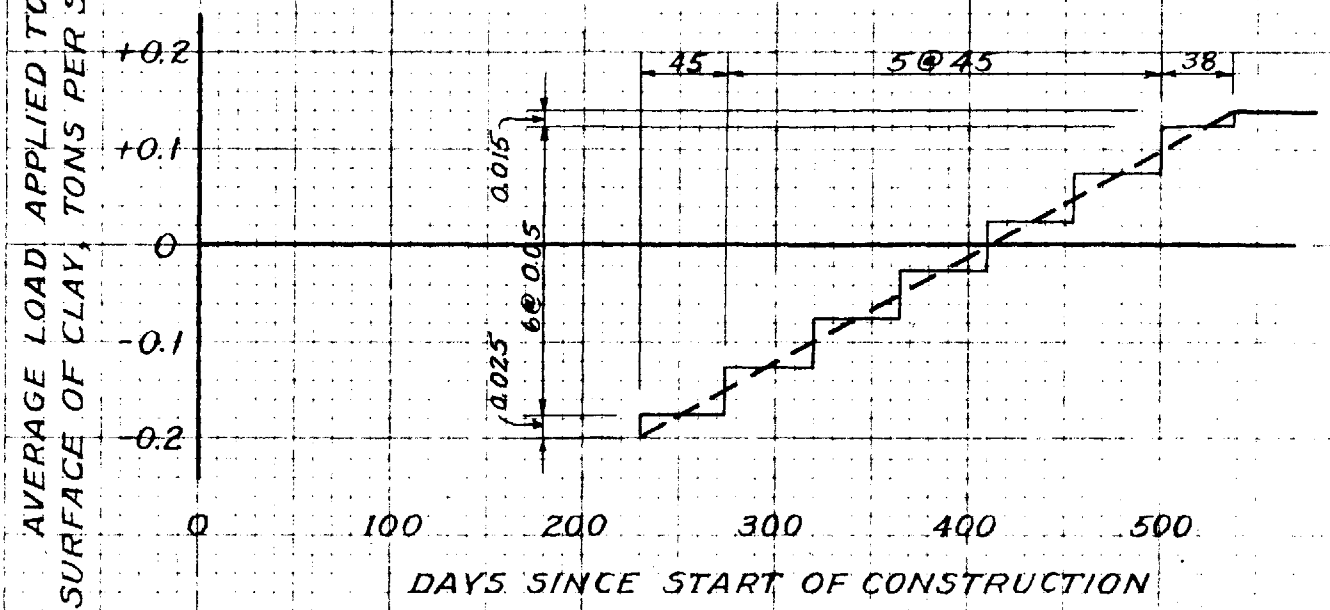
ANALOGY MODEL

FIG. 30



MODEL-PROTOTYPE RELATIONSHIP

FIG. 31



MODEL STANDPIPE	INITIAL EXCESS PRESSURE (1)	INFLUENCE VALUE OF EXCESS PRESSURE (2)
	FT.	FT.
1	0	-
2	-0.5	30
3	-1.2	27.5
4	-1.9	25
5	-2.1	23
6	-1.9	21
7	-1.7	19.5
8	-1.2	18
9	0	-

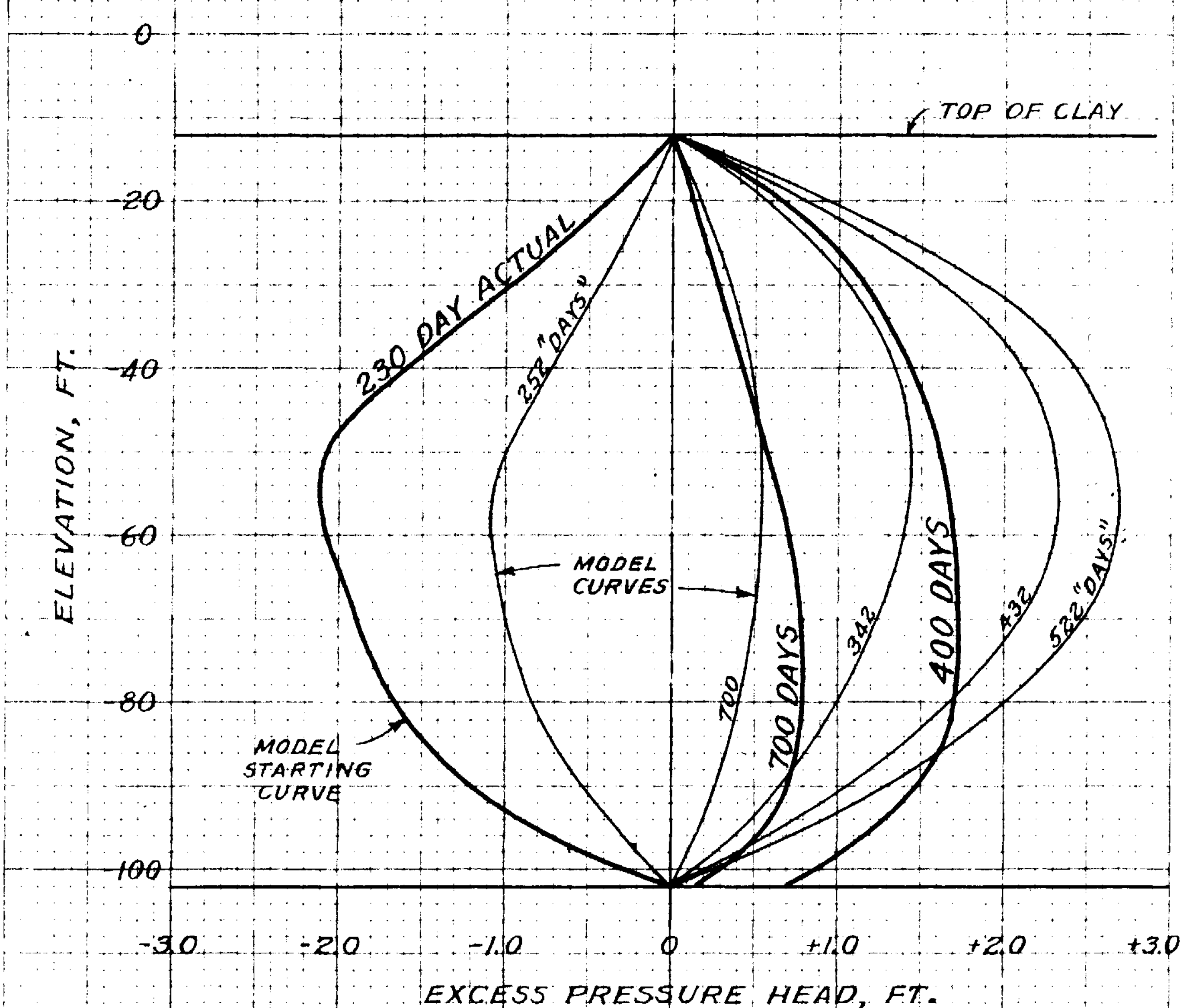
NOTE:
WHEN $c_m = 800 \times 10^{-4}$ CM.^{2} PER SEC. — t_m (MIN.) = 0259 t_p (DAYS)

(1) FROM FIGURE 17 AT 270 DAYS.

(2) EQUIVALENT EXCESS PRESSURE HEAD CAUSED BY A LOAD OF 1 TON PER PER SQ. FT. APPLIED OVER THE BUILDING AREA

MODEL STUDY
CASE 2

FIG. 32



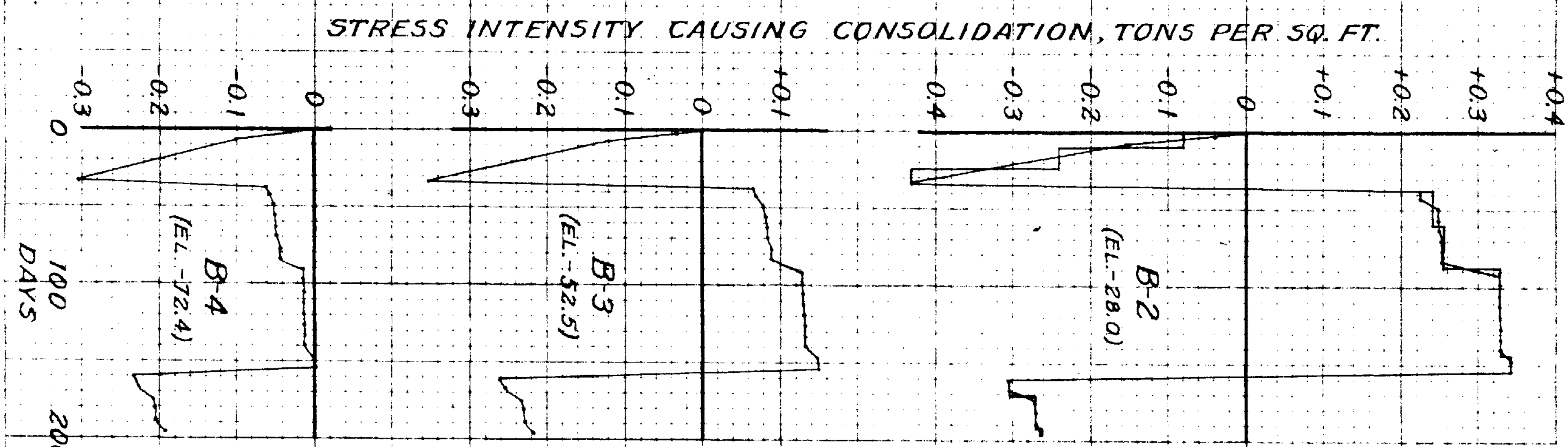
NOTE:

MODEL CURVES FOR $C_m = 800 \times 10^{-4}$ CM^2 PER SEC AND $t = 25.0°C$

SEE FIGURE 17 FOR ACTUAL PIEZOMETER CURVES (GROUP B)

PIEZOMETER CURVES
CASE 2

FIG. 33

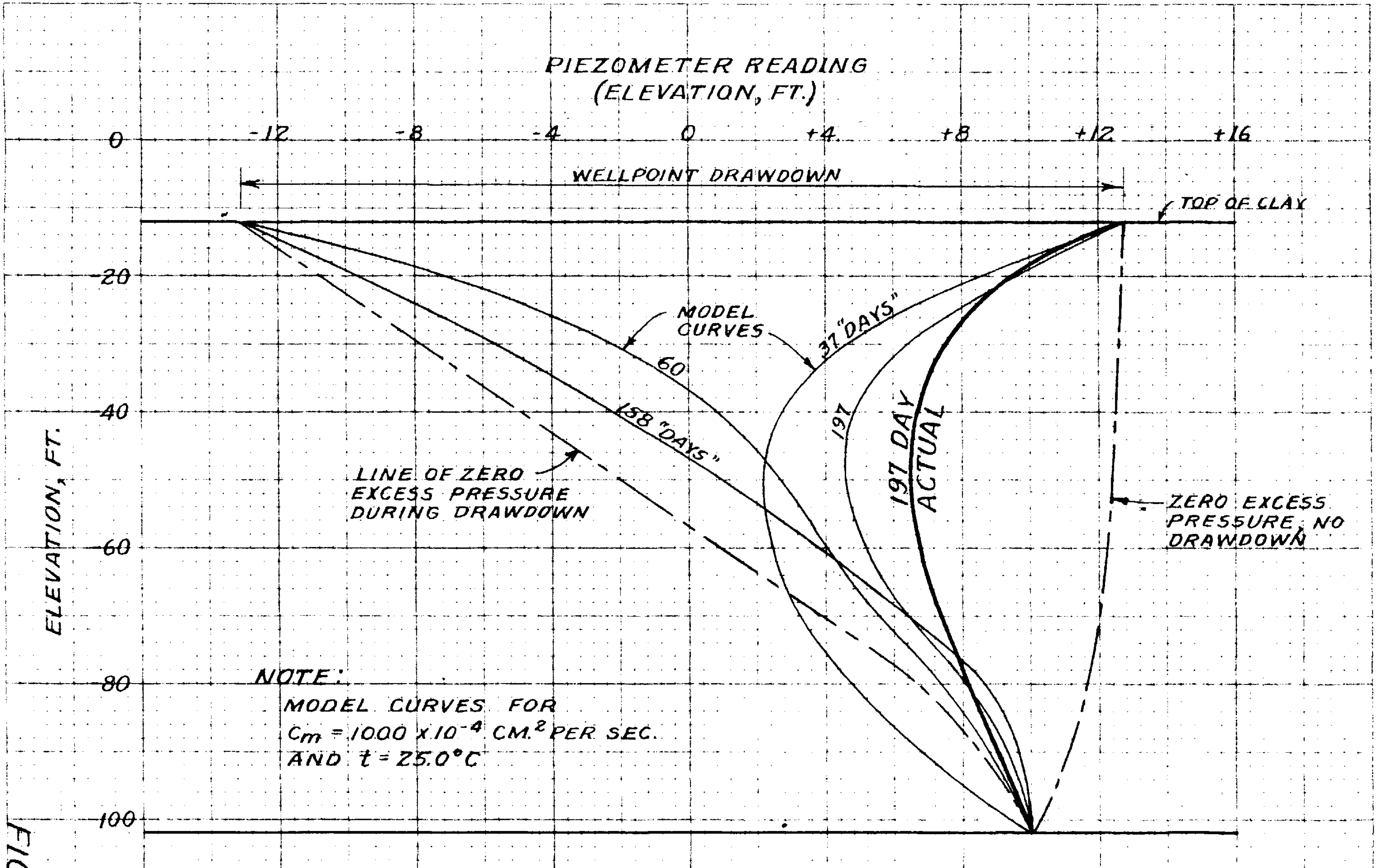


DAYS	MODEL TIME (1)	INCREMENT OF EXCESS PRESSURE HEAD, FT. STANDPIPE 1	2	3	4	5	6	7	8	9
0	0^{m} 00^{s}	-	-2.6	-2.6	-2.5	-2.4	-2.4	-2.2	-2.1	-
10	3 14	-	-5.4	-4.6	-3.9	-3.4	-3.0	-2.7	-2.4	-
23	7 27	-	-6.4	-5.9	-5.4	-5.0	-4.6	-4.3	-4.1	-
37	12 00	-	+23.0	+19.5	+15.7	+12.5	+9.2	+6.0	+3.1	-
60	19 30	-	+0.6	+0.5	+0.5	+0.4	+0.3	+0.2	+0.1	-
88	28 36	-	+2.4	+2.0	+1.5	+1.2	+1.0	+0.8	+0.6	-
144	46 45	-	+0.7	+0.7	+0.6	+0.6	+0.5	+0.4	+0.4	-
158	51 18	-	-22.5	-19.0	-15.5	-12.0	-8.7	-5.4	-2.7	-
172	55 48	-	+1.0	+1.1	+1.1	+1.0	+0.8	+0.6	+0.5	-
192	62 21	-	+0.3	+0.3	+0.4	+0.4	+0.4	+0.4	+0.4	-
197	63 54	-	-	-	-	-	-	-	-	-

(1) FOR $C_m = 1000 \times 10^{-4}$ CM^2 PER SEC.

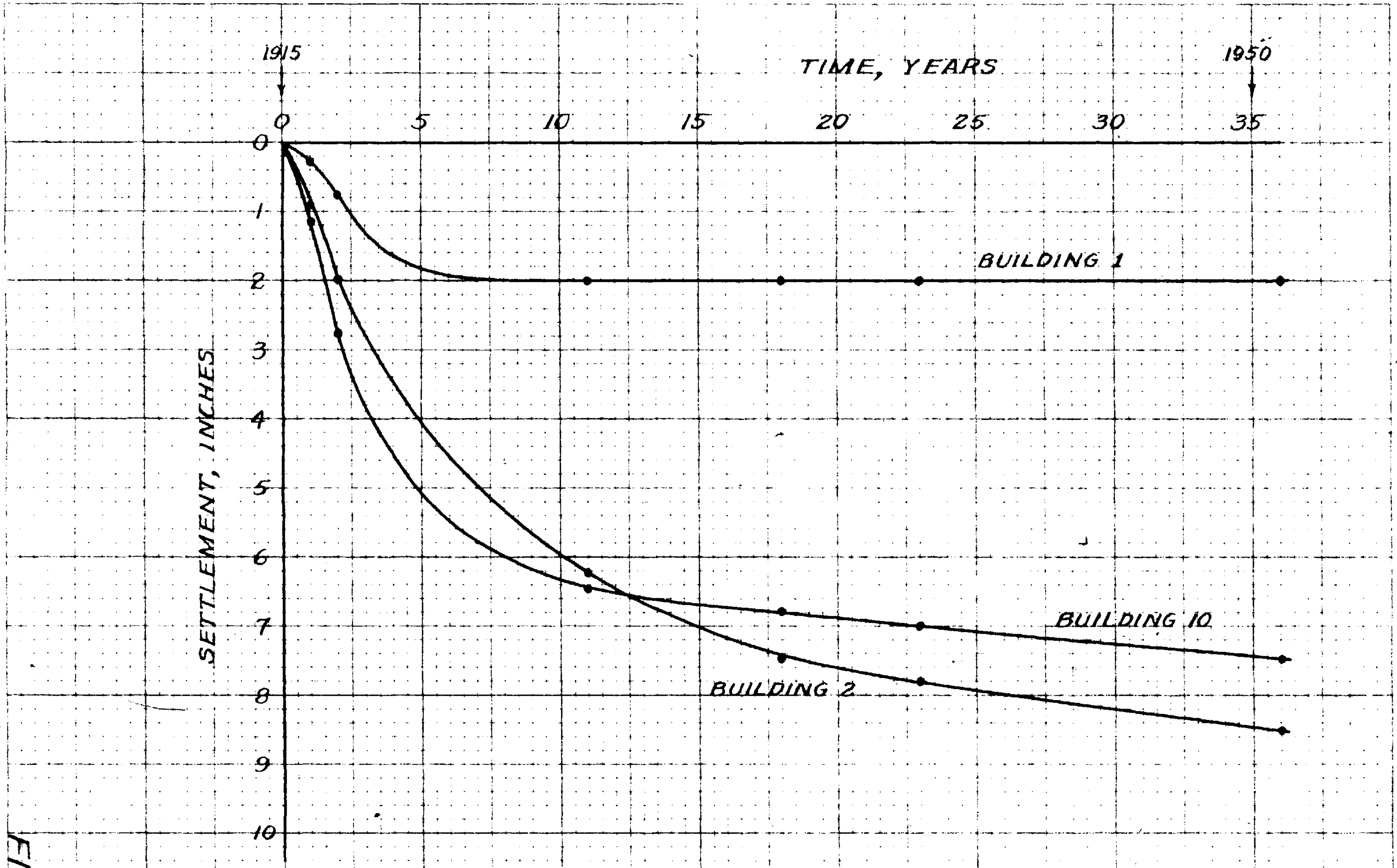
MODEL STUDY
CASE 3

FIG. 34



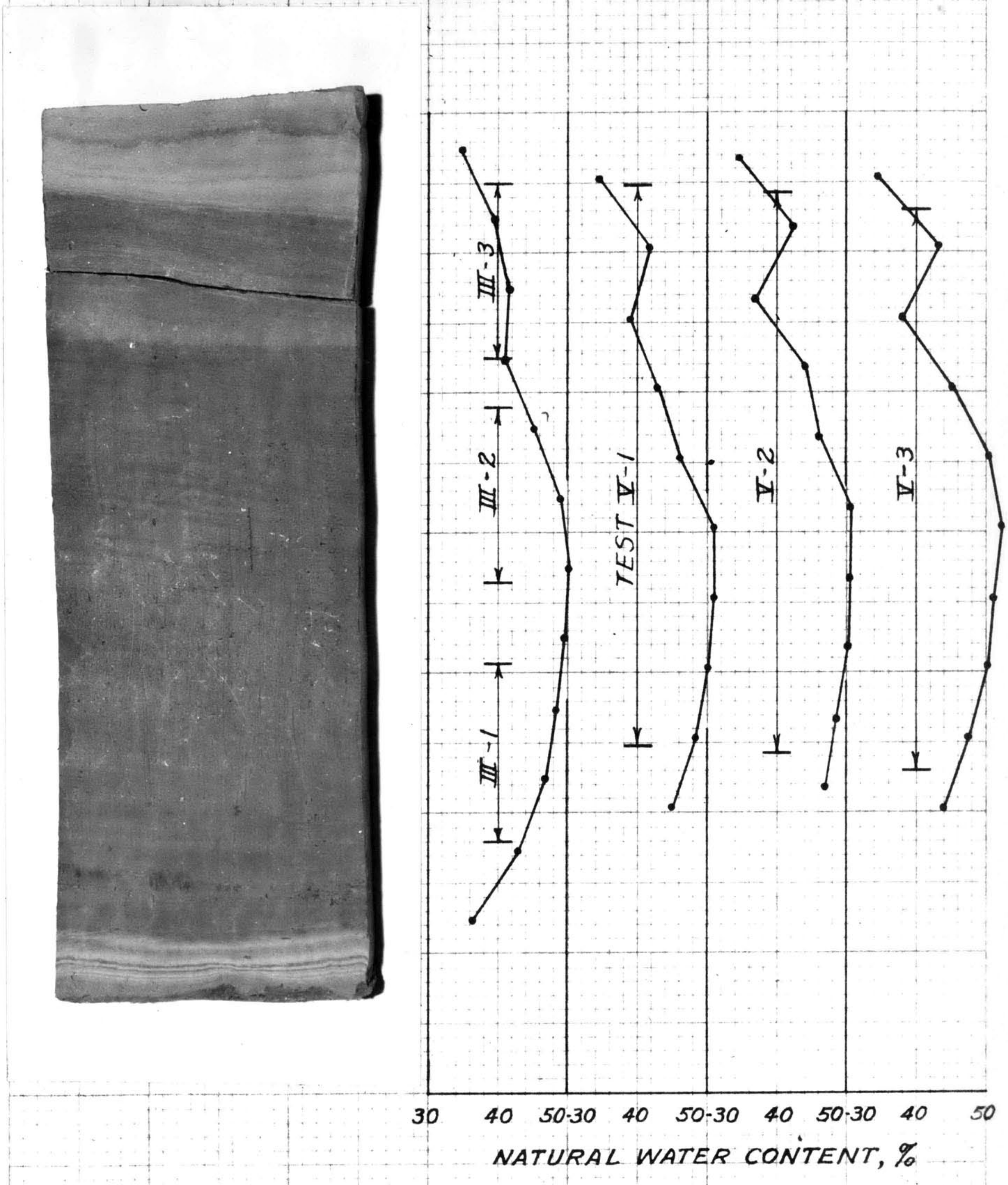
PIEZOMETER CURVES
CASE 3

FIG. 35



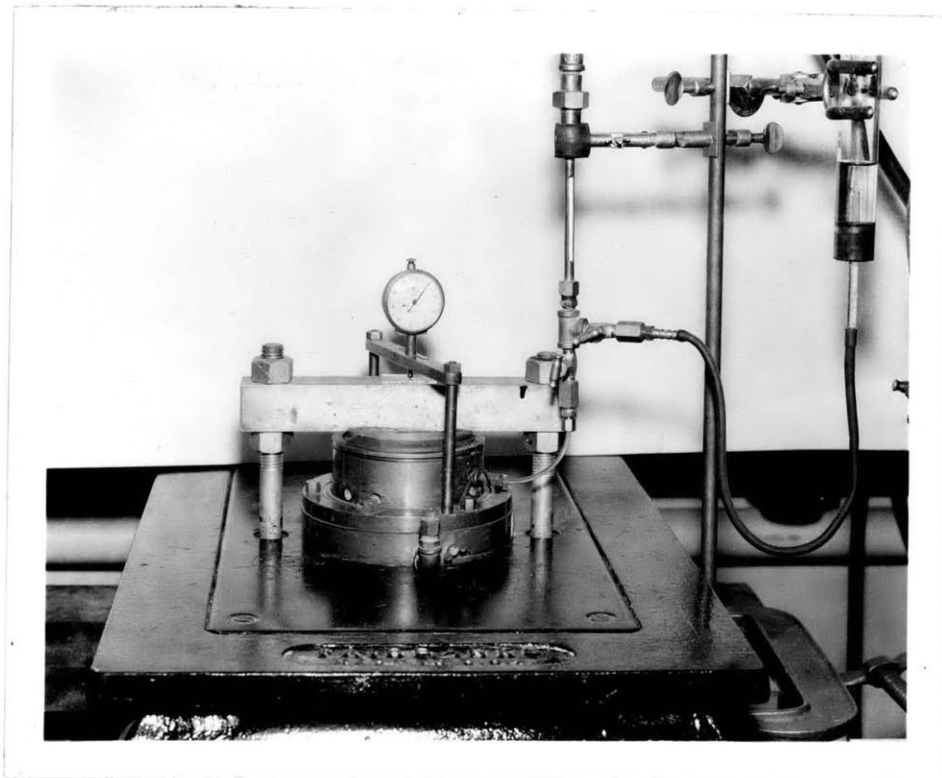
TIME-SETTLEMENT CURVES
MAIN M.I.T. BUILDING

FIG. 36

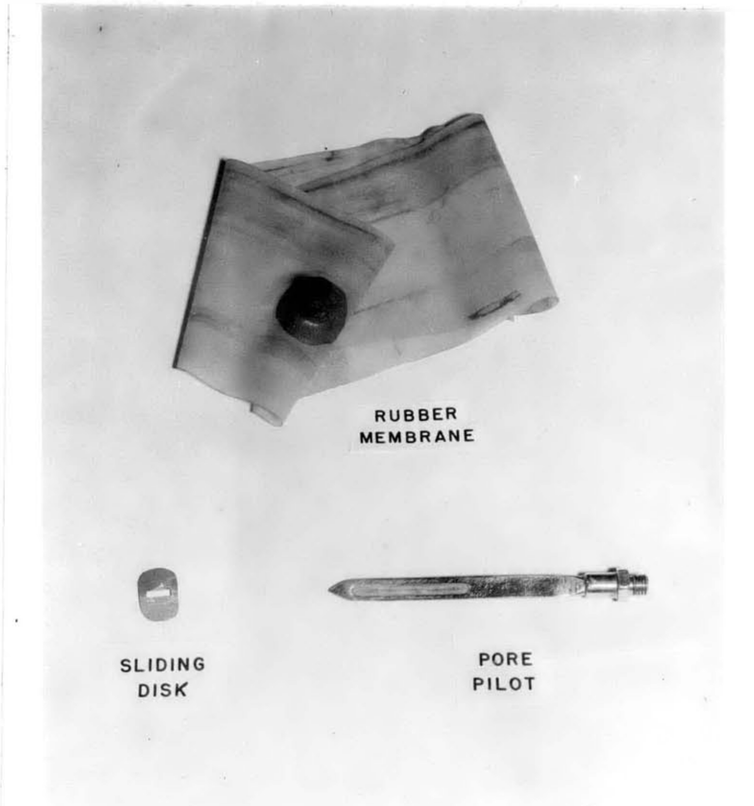


CLAY USED FOR CONSOLIDATION-
PORE PRESSURE RESEARCH

FIG. 37



(a)



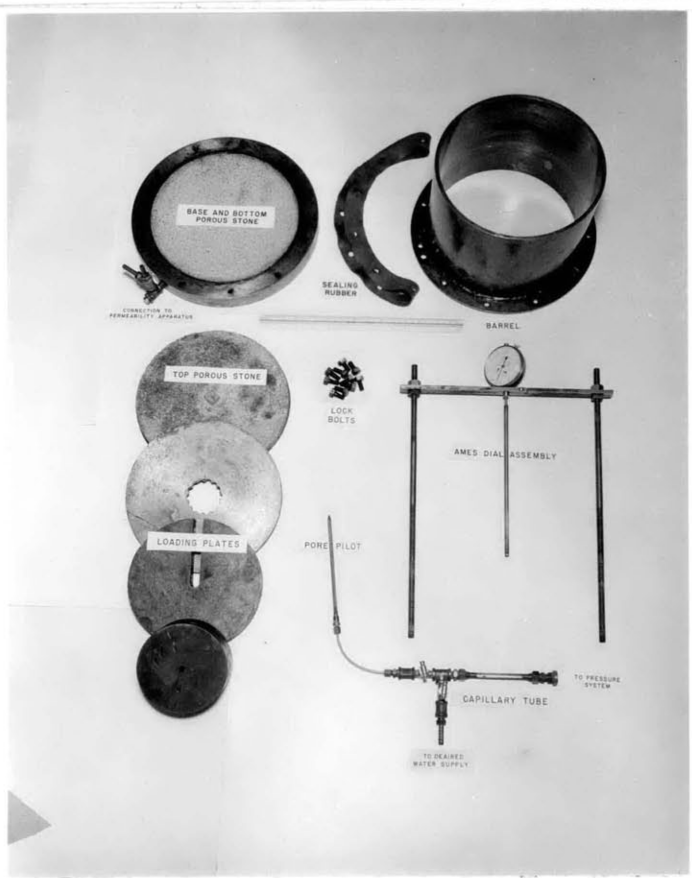
(b)

CONSOLIDATION APPARATUS

SET III TESTS

FIG. 38

(a)



(b)

CONSOLIDATION APPARATUS

SET V TESTS

FIG. 39

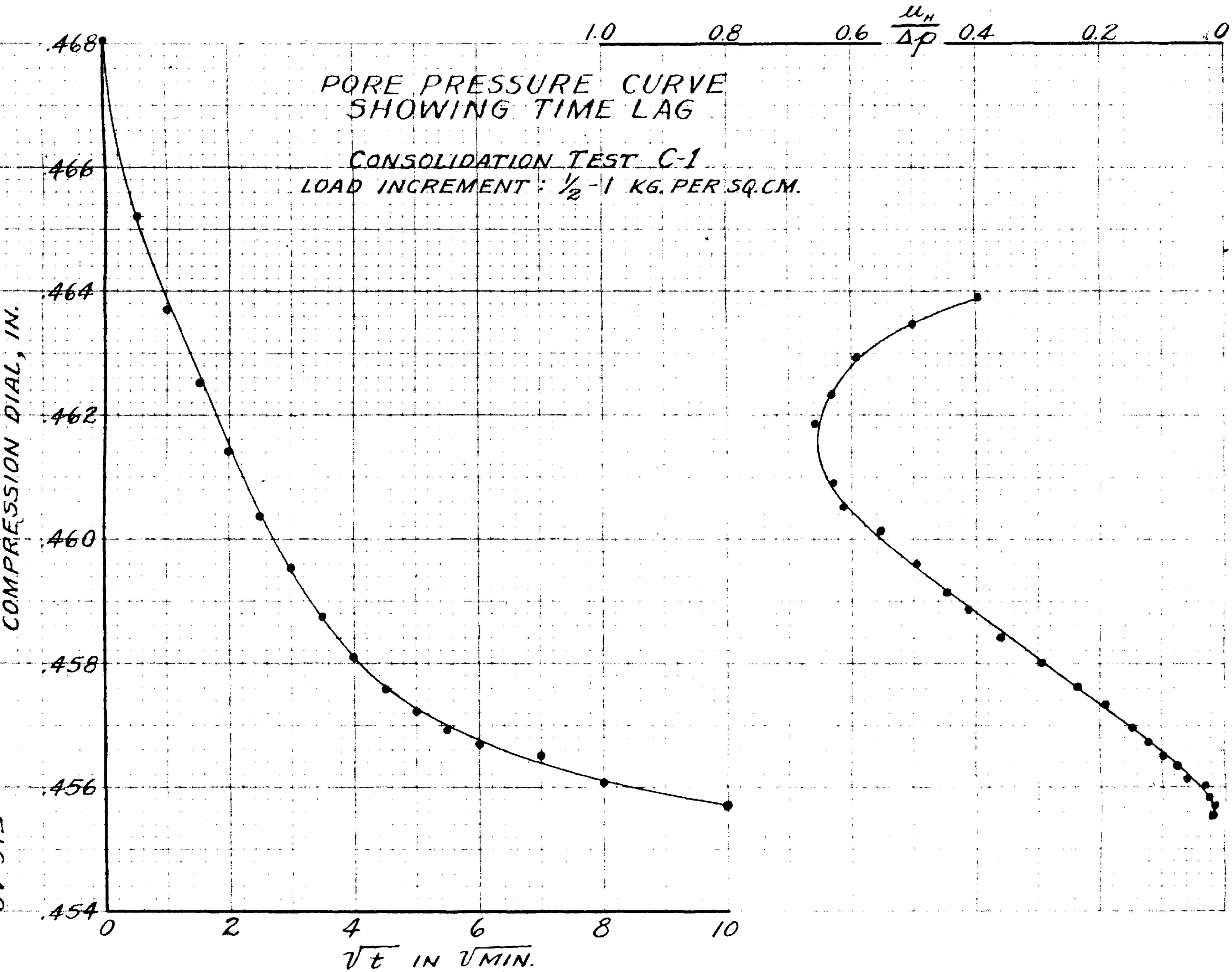

PORE PRESSURE CURVE
SHOWING TIME LAG
CONSOLIDATION TEST C-1
LOAD INCREMENT: ½-1 KG. PER SQ.CM.
COMPRESSION DIAL, IN.
.468
.466
.464
.462
.460
.458
.456
.454
0
2
4
6
8
10
√t IN √MIN.
$\frac{u_H}{\Delta p}$
1.0
0.8
0.6
0.4
0.2
0

FIG. 40

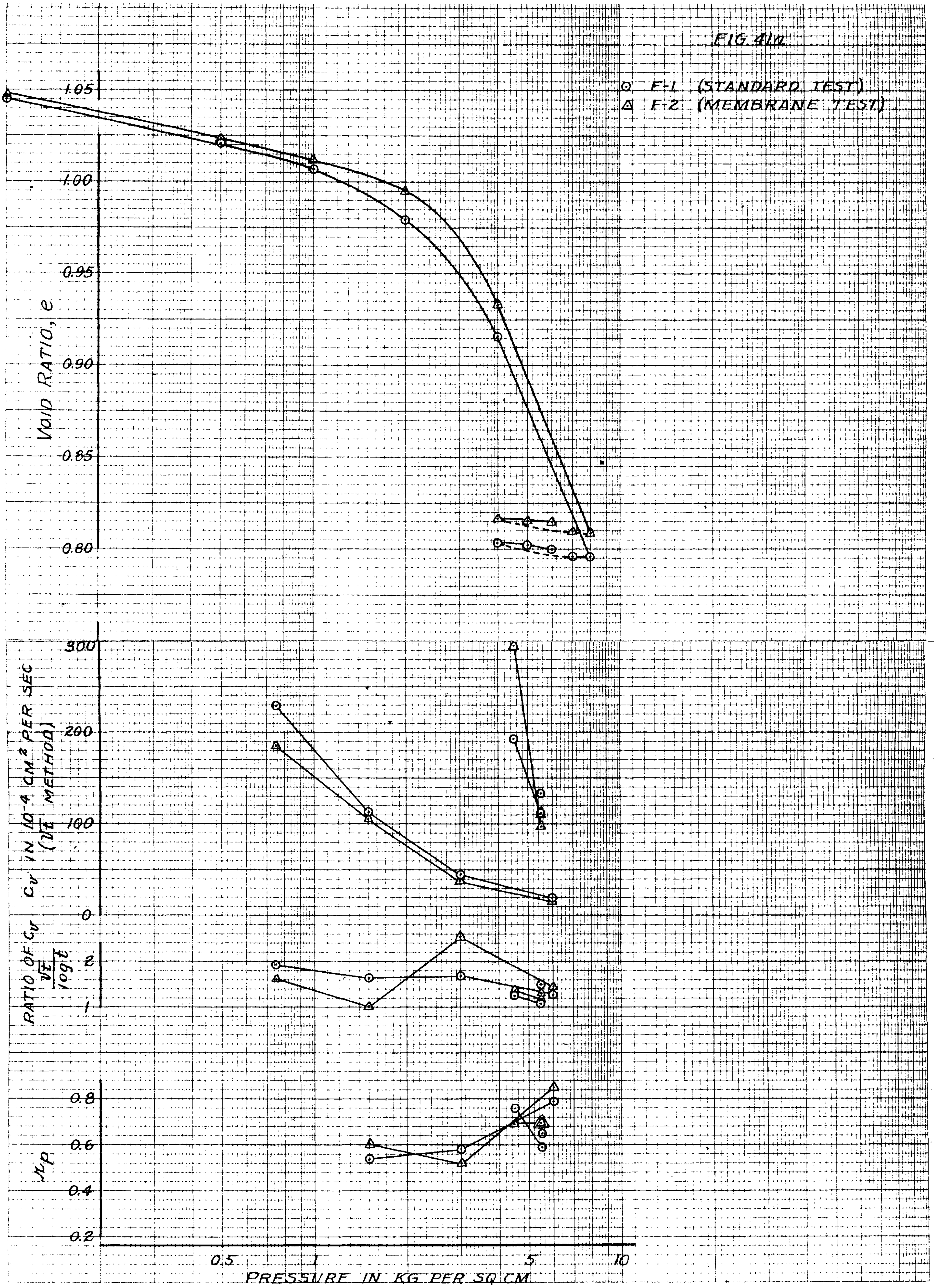


FIG. 41a
⊙ F-1 (STANDARD TEST)
△ F-2 (MEMBRANE TEST)
VOID RATIO, e
1.05
1.00
0.95
0.90
0.85
0.80
Cv IN 10-4 CM2 PER SEC
(√t METHOD)
300
200
100
0
RATIO OF Cv √t / log t
2
1
0.8
0.6
0.4
0.2
0.5
1
5
10
PRESSURE IN KG PER SQ CM

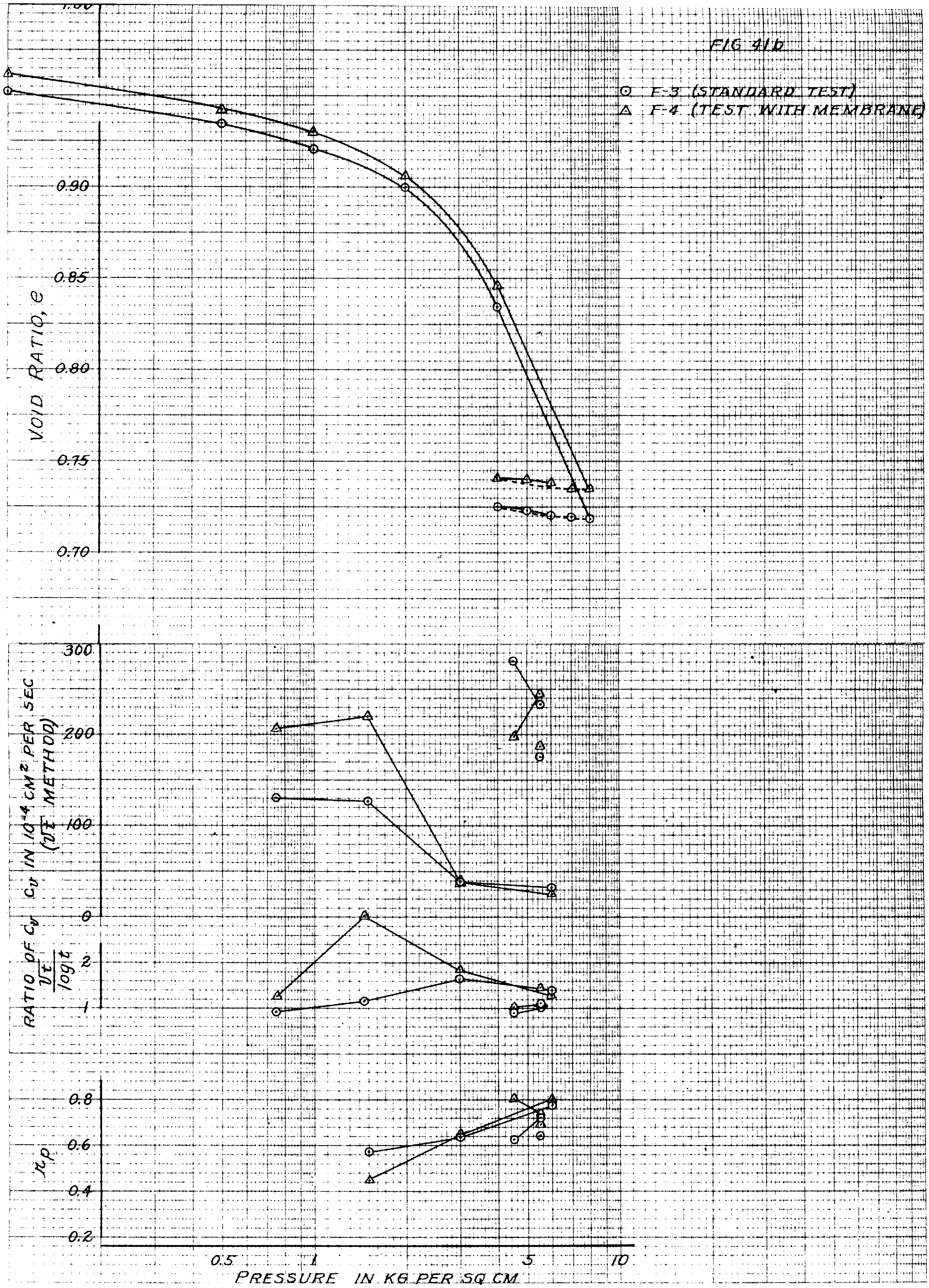


FIG 41b
F-3 (STANDARD TEST)
F-4 (TEST WITH MEMBRANE)
VOID RATIO, e
0.90
0.85
0.80
0.75
0.70
RATIO OF c_v c_v IN 10^{-4} CM^2 PER SEC ($\sqrt{t}$ METHOD)
300
200
100
0
$\frac{\sqrt{t}}{\log t}$
2
1
r_p
0.8
0.6
0.4
0.2
0.5
1
5
10
PRESSURE IN KG PER SQ CM

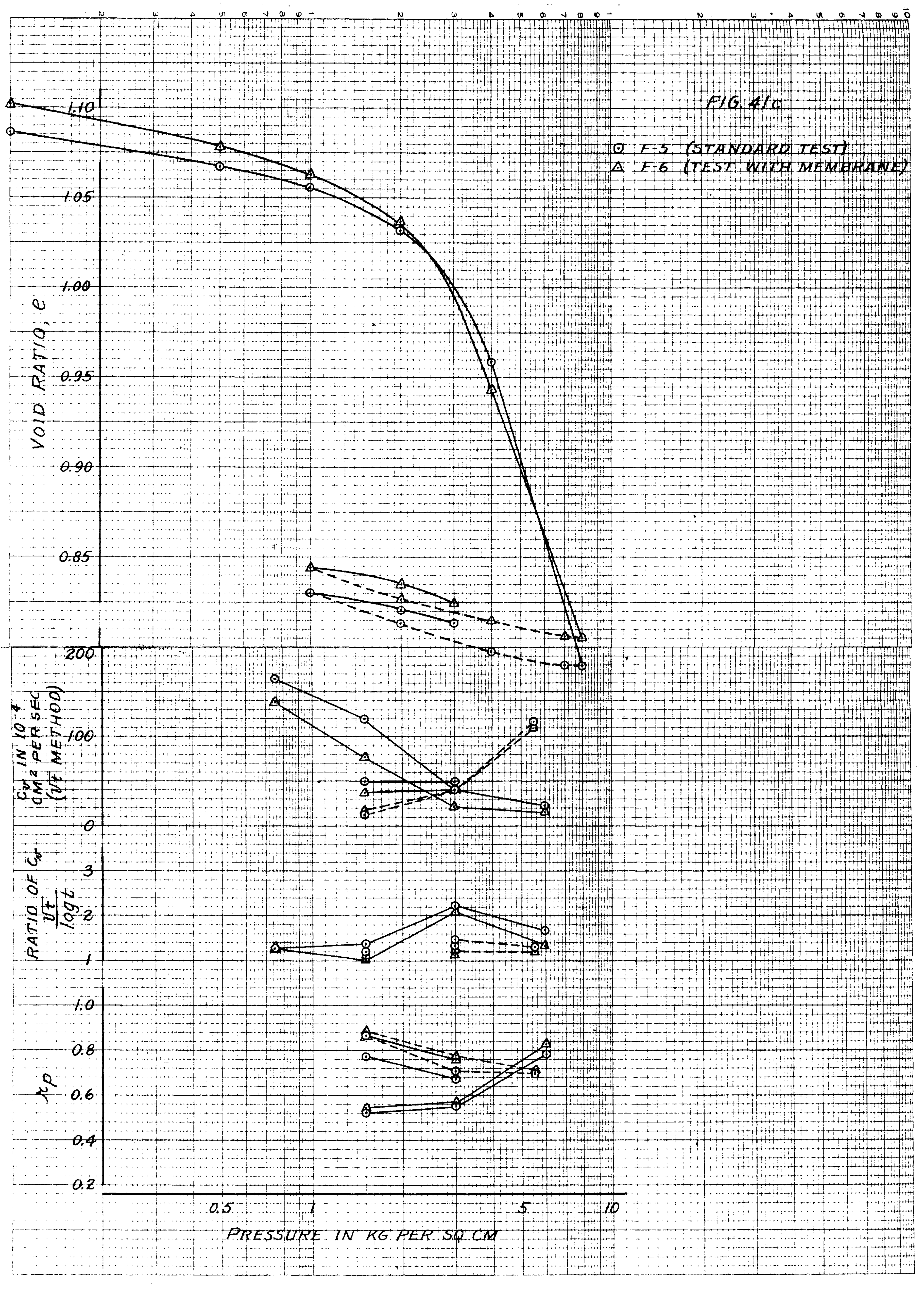


FIG. 41c
⊙ F-5 (STANDARD TEST)
△ F-6 (TEST WITH MEMBRANE)
VOID RATIO, e
1.10
1.05
1.00
0.95
0.90
0.85
200
100
0
C_v IN 10^{-4} CM^2 PER SEC ($\sqrt{t}$ METHOD)
RATIO OF C_v $\frac{\sqrt{t}}{\log t}$
3
2
1
r_p
1.0
0.8
0.6
0.4
0.2
0.5
1
5
10
PRESSURE IN KG PER SQ. CM

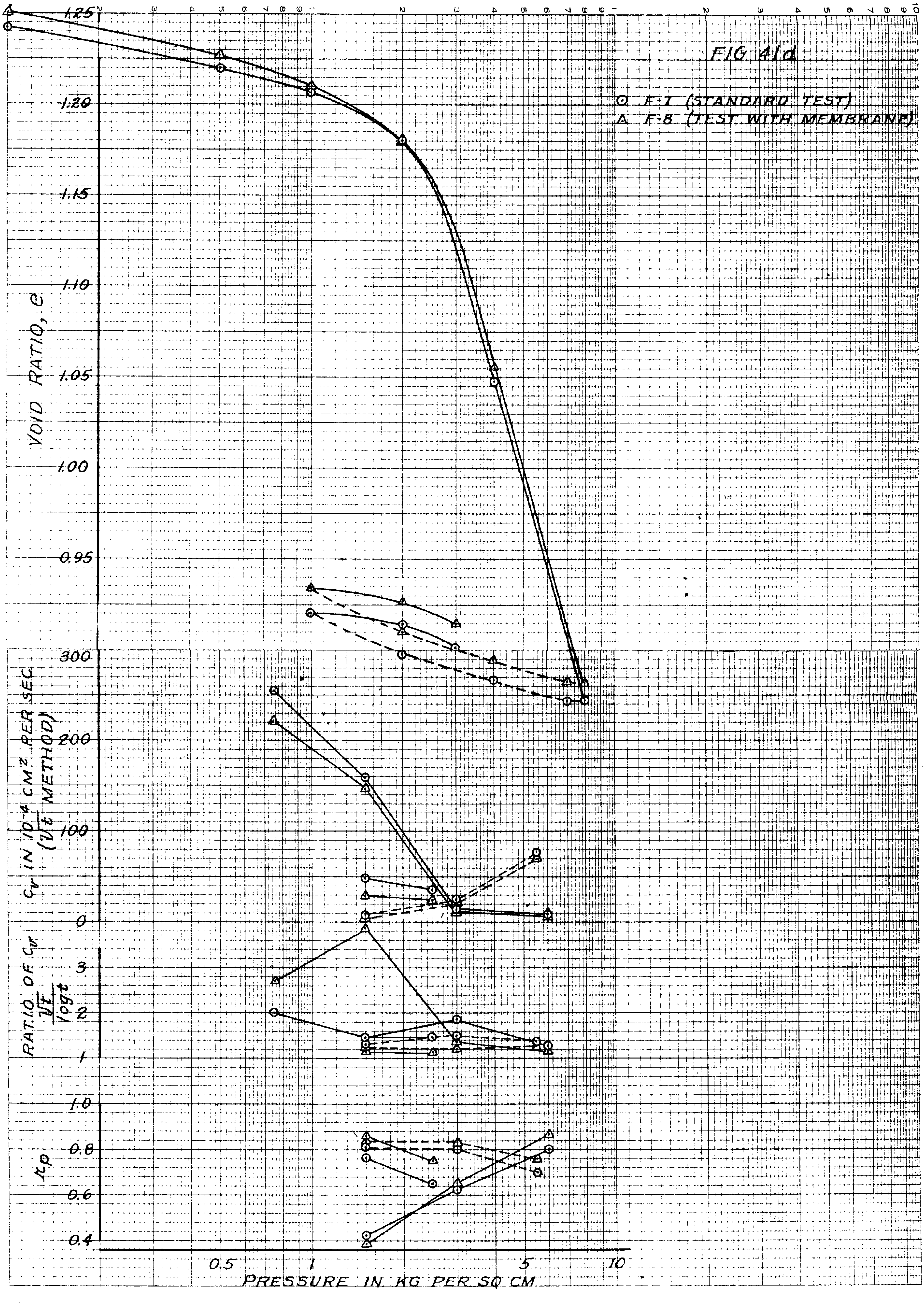


FIG 41d
⊙ F-7 (STANDARD TEST)
△ F-8 (TEST WITH MEMBRANE)
VOID RATIO, e
1.25
1.20
1.15
1.10
1.05
1.00
0.95
c_v IN 10^{-4} CM^2 PER SEC ($\sqrt{t}$ METHOD)
300
200
100
0
RATIO OF c_v $\frac{\sqrt{t}}{\log t}$
3
2
1
r_p
1.0
0.8
0.6
0.4
0.5
1
5
10
PRESSURE IN KG PER SQ CM

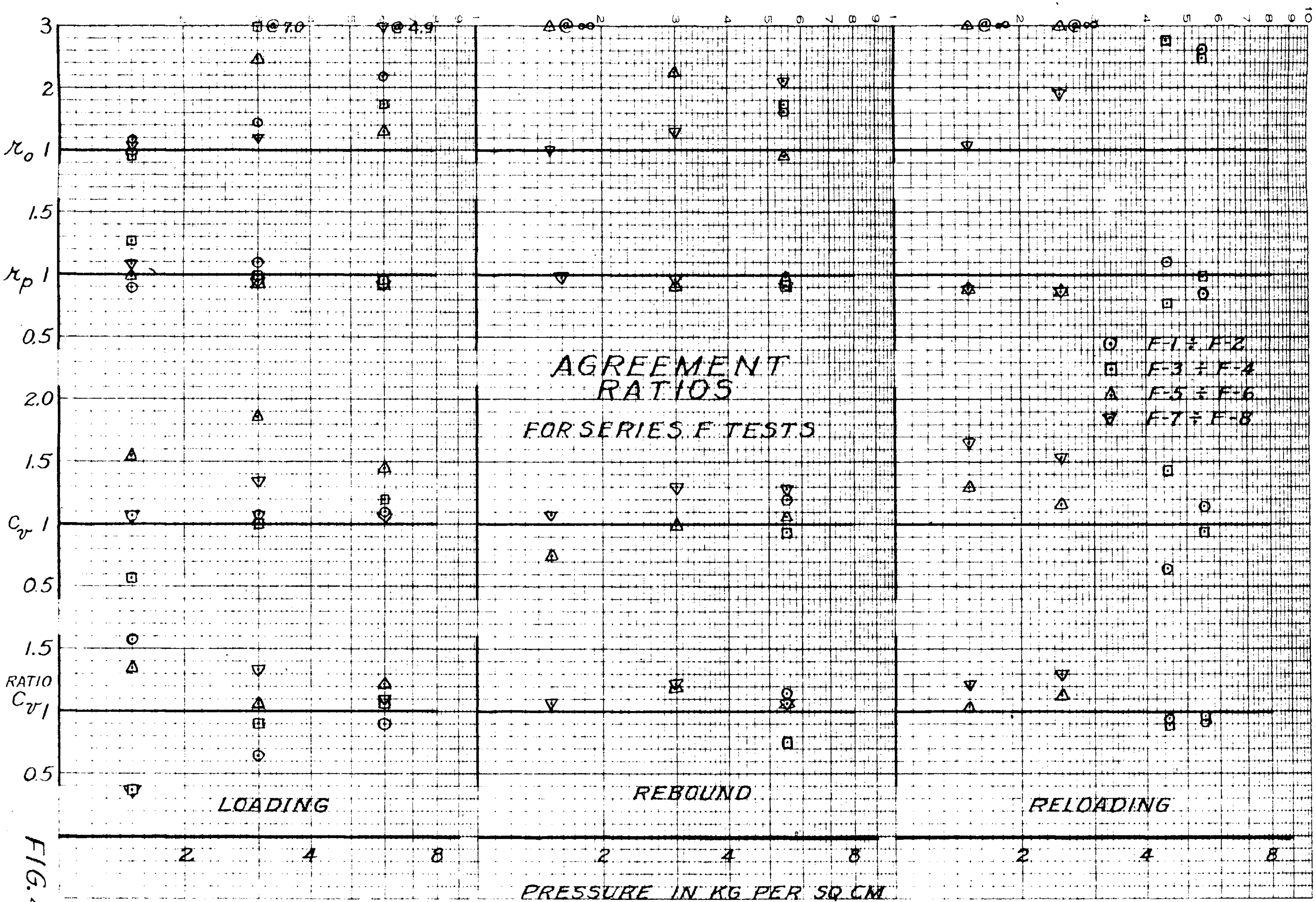
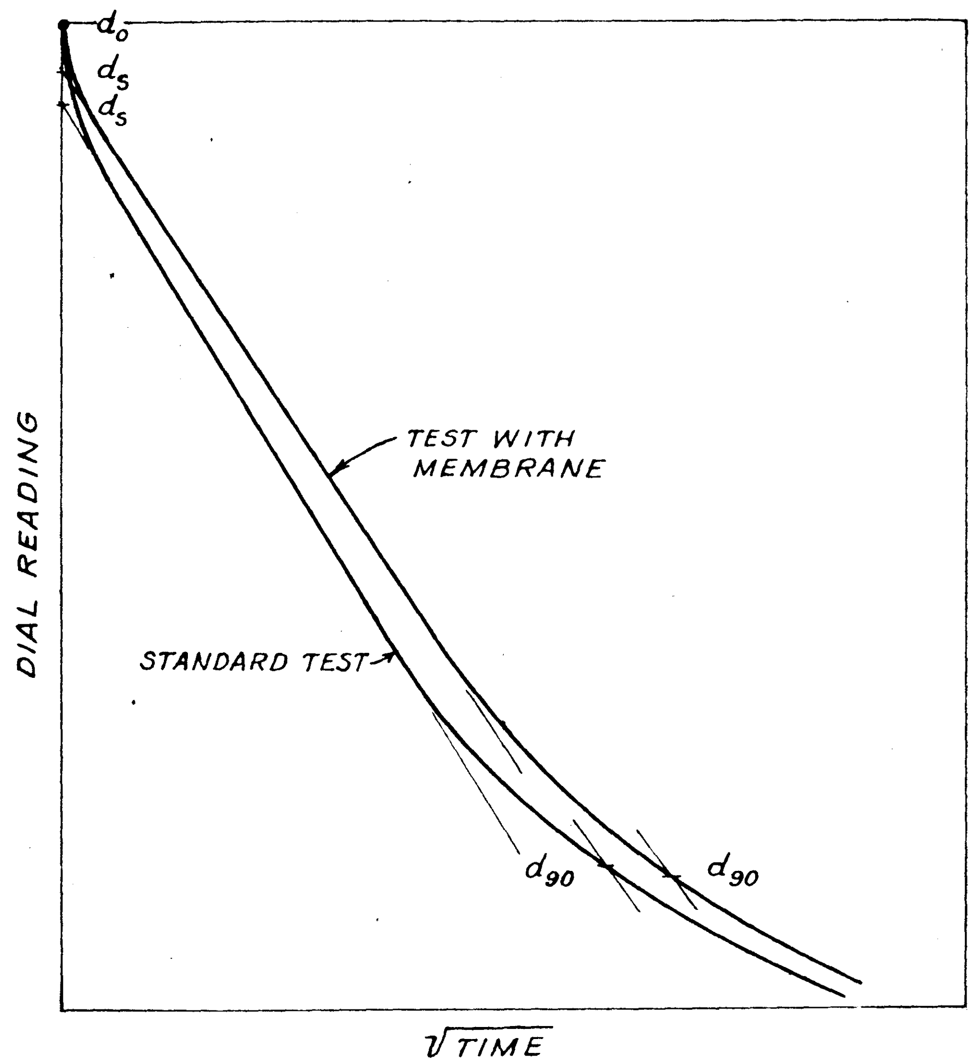

AGREEMENT RATIOS
FOR SERIES F TESTS
F-1 ÷ F-2
F-3 ÷ F-4
F-5 ÷ F-6
F-7 ÷ F-8
r_o
r_p
C_v
RATIO C_v
@7.0
@4.9
@∞
LOADING
REBOUND
RELOADING
PRESSURE IN KG PER SQ CM

FIG. 42



COMPARISON OF
COMPRESSION CURVES

FIG. 43

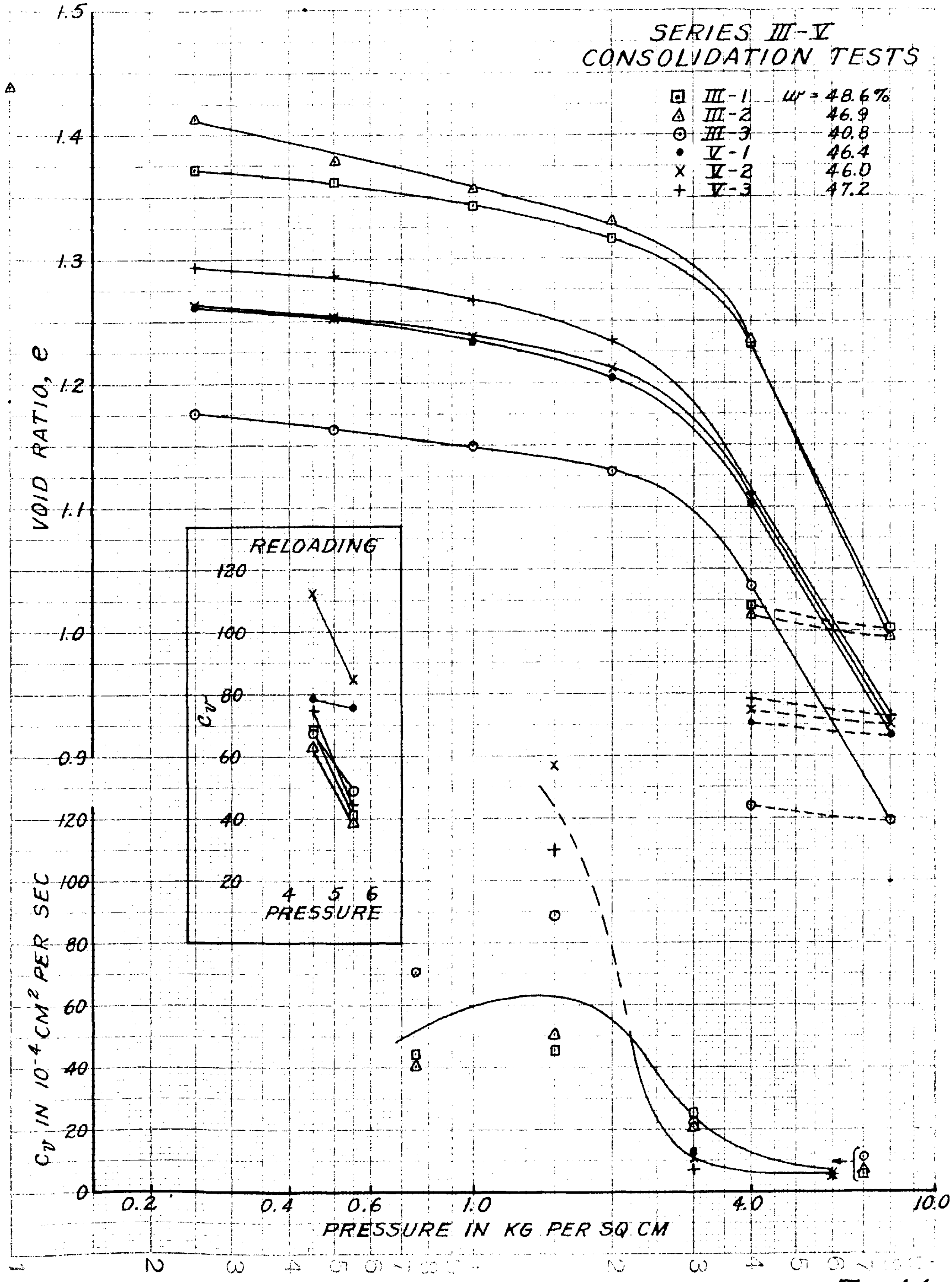
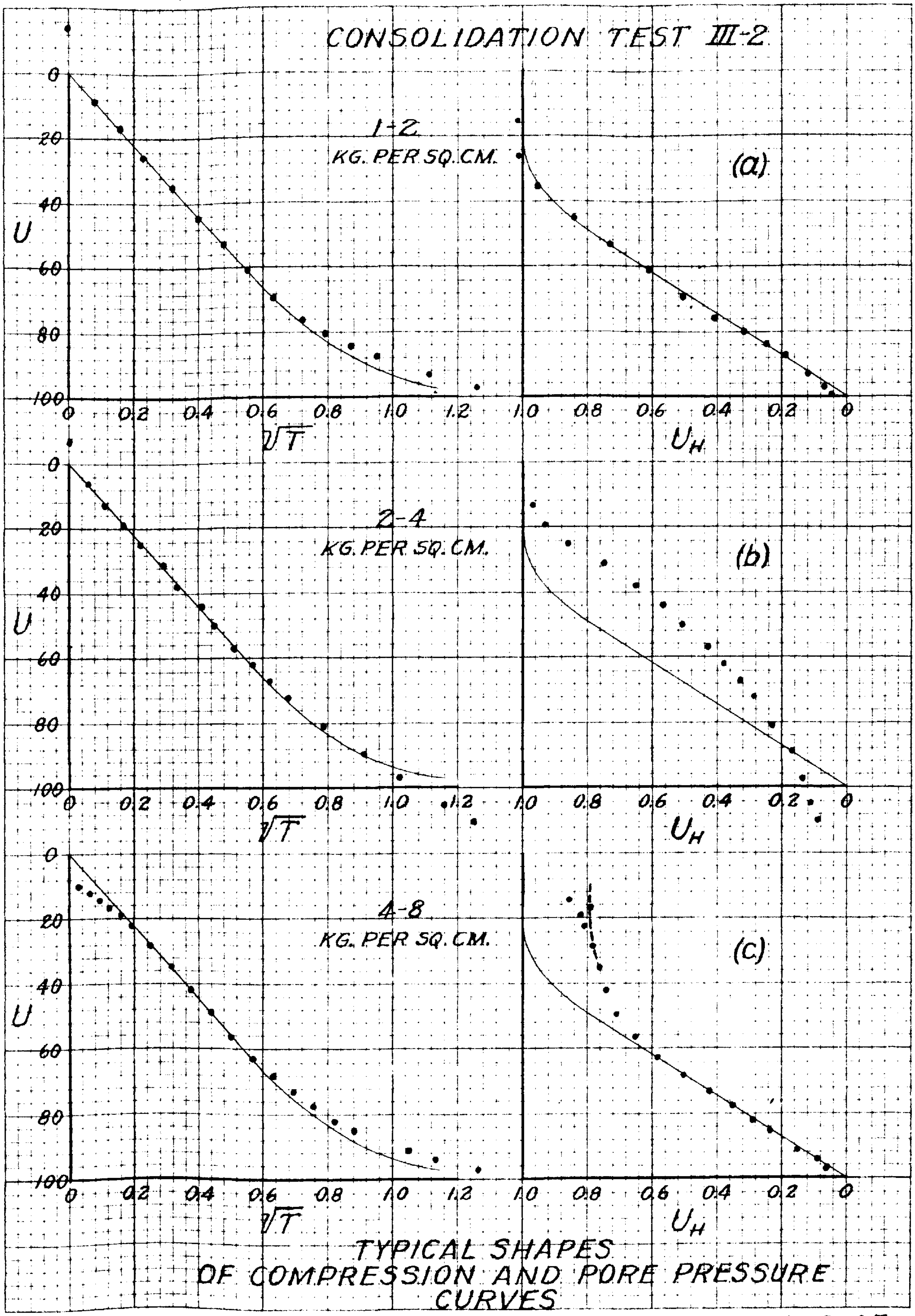
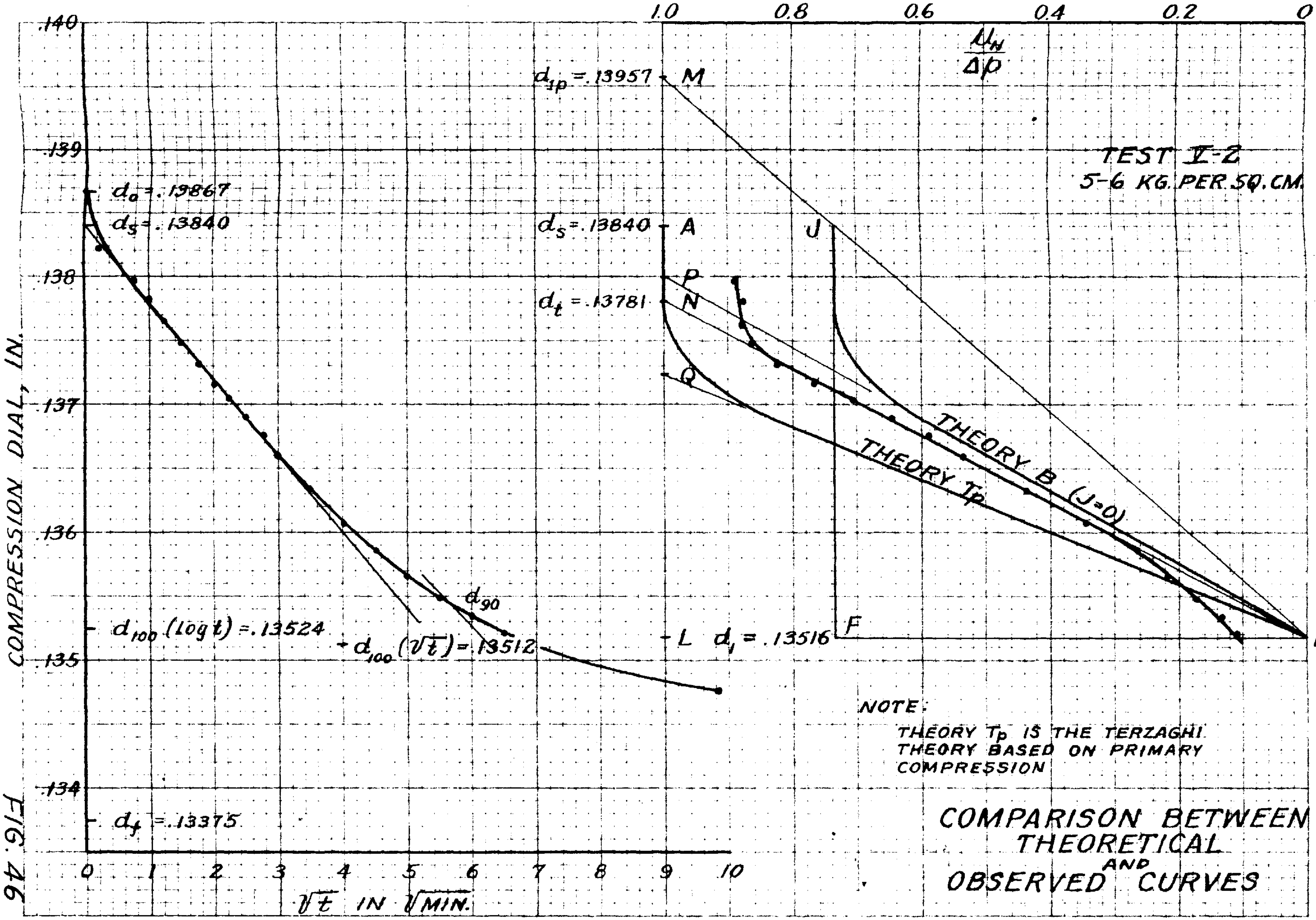

SERIES III-V
CONSOLIDATION TESTS
III-1 w = 48.6%
III-2 46.9
III-3 40.8
V-1 46.4
V-2 46.0
V-3 47.2
VOID RATIO, e
RELOADING
c_v
PRESSURE
C_v IN 10^{-4} CM^2 PER SEC
PRESSURE IN KG PER SQ CM

FIG. 44



TYPICAL SHAPES
OF COMPRESSION AND PORE PRESSURE
CURVES

FIG. 45



COMPARISON BETWEEN THEORETICAL AND OBSERVED CURVES

FIG. 46

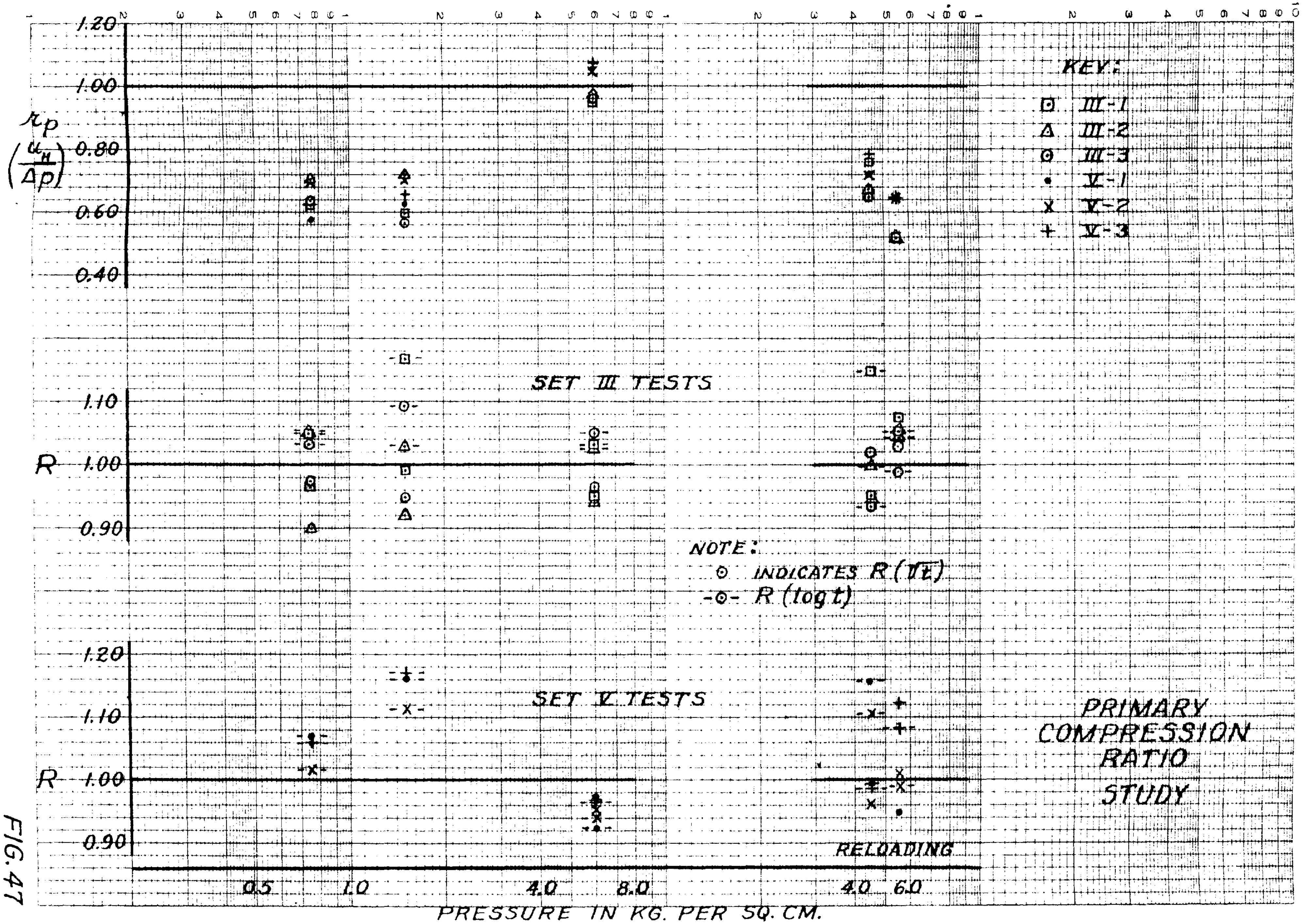

KEY:
III-1
III-2
III-3
V-1
V-2
V-3
r_p
$\left(\frac{u_H}{\Delta p}\right)$
1.20
1.00
0.80
0.60
0.40
SET III TESTS
1.10
R
1.00
0.90
NOTE:
⊙ INDICATES R($\sqrt{t}$)
-⊙- R(log t)
1.20
SET V TESTS
1.10
R
1.00
0.90
PRIMARY
COMPRESSION
RATIO
STUDY
RELOADING
0.5
1.0
4.0
8.0
4.0
6.0
PRESSURE IN KG. PER SQ. CM.

FIG. 47

LIST OF TABLES

TABLE I

SUMMARY OF CONSOLIDATION TEST DATA - BORING NO. 11

Depth	Elev. (Camb. Datum)	(1) Initial Void Ratio e_1	Max. Past Pressure p_c (2)	Pressure kg/sq cm 0.75 c_v (3)	0.75 C_c (4)	1.50 c_v	1.50 C_c	3.00 c_v	3.00 C_c	6.00 c_v	6.00 C_c	
ft	ft		kg/sq/cm	10^{-4} cm²/sec		10^{-4} cm²/sec		10^{-4} cm²/sec		10^{-4} cm²/sec		
41.1	-19.6	1.027	4.9	40	.036	25	.071	28	.118	30	.194	35'-55' Upper Foundation Clay
43.5	-22.0	1.013	5.1	38	.044	50	.069	52	.104	39	.166	
50.0	-28.5	1.016	5.2	53	.038	71	.056	56	.091	38	.222	
53.6	-32.1	0.928	5.4	80	.031	52	.039	61	.084	44	.202	
57.0	-35.5	1.133	3.6	44	.047	39	.092	29	.193	18	.310	55'-125' Lower Foundation Clay
63.6	-42.1	1.264	3.7	32	.044	29	.087	28	.167	9	.449	
72.6	-51.1	1.060	3.1	53	.035	120	.066	62	.226	12	.439	
78.6	-57.1	0.842	2.9	23	.040	126	.069	121	.195	32	.270	
84.8	-63.3	1.121	2.5	87	.040	72	.077	23	.313	15	.425	
94.1	-72.6	1.057	2.4	66	.050	49	.096	17	.290	14	.368	
100.6	-79.1	0.969	2.4	48	.050	40	.091	22	.227	16	.328	
Ave.	Upper Found. Clay	0.996	5.15	53	.037	49	.059	49	.099	38	.196	
Ave.	Lower (5) Found. Clay	1.101	2.95	55	.044	58	.085	30	.236	14	.386	

Notes:

(1) Void ratio at zero pressure in consolidation test.

(2) Casagrande graphical construction.

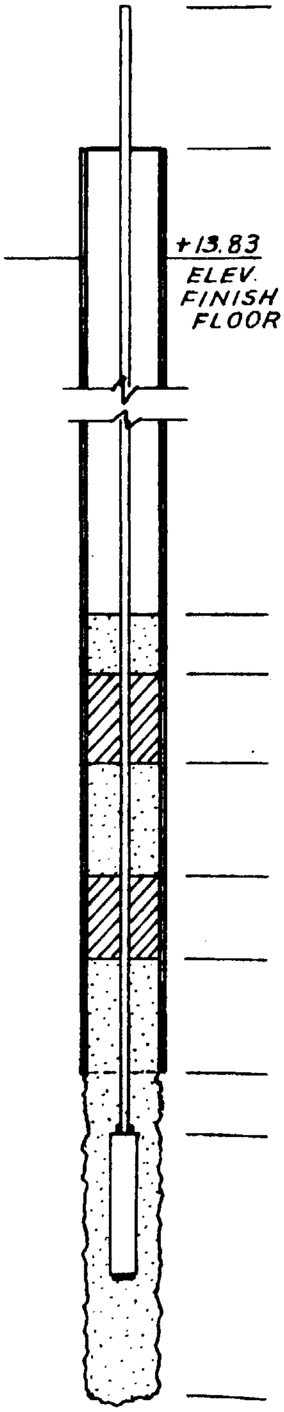
(3) $c_v = \frac{TH^2}{t}$ (see Chapter 6).

(4) $C_c = \frac{\Delta e}{\Delta(\log_{10} p)}$ (see Chapter 6).

(5) Does not include depth 78.6 ft. since test results are questionable because of silt.

TABLE II

SUMMARY ELEVATIONS OF COMPONENT PARTS OF EACH PIEZOMETER



Piezometer					Piezometer				
A-1	A-2	A-3	A-4	A-5	B-1	B-2	B-3	B-4	B-5
17.00	17.00	17.00	17.00	17.00	19.50	19.50	19.50	19.50	19.45
+15.7	+15.8	+15.9	+16.0	+ 14.7	+19.4	+19.4	+19.4	+19.4	+ 19.4
					- 6.6	-19.8	-43.2	-63.8	- 90.2
					- 7.3	-20.6	-45.7	-64.8	- 91.2
					- 8.6	-22.0	-46.4	-66.5	- 92.5
- 5.7	-26.2	-47.1	-66.9	- 95.2	- 9.3	-22.8	-48.4	-68.0	- 94.6
- 6.8	-27.5	-48.4	-67.8	- 96.8	-10.4	-23.9	-49.2	-69.2	- 95.7
- 9.5	-30.0	-50.0	-70.0	- 99.3	-11.2	-26.3	-50.0	-70.0	-100.0
-10.7	-30.8	-51.0	-68.9	-100.2	-12.5	-27.5	-51.9	-70.4	-100.3
-14.5	-35.0	-55.0	-75.0	-104.3	-16.2	-29.8	-55.0	-74.7	-105.3

NOTE:

(1) Elevations are on the Cambridge Base. (GROUND SURFACE = El.+21.5)

(2) See Figures 1 and 14 for location of piezometers in plan.

TABLE III

COMPUTATION OF COMPRESSION INDEX FROM PIEZOMETER CURVES AND SETTLEMENTS

Group A Piezometers

Time Interval	Found. Clay Layer	Ave. Δu	Ave. Δp	p_2	$\log_{10} p_2$	$\Delta(\log_{10} p)$	Total ρ	ρ	Δe	C_c
days		ft.	tons per sq. ft.	tons per sq. ft.			in.	in.	$\times 10^{-4}$	
600-700	Upper	-	-	-	-	-	0.07	0	-	-
	Lower	0.14	.0043	2.6043	.41569	.00072		0.070	1.750	~~.243~~
700-800	Upper	0.31	.0097	1.4097	.14912	.00299	0.04	0.003	.250	.008
	Lower	0.16	.0050	2.6050	.41581	.00084		0.037	.925	.110
800-900	Upper	0.11	.0034	1.4034	.14718	.00105	0.02	0.004	.333	.034
	Lower	0.17	.0053	2.6053	.41586	.00089		0.016	.400	.045
Group B Piezometers										
600-700	Upper	0.70	.0218	1.4218	.15284	.00671	0.07	0.010	.834	.012
	Lower	0.69	.0216	2.6216	.41857	.00360		0.060	1.500	.042
700-800	Upper	0.09	.0028	1.4028	.14699	.00086	0.02	0.004	.333	.039
	Lower	0.15	.0047	2.6047	.41576	.00079		0.016	.400	.051
800-900	Upper	0	-	-	-	-	0.01	-	-	-
	Lower	0	-	-	-	-		-	-	-

Ave. C_c Upper Clay 0.023

Ave. C_c Lower Clay 0.060

Note:

Upper Clay $p_1 = 1.4$ tons per sq. ft.

$\log_{10} p_1 = 0.14613$

Lower Clay $p_1 = 2.6$ tons per sq. ft.

$\log_{10} p_1 = 0.41497$

TABLE IVa

SUMMARY OF SERIES F CONSOLIDATION TESTS

Test No.	Ave. Natural Water Content	Load Increment	1 Total Compression	2 Initial Compression (1)	3 Primary Compression (1)	4 Initial Comp. Ratio r_o	5 Primary Comp. Ratio r_p	6 c_v ($\sqrt{t}$)	7 c_v (log t)	8 Ratio c_v ($\sqrt{t}$/log t)	Agreement Ratios (2) 9 r_o	10 r_p	11 c_v ($\sqrt{t}$)	12 Ratio c_v	13 Comp. Dial Rebound 8-7 kg/sq cm	Ratio of Slopes (3) 14 Primary Line ($\sqrt{t}$ plot)	15 Secondary Line (log t plot)
	%	kg/sq cm	in	in	in			10^{-4} cm²/sec	10^{-4} cm²/sec						in		
F-1	36.6	1-2	.01656	.00220	.0090	.133	.543	112.2	68.8	1.63					.00019		
		2-4	.03691	.0006	.0214	.016	.580	41.9	25.0	1.68					recom-		
Standard		4-8	.07235	-.0015	.0572	-.021	.791	18.1	14.0	1.29					pressed		
Test		7-4	-.00419	.0004	.00272	.095	.649	131.6	88.2	1.49					to		
		4-5	.00128	.00008	.00097	.063	.758	191.7	151.3	1.27					.00007		
		5-6	.00145	.00020	.00086	.138	.593	110.9	103.0	1.08							
F-2	36.8	1-2	.01093	.00122	.0066	.112	.604	104.9	102.0	1.03	1.19	.90	1.07	1.58		+	+
		2-4	.03650	.0004	.0192	.011	.526	38.6	15.0	2.57	1.45	1.10	1.09	.65		+	-
With		4-8	.07538	-.0035	.0645	-.046	.856	16.3	11.3	1.44	2.19	.92	1.11	.90	.00074	same	same
Membrane		7-4	-.00476	.00028	.00331	.059	.696	109.8	83.7	1.31	1.61	.93	1.20	1.14		-	-
		4-5	.00110	-.00003	.00076	-.027	.691	294.0	217.0	1.35	>1	1.10	.65	.94		same	-
		5-6	.00155	.00008	.00108	.052	.697	96.0	82.9	1.16	2.65	.85	1.15	.93		-	+
F-3	35.0	1-2	.01380	.00134	.0079	.097	.572	127.0	110.0	1.15							
		2-4	.04207	.0009	.0268	.021	.637	39.4	23.6	1.67							
Standard		4-8	.07515	-.0024	.0583	-.032	.775	31.2	22.4	1.39					.00036		
Test		7-4	-.00390	.00064	.00250	.164	.641	175	159	1.10							
		4-5	.00094	.00024	.00059	.255	.627	281	312	.90							
		5-6	.00124	.00018	.00090	.145	.726	232	233	1.00							
F-4	33.7	1-2	.01487	.0016	.0067	.108	.450	220.0	72.3	3.04	.91	1.27	.58	.38		-	same
		2-4	.03855	.0001	.0249	.003	.646	39.1	21.2	1.84	7.00	.99	1.01	.91	.00040	+	"
With		4-8	.07118	-.0040	.0572	-.056	.805	25.8	19.9	1.30	1.75	.96	1.21	1.07	recom-	+	"
Membrane		7-4	-.00437	.00042	.00304	.096	.696	187	128	1.46	1.71	.92	.94	.75	pressed	-	+
		4-5	.00098	.00009	.00079	.092	.806	197	194	1.01	2.77	.78	1.43	.89	to	-	+
		5-6	.00137	.00008	.00101	.058	.738	245	234	1.05	2.50	.98	.95	.95	.00028	-	-

(1) Determined from dial reading vs $\sqrt{t}$ plot and $\sqrt{t}$ time fitting method.

(2) Standard test divided by test with membrane.

(3) +(plus) indicates standard test has greater slope.

TABLE IVb

<u>SUMMARY OF SERIES F CONSOLIDATION TESTS</u> (CONT'D)

			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
											Agreement Ratios(2)						
Test No.	Ave. Natural Water Content	Load Increment	Total Compression	Initial Compression (1)	Primary Compression (1)	Initial Comp. Ratio r_o	Primary Comp. Ratio r_p	c_v ($\sqrt{t}$)	c_v (log t)	Ratio c_v ($\sqrt{t}$/log t)	r_o	r_p	c_v ($\sqrt{t}$)	Ratio c_v	Comp. Dial Rebound 8-7 kg/sq cm	Primary Line ($\sqrt{t}$ plot)	Secondary Line (log t plot)
	%	kg/sq cm	in	in	in			10^{-4} cm^2/sec	10^{-4} cm^2/sec						in		
F-5	37.8	1-2	.01458	.0013	.0077	.089	.528	120	87.3	1.38							
		2-4	.04525	.0012	.0251	.027	.555	40.8	18.2	2.24							
Standard Test		4-8	.10117	-.0038	.0795	-.037	.785	22.8	13.5	1.69					.00044		
		7-4	-.00456	.00048	.0032	.105	.702	117	90	1.30							
		4-2	-.00937	.0010	.00667	.107	.712	41.5	28.6	1.45							
		2-1	-.01045	.0007	.0091	.067	.871	13.3	–	–							
		1-2	.00566	.0005	.00439	.088	.777	50.0	41.4	1.21							
		2-3	.00485	.0004	.00327	.082	.675	49.6	37.4	1.32							
F-6	37.3	1-2	.01645	.0015	.0089	.091	.541	77.2	75.4	1.02	.98	.98	1.55	1.35		+	–
		2-4	.05682	.0006	.0334	.011	.588	21.8	10.4	2.10	2.46	.94	1.87	1.07		+	–
With Membrane		4-8	.08368	-.0040	.0695	-.048	.830	15.7	11.4	1.38	1.30	.95	1.45	1.22	.00062	+	–
		7-4	-.00483	.00056	.00341	.116	.706	110	90	1.22	.91	.99	1.06	1.07		–	same
		4-2	-.00842	.00040	.00656	.048	.780	41.5	33.5	1.24	2.23	.91	1.00	1.17		+	+
		2-1	-.00993	0	.0088	.000	.886	17.7	–	–	∞	.98	.75	–		–	
		1-2	.00525	0	.00458	.000	.873	38.2	32.4	1.18	∞	.89	1.31	1.03		+	+
		2-3	.00462	0	.00356	.000	.770	42.6	35.7	1.16	∞	.88	1.16	1.14		–	+
F-7	43.1	1-2	.01543	.00130	.00655	.084	.424	159	110	1.45							
		2-4	.07334	.0009	.0459	.012	.626	13.6	7.4	1.84							
Standard Test		4-8	.09805	-.0010	.0787	-.010	.803	9.8	7.9	1.24					.00023		
		7-4	-.00530	.00056	.00370	.106	.698	78.7	58.0	1.36							
		4-2	-.00939	.00041	.00754	.044	.803	22.8	15.4	1.48							
		2-1	-.01367														
		1-2	.00530														
		2-3	.00680	.00017	.00439	.025	.647	35.2	23.9	1.47							
F-8	42.4	1-2	.01719	.00137	.00666	.080	.388	147	38.2	3.85	1.05	1.09	1.08	.38		+	same
		2-4	.06979	.0007	.0456	.010	.654	10.1	7.4	1.37	1.20	.96	1.35	1.34		+	"
With Membrane		4-8	.09739	-.0048	.0844	-.049	.866	9.2	8.1	1.14	4.90	.93	1.07	1.09	.00082	same	–
		7-4	-.00555	.00028	.00422	.050	.760	69.0	54.1	1.28	2.12	.92	1.28	1.06		–	–
		4-2	-.00962	.00033	.00801	.034	.833	17.5	14.4	1.22	1.29	.96	1.30	1.21		+	same
		2-1	-.01386														
		1-2	.00516														
		2-3	.00680	.00009	.00511	.013	.751	23.0	20.6	1.12	1.92	.86	1.53	1.31		+	–

TABLE V

SUMMARY OF SERIES III-V CONSOLIDATION TESTS[1]

Test No.	Ave. Natural Water Content	Load Increment	Total Compression	1 Max.[2] $\left(\frac{u_H}{\Delta p}\right)$ (Observed)	2 Max. $\left(\frac{u_H}{\Delta p}\right)$[3] (Theory B)	3 $\frac{NL}{AL}$	4 Theory T_p $\frac{NL}{0.636AL}$	5 Theory B $\frac{NL}{PL}$	6 r_p ($\sqrt{t}$)	7 r_p (log t)	8 r_p $\left(\frac{u_H}{\Delta p}\right)$	9 R($\sqrt{t}$)	10 R(log t)
	%	kg/sq cm	in										
III-1	48.6	0.475-1	.01041	1.10	-	0.68	1.07	-	.584	.635	.606	.965	1.049
		1-2	.01413	1.01	0.68	0.74	1.16	0.79	.593	.699	.598	.992	1.169
d=4.25 in		2-4	.04445	1.01	-	-	-	-	.467	.666	-	-	-
		4-8	.12547	0.70	0.75	0.84	1.32	1.00	.898	.976	.944	.951	1.033
		4-5	.00190	0.88	-	0.65	1.02	-	.722	.868	.758	.953	1.147
		5-6	.00198	0.86	0.565	0.82	1.29	0.75	.560	.546	.520	1.077	1.050
III-2	46.9	1/2-1	.01211	1.09	-	0.59	0.93	-	.638	.746	.710	.898	1.051
		1-2	.01394	1.01	0.77	0.63	0.99	0.76	.659	.736	.717	.920	1.026
d=4.25 in		2-4	.05108	0.98	-	-	-	-	.438	.824	-	-	-
		4-8	.12810	0.79	0.87	0.63	0.99	0.86	.916	.999	.975	.940	1.026
		4-5	.00204	0.87	-	0.60	0.94	-	.610	.634	.634	.947	1.000
		5-6	.00309	0.87	0.61	0.77	1.21	0.73	.544	.537	.515	1.057	1.043
III-3	40.8	1/2-1	.00778	1.05	-	0.56	0.88	-	.616	.653	.633	.973	1.031
		1-2	.01217	1.05	0.69	0.69	1.09	0.75	.534	.616	.563	.949	1.093
d=4.25 in		2-4	.05498	0.95	-	-	-	-	.463	.792	-	-	-
		4-8	.10985	0.79	0.84	0.54	0.85	0.71	.930	1.010	.965	.963	1.047
		4-5	.00167	0.80	-	0.74	1.16	-	.653	.599	.641	1.019	.936
		5-6	.00238	0.70	0.66	0.86	1.35	0.90	.534	.516	.521	1.027	.990
V-1	46.4	1/2-1	.02945	Loading beam tilted considerably during test. Results are erratic.					-	.614	.574	-	1.070
		1-2	.05055						-	.725	.624	-	1.161
d=9.55 in		2-4	.17490						.744	.824	-	-	-
		4-8	.32740						1.028	.975	1.053	.976	.925
		4-5	.00358						.716	.832	.718	.997	1.159
		5-6	.01092						.611	-	.644	.947	-
V-2	46.0	1/2-1	.02774	1.00	-	0.57	0.90	-	-	.710	.699	-	1.016
		1-2	.04614	1.02	0.83	0.72	1.13	0.93	~~.556~~	.782	.702	-	1.114
d=9.55 in		2-4	.17974	0.98	-	-	-	-	.804	.874	-	-	-
		4-8	.31760	0.82	0.93	0.65	1.02	0.96	1.004	.988	1.051	.956	.940
		4-5	.00345	0.88	-	0.78	1.23	-	.714	.820	.743	.961	1.104
		5-6	.00492	0.89	0.73	0.82	1.29	0.94	.667	.643	.659	1.012	.975
V-3	47.2	1/2-1	.03064	1.03	-	0.72	1.13	-	-	.656	.617	-	1.061
		1-2	.05560	1.05	0.80	0.77	1.21	0.96	~~.544~~	.763	.653	-	1.169
d=9.55 in		2-4	.21265	0.94	-	-	-	-	.880	.950	-	-	-
		4-8	.30790	0.81	0.98	0.65	1.02	1.00	1.040	1.034	1.073	.969	.962
		4-5	.00331	0.84	-	0.90	1.42	-	.776	.770	.779	.995	.988
		5-6	.00538	0.82	0.68	0.93	1.46	0.98	.656	.634	.584	1.122	1.086

(1) See Figure 46 and text for notations used. (2) Not necessarily the absolute maximum (see text). (3) Numerically equal to r'_p.

BIOGRAPHICAL SKETCH

OF

HARL PRESLAR ALDRICH, JR.

EDUCATION:

University of Idaho, Moscow, Idaho, 1941-1943.

Massachusetts Institute of Technology, 1945-1951.
S.B. in Civil Engineering, 1947.

SOCIETIES:

Junior, American Society of Civil Engineers.
Chi Epsilon, Tau Beta Pi, and Sigma Xi.

TEACHING AND PROFESSIONAL EXPERIENCE:

Draftsman with Clifton, Applegate, and Henry Georg, Contractors. Spokane, Wash. Summer, 1942.

Field Engineer for Clifton, Applegate, and Henry Georg, Contractors. Spokane, Wash. Summer, 1943.

Assistant in Civil Engineering, Massachusetts Institute of Technology. 1947-1949

Instructor of Soil Mechanics, Massachusetts Institute of Technology. 1949-1951