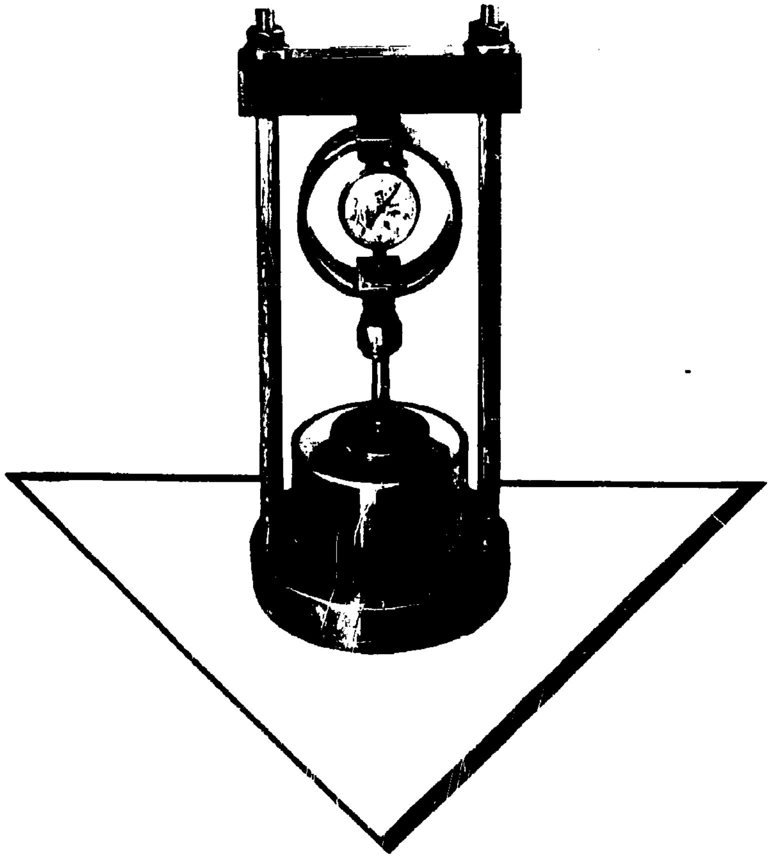
AD 286 951

6.3-1-3

FHA 286 951

SOIL PVC METER



A TECHNICAL STUDIES REPORT
DECEMBER 1960

FHA-701

FEDERAL HOUSING ADMINISTRATION • WASHINGTON 25, D. C.

THE CHARACTER AND IDENTIFICATION OF EXPANSIVE SOILS

A Report completed for the

TECHNICAL STUDIES PROGRAM

of the

FEDERAL HOUSING ADMINISTRATION

by

T. William Lambe
Consulting Soil Engineer
Massachusetts Institute of Technology

May, 1960

FOREWORD

This report is published to provide background information and other essential data which lead to the development of the FHA Soil PVC Meter through our Technical Studies Program.

The Federal Housing Administration only insures mortgages on residential construction that conforms to specific minimum standards of quality and durability. The agency's responsibility involves architectural design which in turn is influenced by the behavior of foundation soils. Significant soil characteristics, including expansive and shrinkage qualities, must be considered in determining the structural design required to withstand this type of instability.

In seeking a practical, simple, and quick method for determining these qualities, FHA contracted with Dr. T. William Lambe, of the Massachusetts Institute of Technology, to develop a simple soil testing device suitable for use in identifying potential volume change of clay soils. The result is the FHA PVC Meter, a small apparatus designed to measure the swell index of some clay soils. Elvin F. Henry and James R. Simpson, of the FHA Architectural Standards Division, guided the development of this soil testing device.

The completed report includes all research background, a summary of environmental and moisture conditions related to volume change and behavior, a full explanation of the laboratory testing program conducted in the development and calibration of the soil testing device, and instructions for the operation and use of the FHA PVC Meter.

FHA believes considerable savings may accrue to homeowners and builders by pre-testing soils before construction begins for the purpose of identifying potentially dangerous soil conditions.

The development of the PVC Meter is another example of FHA's service to the public without cost to the taxpayers since FHA is an entirely self-supporting agency.

Office of the Commissioner
Federal Housing Administration

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd.)

LIST OF TABLES

TABLE OF CONTENTS (Cont'd.)

LIST OF FIGURES

TABLE OF CONTENTS (Cont'd.)

LIST OF FIGURES (Cont'd.)

TABLE OF CONTENT (Cont'd.)

LIST OF DRAWINGS

TABLE OF CONTENTS (Cont'd.)

LIST OF PLATES

SUMMARY

A. Objectives of Project

The main objective of the work described in this report was to develop a device and method to permit the expeditious identification of foundation (for light structures) soils which would be potentially troublesome due to excessive shrinkage or swelling. Secondary objectives were to make a literature survey of the problem of building on "expansive" soils and to summarize the fundamentals of soil volume change behavior.

B. Shrinkage and Swelling Behavior of Inorganic Soils

1. Soils most capable of large volume changes are plastic clays with high surface areas.

2. Volume changes occur when the effective stress (externally applied stress minus pore pressure) acting on a soil changes, thus requiring changes in interparticle spacing (double-layer thickness) in order that the interparticle electrical forces again achieve equilibrium with the effective stress.

3. The type of volume change, i.e., shrinkage versus swelling, depends upon the initial water content (wet versus dry) and the change in "moisture conditions" (drying versus wetting). The environment (climatic, pedologic and hydrologic factors and the influence of man-placed structures) around a soil controls the moisture conditions in the soil.

4. Damage due to swelling of soils is more common than that due to shrinking of soils, both on a world-wide basis and on a national basis. This is particularly true in the south and southwest parts of the United States. Damage due to alternate cycles of swelling and shrinking are infrequently reported.

5. Swelling is most prevalent in climates with a high rate of water evaporation compared to rainfall so that buildings are often constructed on desiccated soils. These desiccated foundation soils can imbibe water and swell due to a reduced rate of water evaporation combined with the capillary rise of water from a water table, and/or from the seepage of water from irrigation projects, heavy rainfalls, faulty

water mains, septic systems, etc. The amount of swelling of a given soil is a function of the initial water content and density, the confining pressure, and the time available for swelling in relation to the thickness of soil.

6. Shrinkage is most prevalent where climate and hydrologic factors are conducive to predominantly wet soils, but where prolonged droughts sometimes occur. The amount of shrinkage of a given soil is a function of the initial water content and the time of drying in relation to the thickness of soil. Trees can aggrevate the situation by taking moisture from the soil.

7. Cyclic volume changes are most likely in climates characterized by cool wet seasons followed by warm dry seasons.

C. <u>Laboratory Test Program and Swell Index Device</u>

1. Numerous classifications tests, and heave and swell pressure tests on samples at three relative contents ("Dry," "Moist" and "Wet") were performed on ten soils ranging from slightly plastic silts to very plastic clays. These results were correlated with values of "Swell Index" defined herein as the pressure exerted by a compacted sample that has been immersed in water for two hours. Photos of the apparatus used to measure the Swell Index (SI) are shown in Plates 1 through 4.

2. A good correlation was found to exist between the Swell Index and both the swelling magnitude and swell pressure behavior for samples of similar density and water content.

3. The Swell Index test can indicate the Plasticity Index (PI) and Potential Volume Change (PVC) of a soil if the Relative water content of the sample tested is known.

4. The PVC of a soil refers to the maximum possible volume change that the soil can undergo due to swelling or shrinkage. Four categories of PVC are established ranging from "noncritical" to "Very Critical." The PVC ratings were established on the basis of the swelling and shrinkage behavior of the soil, its plasticity index and grain size, and its water content at 85 per cent relative humidity.

I. INTRODUCTION

A. Background

The Federal Housing Administration (FHA) among other responsibilities, insures mortgages on residential buildings issued by commercial banks to private individuals. If the mortgagor defaults on his payments the FHA assumes ownership of the residence and pays to the bank that portion of the mortgage which is unpaid. In order to help keep losses to a minimum, the FHA only insures mortgages on residences that meet certain standards of quality and durability. These standards dictate that the building must meet, among other things, a certain minimum design and construction specification and must be constructed on foundation soils that will not cause excessive damage to the structure.

One type of foundation soil known to cause damage to dwellings are the so-called "swelling" or "expansive" soils. These soils can undergo volume changes in the field which cause large differential movements within the structure, and hence, excessive cracking of walls, floors, piping, etc. One must identify potentially expansive foundation soils in order to evaluate properly the quality and durability of a dwelling. In order to assist personnel who are unfamiliar with soil engineering and soil classification the FHA needed a field testing device capable of identifying expansive soils. On January 6, 1959, the FHA entered into a one year contract with T. William Lambe, Consulting Soil Engineer, Massachusetts Institute of Technology, to work on this problem.

B. Scope of Contract

A summary of the scope of the contract as finally executed[1], is as follows:

1. Make a limited survey of the literature pertaining to the subject of damage to buildings as a result of expansive soils.

---

1. It should be stated that one important misunderstanding arose with respect to the scope of the contract. The Contractor originally interpreted the purpose of the field testing device to to as follows: The device would be used to test within a period of 2 hours a sample of soil at its in situ water content and would permit the operator to predict how much this soil could swell (i.e., potential swell) if given unlimited access to water. If the operator desired the potential swell of a soil for initial water contents differing from the in situ water content then the operator would have to change (either wetting or drying) the in situ water content to the desired water content.

2. Summarize the mechanisms causing volume changes in soils (both swelling and shrinking) and the factors influencing volume change behavior.

3. Develop a field testing device that could be used to determine the Potential Volume Change (PVC) of a soil. The PVC of a soil refers to the maximum possible volume change that the soil could undergo from water content changes (i.e., starting dry and swelling if wetted or starting wet and shrinking if dried). Soils were to be divided into four categories: "Very Critical," "Critical," "Marginal" and "Noncritical." This device should be portable, self-contained, simply operated and capable of testing a sample within two hours.

4. Perform laboratory test on ten soils in order to correlate measurements made with the field device with the volume change behavior of soils.

5. Deliver a working model of the field testing device, with specifications, to the FHA and describe how to obtain and interpret test results.

6. Write a report covering the work performed.

C. Acknowledgements

The laboratory tests at M.I.T. were performed by Messrs. D. Leary and R. Ladd, cooperative students at Northeastern University, under the direction of Mr. C. Ladd, Instructor of Soil Engineering at M.I.T. Mr. Ladd performed the literature survey and prepared this report. Dr. T. William Lambe generally supervised the contract work.

D. Nature of Report

This report is written so that a person with little knowledge of soil engineering can understand the essential features. However, many of the important aspects of the problem of expansive soils, although complex, will be briefly discussed. Presented is a list of selected references which will enable the reader to comprehend the fundamentals of soil engineering or to delve more deeply into problems only summarized in this report.

II. FUNDAMENTALS OF SOIL VOLUME CHANGES AND VOLUME CHANGE BEHAVIOR

A. Nature of Clay and Clay Water Forces

The volume change behavior of soils is greatly influenced by the amount and type of clay present in the soil. Clay particles are colloids with electrical properties which cause forces of interaction (so-called physicochemical forces) between particles and between particles and water. The term clay "micelle" refers to a clay particle[2] and the water and ions, called the "double-layer" associated with the particles. Because of the electrical, colloidal nature of clay, clay micelles have a great attraction, or "thirst" for water. If this thirst is not satisfied, a "double-layer deficiency" occurs (Lambe, 1958, 1960), i.e., the double-layer around the particle does not have as much water as it would like to have.

An equation of statics may be written for interacting particles in an expansive clay (Lambe, 1958, 1960) which relates the physicochemical stresses acting between particles with the stresses which soil engineers measure and/or use to predict soil behavior. In simplified form[3], this equation can be presented as follows:

$$\bar{\sigma} = \sigma - u = R - A \qquad (1)$$

$\bar{\sigma}$ = effective or intergranular stress[4] which is the force transmitted between interacting particles per unit of soil, and which can be well correlated with soil behavior.

---

2. Clay particles are composed of clay minerals which are natural inorganic substances of a definite crystalline structure and chemical composition, such as kaolinite, illite and montmorillonite.

3. Assuming no actual mineral to mineral contact between particles and a saturated system. See Lambe (1960) for the complete equation.

4. See Taylor, 1948, Terzaghi and Peck, 1948, or other test books on soil mechanics for references on soil engineering terminology and principles.

σ = total external stress applied to soil.

u = pressure in the free pore water of the soil mass (that water outside the influence of the physicochemical forces from the particles).

R = the repulsive pressure which arises from the electrical nature of the particles.

A = the attractive pressure between clay particles which also originates from the electrical nature of the particles.

A diagram depicting the above relationships is shown in Fig. 1 for an equilibrium particle spacing of 2d. The $\bar{\sigma} = \sigma - u$ portion of Eq. 1 is called the effective stress equation. Both σ and u can be measured to obtain a value of $\bar{\sigma}$, the effective stress, which in turn is used to predict soil behavior. R-A, the net repulsive pressure acting between particles, which increases with decreasing interparticle spacing, is the stress which actually causes soil behavior; however, neither R nor A can be measured.

In summary, Eq. 1 means that for a given soil-water system with a given interparticle spacing, there is a net repulsive pressure between particles (R-A) which requires the application of an effective stress $\bar{\sigma}$ (equal to total applied load minus pressure in pore water) to the soil to maintain volume equilibruim.

A detailed discussion of the nature of clay and clay-water forces can be found in Grim (1953), Lambe (1953, 1958, 1960), Taylor (1959), HRB (1958), Low (1959), Martin (1959), and Bolt (1956). Lambe and Whitman (1959), Aitcheson (1957), and Hilf (1956) discuss the validity of the effective stress equation.

B. Processes Causing Volume Changes

Any process which changes the effective stress on a given soil-water system will cause a volume change (in saturated soils, drainage must occur to allow water to flow in or out of the sample). The effective stress can be changed by changes in σ, the externally applied loan. Typical examples are consolidation and rebound, compaction, and shearing. The effective stress can also be changed due to changes in the pore pressure, u, caused by changes in the "moisture conditions" (to be discussed in detail in Section III) around the soil, such as from wetting or drying a soil. The processes of swelling and shrinkage, which are of primary concern, fall in this latter category.

Although volume changes due to swelling and shrinkage are considered in terms of changes in effective stress, it is emphasized that the physicochemical characteristics of a soil determine the amount of volume change for a given effective stress change. In other words, the greater the surface area of the clay (the smaller the particles), the greater is the overall volume change per change in particle spacing required to make $\Delta(R-A)$ equal to $\Delta\bar{\sigma}$.

C. Shrinkage of Soils

1. Mechanism

The mechanism of soil shrinkage, from an effective stress viewpoint, is stated in several references (e.g., Terzaghi and Peck, 1948, and Means, 1959). Briefly, when a saturated clay is, for example, dried, the evaporation of water from the exterior pore water-air interfaces (menisci) causes a tension to be set up in the pore water. This negative pore water pressure increases the effective stress on the clay, which in turn causes a reduction in interparticle spacing. Water then flows from between the interacting particles to the exterior air-water interfaces where it evaporates. This reduction in the volume of soil and evaporation of "surface" water continues until the soil can shrink no further, at which point the air-water interfaces retreat into the soil.

The rate of surface evaporation of water depends predominantly on the relative humidity of the air and, to a much smaller extent, on the tension in the water. The magnitude of the pore water tension at any time (and hence the effective stress) will depend on the size of the pores and the relative rates of surface evaporation and flow of water to the surface. In any case, if drying proceeds long enough, the pore water tension can theoretically reach extremely high values (several hundred atmospheres).

2. Behavior

A detailed description of the shrinkage behavior of soils and the factors which influence the behavior can be found in Wooltorton (1954).

A summary is presented in Fig. 2, which shows the shrinkage behavior of three types of samples of a given clay in the form of sample volume versus water content (and the corresponding tension in the pore water as the water content decreases from drying).

A remolded saturated clay starts drying from the volume and water content conditions shown in A_1. During the initial portion of drying (A_1 to B_1), the volume change is governed by the equation

$$\frac{\Delta V}{V_o} (\%) \frac{w_1 - w}{\frac{w_1}{100} + \frac{1}{G}} \qquad (2)$$

where

w_1 = water content in per cent at A_1,
w = water content in per cent,
G = specific gravity of soil solids.

At B_1, the exterior air-water interfaces begin to retreat into the soil voids, but the volume continues to decrease until D_1 is reached upon which further drying has no effect on volume. The water content at C_1 is the Shrinkage Limit of the soil. (Water content required to fill voids of dried soil.)

The shrinkage of a partially saturated sample is shown by A_2 through D_2 (as the initial degree of saturation decreases, line A-B shifts to the left and the final volume increases).

An undisturbed natural nonsaturated sample with initial conditions at A_3 (i.e., same conditions as for the remolded sample) would act similar to the remolded nonsaturated sample, but would usually shrink to a <u>larger</u> final volume due to the natural "fabric"[5] of the clay which prevents a very close packing of particles. Thus the Shrinkage Limit of the natural sample would be larger than that of the remolded sample.

The amount of volume change that a soil would exhibit in the field would depend primarily upon the initial water content and degree of saturation, i.e., location of Point A, and on the drying conditions (e.g., the relative humidity, and the size of the sample in relation to the elapsed time of drying), i.e., how far along line A-D has the water content decreased. In general, the more plastic the soil, the more water it can hold (assuming all soils to have the same <u>thickness</u> of water around the particles for a given set of conditions, then the larger the surface area, the higher the water content) and the higher the value of Point A for a given set of climatic or loading conditions.

---

5. Natural soils often have an "edge-to-face" arrangement of particles which can be destroyed by remolding, i.e., remolded soils tend to have a "parallel" arrangement of particles.

D. Swelling of Soils

1. Mechanism

In general, it can be stated that the swelling of soils is due to decrease in the effective stress, $\bar{\sigma}$, acting on the soil mass so that the net repulsive pressure (R-A) between interacting soil particles pushes the particles apart. In particular, when a partially saturated clay at a relatively low water content is given access to water, the pore pressure increases, $\bar{\sigma}$ decreases, and swelling occurs. Since this case is prevalent in practice, a discussion is warranted of the stresses in such soils before and after swelling.

First consider a clay sample which is partially dried or is compacted at a relatively low water content and which has no externally applied load and no access to water (e.g., a sample sitting on a table). The pore water in this sample will be in a state of tension (u is negative[6]). This pore water tension arises since the clay particles want to imbibe more water in order to expand their double layers, i.e., to satisfy their "double-layer deficiency" (Lambe, 1958) or "thirst" for water. Capillarity may also enter the picture if any soil void contains both air and "free" water. The desire of the clay micelles to imbibe this free water would be resisted by the surface tension at the air-water interface in the void. Thus the pore water tensions in the sample represent a balance between double layer deficiencies and surface tensions at air-water boundaries.

When the sample is put in contact with water, any air-water menisci at the surface of the sample is broken. Water will flow into the clay because of the water tensions within the sample. With time, the pressure in the pore water increases (for example, to atmospheric) with a resultant decrease of the effective stress within the sample. Concurrently, the clay micelles expand their double layers and swelling occurs between interacting clay particles until the net repulsive pressure between particles is in equilibrium with any applied effective stress.

A more detailed discussion of the possible mechanisms involved in swelling of partially saturated clays may be found in Ladd (1960).

2. Behavior

a. General

The swelling behavior of soil is governed primarily (but not solely) by the following factors (assuming that the soil is

---

6. Atmospheric pressure is taken as zero, e.g., the pressure at the surface of ocean water.

given unlimited access to water):

1. Composition of the soil: composition and amount of clay minerals, nature and amount of exchangeable actions, proportions of sand and silt, and presence of organic matter and cementing agents.

2. Initial water content, dry density (and hence, degree of saturation).

3. Fabric: arrangement and orientation of particles (as a result of natural processes or man-made processes, such as compaction).

4. Chemical properties of the pore-fluid - both before and during swelling.

5. Confining pressure applied during swelling.

6. Time allowed for swelling.

Of the above factors, only Items 2, 5, and 6 will be discussed in any detail. The effect of soil composition is covered by the experimental data presented in this report, and the subsequent discussion of swelling behavior will concern only "expansive" i.e., plastic, clays. Little is known relative to the effect of "fabric" on swelling behavior, particularly with respect to natural undisturbed clays. Only the behavior of compacted clays will be covered - one must assume undisturbed clays to have the same behavior under similar conditions of water content, density, confining pressure, etc.[7] The effect of Item 4, which is usually of little practical concern, can be deduced from Lambe (1958) and Ladd (1960).

b. Effect of Molded Water Content and Density

The effect of molded water content and density on swelling behavior under low confining pressures can be summarized as follows:

1. For a constant water content, the higher the dry density, the greater the amount of swell. The effect is more pronounced at the lower water contents.

---

7. Such as assumption should usually prove conservative due to "cementing," etc., which may occur in natural clays. It should be emphasized, however, that the type of compaction, i.e., dynamic, static, kneading, etc., can influence the swelling behavior of some soils (Seed and Chan, 1959, Seed, et al, 1954). Holtz (1959) shows data where compacted samples swelled more than undisturbed samples.

2. For a constant dry density, the lower the water content, the greater the amount of swell. The effect is more pronounced at the higher densities.

Figure 3 summarizes these trends with a plot of swell versus dry density for equal molded water contents (data replotted from Holtz and Gibbs, 1956). (The samples were compacted in consolidometers, a 1 psi load applied, water added, and the amount of volume increase measured.) Thus, the drier and denser a soil, the greater the swell when the soil has access to water.

c. Effect of Confining Pressure

An increase in the confining pressure causes a decrease in the amount of swell. This fact is shown in Fig. 4.[8] The top curve represents the amount of swell, or heave, (height of sample increased 21.5 per cent) that occurs when water is added to the sample with a surcharge pressure of only 200 lb./sq.ft., followed by the application of increased pressures with a resultant decrease in sample height. The bottom curve was obtained by applying a sufficient surcharge (12,000 lb./sq.ft.) to maintain no volume change when water was added to the sample. This pressure represents the "swell pressure" of the sample. Then the pressure was decreased, with a resultant sample expansion (the final heave under 200 lb./sq.ft. for the bottom curve is somewhat lower than that shown by the top curve, i.e., 18.5 per cent versus 21.5 per cent). The dotted line represents the approximate relationship between the amount of heave and the value of the confining pressure applied during swelling process.

Three important trends, which are representative of the behavior of expansive soils should be noted from the dotted curve in Fig. 4: 1) the magnitude of swell increases sharply with decreasing pressure at low levels of confining pressure (such as might be encountered from dwellings), 2) the magnitude of swell increases very gradually with decreasing pressure at high levels of confining pressure (in the vicinity of the swell pressure) and 3) soils with high swell pressures also exhibit large heaves under low confining presures.

d. Effect of Time

The swelling process requires time since water must flow into the sample before the particles can expand their double layers. The time required for equilibrium, i.e., when the final value of swell is reached, depends on the permeability of the soil, the distance the water must flow, and the amount of expansion. Plots of heave versus time for samples compacted at different water contents at the same compactive effort are shown in Fig. 5.[9] These samples were only about 3/4 inches thick with top and bottom porous stones (i.e., the water had to flow a maximum distance of 3/8 inches).

---

8. Same general testing procedure as previously described.

9. "Opt." -5% in Fig. 5 denotes compaction at 5% water content below optimum water content.

While some researchers (e.g., de Wet, 1957) equate the swelling process to a "reverse" consolidation, so that the time required for swelling would vary as the square of the sample thickness,[10] the Writer believes that the process is considerably more complex, at least with partially saturated clays. However, such a time-thickness relationship can serve as a general guide.

E. Summary of Principal Factors Controlling Volume Changes

1. The soils most susceptible to large volume changes are plastic clays with high surface areas which can imbibe large quantities of water, and thus can attain high water contents and which show large overall volume changes due to changes in double-layer thickness.

2. The type of volume change, i.e., shrinkage versus swelling, depends upon the initial water content (wet versus dry) and the change in "moisture conditions" (wetting versus drying).

3. The amount of shrinkage, percentage wise, of a given soil is a function of the initial water content and degree of saturation and the water content change.

4. The amount of swelling, percentage wise, of a given soil is a function of the initial water content and density, the confining pressure, and the thickness of sample in relation to the elapsed time since the addition of water to the sample. Decreasing water content and confining pressure and increasing density yield increasing amounts of swell.

---

10. See Taylor, 1948, for the theory governing time versus consolidation.

III. ENVIRONMENT AND MOISTURE CONDITIONS

Volume changes in soil occure when the "moisture condition" around soil changes, where moisture condition is a general term referring to those elements of the physical environment (excluding load changes) which determine whether or not water will enter or leave the soil. In order to predict trends of volume change behavior in the field, one must ascertain which elements of the overall physical environment around a soil mass are important to the moisture conditions in the soil.

A. General

The principal environment factors which effect the moisture conditions of soil are:

1. Climatic
2. Pedologic
3. Hydrologic
4. Man-placed structures.

The most important aspects of climate are the amount of precipitation (rainfall or snowfall) and the rate of evaporation. Vegetation effects the moisture conditions due to the transpiration of water from plants which obtain water from soil by root systems and due to modification of the drainage pattern. The hydrologic factor refers to the location of the water table and the conditions of seepage. One must also include the effects of man-made structures on moisture conditions when considering volume changes under buildings.

In order to better illustrate the importance of the above factors, some types of environment leading specifically to shrinking and swelling will be discussed.

B. Environment and Shrinkage

The principal cause of shrinkage of wet soils is evaporation of water from the pores of the soil. In turn, Thornthwaite (Johns Hopkins University, 1954) has found that the evaporation rate of water from soil can be well correlated with mean daily temperatures and the length of day. Thus periods of warm weather with relatively little rainfall (which would replenish evaporation losses) would favor shrinkage. During such periods, the situation may be aggrevated by the presence of vegetation which would also extract water from soil. On the other hand, large amounts of rainfall, and/or low temperatures would not favor shrinkage. Hence, consideration of only mean temperatures or of only average yearly rainfalls can be very misleading. One must consider the net effect over a given time, such as over a period of several weeks or months.

C. Environment and Swelling

The swelling of dry soil occurs when a change in environment results in a supply of water which can be imbibed by the clay to satisfy its double-layer deficiency. The problem, then, is determining how the soil may obtain water, or, in other words, how moisture can migrate to the soil. Several types of moisture movement are discussed below:

1. Seepage, or the flow of water due to the force of gravity. The seepage may result from natural phenomenon, such as from rainfall, and/or as a result of man-made phenomena, such as from irrigation ditches or faulty water mains.

2. Capillarity, or the flow of water due to the forces acting as the menisci of air-water interfaces in soil voids. The capillary rise[11] of water from a water table to soils above the water table is often a very important source of water.

3. Vapor transfer, or the flow of water through soil air voids in the form of water vapor due to differences in water vapor pressure.

The vapor pressure of water in air voids increases with increased temperature and water content of the soil. Thus water vapor will flow from soil of high temperature and/or water content to soil of low temperature and/or water content (see Road Research Laboratory (1954) for a general discussion of vapor transfer). However, this method of moisture movement is usually only of concern in soils at low degrees of saturation (say, below 80 per cent) under high temperature gradients. (Vapor flow through air voids can contribute to the skrinkage of soil, but this is usually of secondary importance since most of the volume decrease of wet soil occurs at a high degree of saturation.)

---

11. As a very general guide, water can rise to heights of 1, 10, and 100 feet above the water table in sands, silts, and clays respectively; the different heights being inversely proportional to the size of the pores in the soil.

VI. LITERATURE SURVEY OF THE PROBLEM OF BUILDING ON EXPANSIVE SOILS

A. General

Over forty references on the general subject of the behavior of and the problems encountered with "expansive" soils were reviewed. In spite of the numerous references, there are very few really complete case studies of the behavior of buildings on swelling and shrinking soils. A complete study should include: 1) soil properties and depth of soil, 2) water content and density of the soil before and after the volume change, 3) amount of volume change and loads acting on the soil, and 4) climatic, hydrologic, and pedologic data if required for an understanding of what instigated the volume change. Such a study would, of course, require time and money, which partially explains the dearth of reliable data. In general, the most complete data are reported from work in the Union of South Africa.

Before representing the results of the literature survey, a brief review of the requirements for volume changes may be warranted. The first requirement is an "expansive" or "active" soil, i.e., plastic clay that is susceptible to volume changes due to water content changes. The second requirement is a change in the physical environment around the clay which will cause a change in the "moisture conditions," such as a change in climate or the erection of a building.

Tables I through IV present in tabular form a partial summary of the results obtained from the literature survey. These tables show examples of problems due to swelling in the United States (Table I), problems due to swelling in foreign countries (Table II), problems due to shrinkage (Table III), and due to cyclic movements (Table IV). Each table lists the reference, the location, the probable cause of the volume change (if known), the nature of the damage and/or the amount of movement, and the initial condition of the soil (if known) before swelling occurred (for the examples of problems due to swelling). The tables do not contain all the references, but rather attempt to present an overall view of the nature and magnitude of the problems associated with construction on "expansive" soils.[12]

The data in Tables I through IV show that damage to structures resting on soils exhibiting volume changes (exclusive of those resulting from frost heave, consolidation, shear, etc.) can be broken into three categories:

1. Damage due to <u>swelling</u> of the foundation soil;

---

12. See Holtz, 1959 for a map of western United States showing the location of sites encountered by the Bureau of Reclamation where "expansive soils" caused problems.

2. Damage due to shrinkage of the foundation soil;

3. Damage due to cyclic movement of the foundation soil.

Discussion of each of these categories is presented in the following sections. Typical references are cited.

B. Damage Due to Swelling (Tables I and II)

In this category, damage to the structure results from a more or less continuous swelling of the foundation soil until equilibrium is finally reached (Jennings, 1950, 1955; Collins, 1957). This type of movement is commonly reported in the Union of South Africa, Spain, and southwestern United States. In these areas the free surface water evaporation apparently greatly exceeds the annual rainfall (Salas and Serratosa, 1957) so that a permanent "moisture deficiency" normally exists in the ground. Thus buildings are usually placed on desiccated soils. The physical environment of the soil is immediately changed by the building, the most important change being the reduced rate of water evaporation from the foundation soil. Thus if water does move to the foundation soil, it will be imbibed by the desiccated soil rather than evaporated.

The movement of water to the foundation soil commonly occurs for the following reasons:

1. Capillary rise of water from the water table (Jennings, 1950, 1953). The depth to the water table may be several tens of feet.

2. Concentrated periods of high rainfall combined with poor drainage facilities around the structure (Terzaghi, 1950; Baracos and Bozozuk, 1957)

3. Seepage of water from faulty water mains, plumbing facilities, etc., (Means, 1959) or from large scale irrigation projects (Holtz and Gibbs, 1956),(or from lawn watering, etc.,)Dawson, (1953, Holtz, 1959).

4. Vapor flow due to cooler temperatures beneath buildings than in the surrounding uncovered soil (Jennings, 1950, 1953).

Other items of interest in this category are:

1. The amount of differential heave of houses is often 50 to 90 per cent of the average heave (Collins, 1957).

2. The time required from equilibrium (i.e., swelling stops) is often several years (Dawson, 1953; Means, 1959).

3. The depth of desiccated soil in these regions reaches 10 to 60 feet (Jennings, 1950; Wooltorton, 1954).

4. Some cyclic movements are observed, even though the general trend is one of swelling (Dawson, 1953).

5. There are several methods for dealing with expansive soils (Jennings, 1955; Means, 1959; Dawson, 1959; Templer, 1958; Lange, 1958; McDowell, 1959; Boardman, 1958; Holtz, 1959). These methods usually involve treating the soil (proper compaction, removal, prewetting, etc.), placing the structure on special foundations (piles, mats, three point support, grade beams, etc.), and/or using special types of structural framing.

C. Damage Due to Shrinkage (Table III)

Cases of damage due to shrinking soils are far less prevalent in the literature than that due to swelling. Notable cases of shrinkage are reported in southeast England (Ward, 1953; Skemption, 1954), Ottawa, Canada (Baracos and Bozozuk, 1957), and Kansas City (Taylor, 1954). In the former two cases, the clays normally exist in a very wet condition due to uniform rains and high water tables. Shrinkage occurs during an abnormally dry spell. The presence of vegetation, particularly trees, has been known to increase greatly the amount of shrinkage (Bozozuk, 1958; Bozozuk and Burn, 1960; Ward, 1953; Skempton, 1954; Felt, 1953; Barber, 1956).

D. Damage Due to Cyclic Movements (Table IV)

Cyclic movements refer to ground movements corresponding to normal seasonal weather changes (as opposed, for instance, to shrinkage due to prolonged drought). Damages due to cyclic movements appear to be less widespread than those due to either swelling or shrinkage although there are some reports of extensive damage (Baracos and Bozozuk, 1957 and Wooltorton, 1950). Climatic conditions most favorable to cyclic movements would be cool, wet seasons followed by warm, dry seasons.

E. Geographic and Climatic Distributions of Damage Due to Soil Volume Changes

One would expect that damage to buildings due to moisture changes in foundation soils could occur in all areas of the world having plastic surface soils. However, it is known that the climate of the area (besides influencing the type of soil) plays an important role in determining the initial moisture condition of the soil, i.e., humid climates lead to wet soils, arid climates lead to desiccated soils. In this respect, the most important aspect of climate appears to be the relationship between rainfall and rate of evaporation (or daily temperature). Thus climate is very important because it influences the probability of swelling versus shrinkage.

The role of climate in influencing the probability of a volume change or its magnitude is not so clear cut. While climate is certainly important with regard to cyclic movements and shrinkage, it does not appear to be as important to swelling as the reduced evaporation rate due to the erection of the building (for example, a house constructed on a clay compacted very dry of optimum would probably swell under almost all climatic conditions).

In conclusion, the primary effects of climate on volume changes (for a given soil) might be summarized as follows:

1. The ratio of rainfall to surface water evaporation will often determine the initial moisture condition, and hence whether swelling or shrinkage is more likely to be the major problem.

2. The yearly distribution and regularity of rainfall and the temperature during dry periods influence the shrinkage behavior of "wet" soils and cyclic movements. Hence a climatic rating such as that developed for the FHA (Building Research Advisory Board, 1959) is useful for predicting the likelihood of such volume changes.

V. LITERATURE SURVEY OF THE PREDICTION OF VOLUME CHANGE BEHAVIOR IN THE FIELD

A. Engineering Approach

In order to predict volume changes in the field with any degree of accuracy, one should carry out the following test program:

1. Determine the soil profile.
2. Obtain undisturbed samples.
3. Subject these samples to the same "moisture conditions" and loads as will occur in the field.
4. Measure the resultant volume changes.
5. Extrapolate the time-volume relationships obtained in the laboratory to the field.

The problems associated with such a program are:

1. The time and cost involved, particularly in obtaining representative undisturbed samples.
2. The difficulty in reproducing field "moisture conditions," even if one could predict how the field "moisture conditions" will change.
3. Difficulty of reproducing the stress conditions acting in the field.
4. Problem of extrapolating laboratory time - volume relationships to the field.

Several organizations have, however, developed somewhat simplified methods for predicting volume changes based on extreme moisture conditions, i.e., complete drying or complete "saturation" (e.g., Holtz and Gibbs, 1956; Jennings and Knight, 1957; Salas and Serratosa, 1957; McDowell, 1956). In the case of swelling, the Bureau of Reclamation (Holtz and Gibbs, 1956; and Salas and Serratosa, 1957) place undisturbed samples in consolidometers, apply the desired confining pressure, and then give the sample unlimited access to water. Jennings and Knight (1957) run consolidation tests on two undisturbed samples (one sample is always maintained at its natural water content, the other sample is allowed to swell under a low pressure and then consolidated in the normal manner), and ultimately make an approximate effective stress analysis[13] considering the weight of soil, the applied loads, and the depth to the water table. Lambe and Whitman (1959) also approach the problem with an effective stress analysis. McDowell (1956, 1959) saturate samples by capillary rise under a low pressure in a triaxial sell and then modifies the measured volume change to account for one versus three dimension swelling and for the value of the surcharge (by one set of curves of swell versus pressure assumed to hold for all swelling clays).

While the above approaches do involve some major simplifications (particularly with respect to field moisture conditions), they do consider the important factors of soil type and initial conditions and the influence of confining pressure.

B. Empirical Approach

The empirical approaches to predicting soil volume changes do not actually predict a volume change, but usually only indicate how potentially troublesome the soil might be as a foundation material. That is, these methods attempt to classify the soil in terms of possible volume changes for extreme changes in moisture conditions. These classifications are usually based on a correlation between field (or laboratory) behavior and some property of the soil, such as: Atterberg Limits, per cent clay size, linear shrinkage, activity, free swell, swell under certain loads, specific surface area, and water content at 85 per cent relative humidity.[14] Of the above the Atterberg Limits are most commonly used. The FHA (Building Advisory Research Board, 1959) combines soil plasticity with climate to arrive as a classification.

---

13. Only applicable to swelling due to the capillary rise of water from the water table to the soil and where there is no evaporation.

14. Holtz and Gibbs (1956), Altmeyer (1956), McDowell (1956), Williams (1957), De Bruyn, Collins, Williams (1956), Kantey and Brink (1952), Dawson (1959).

VI. LABORATORY TEST PROGRAM

A. Objectives and Background

The ultimate objective of the test program was the development of a field testing device to predict, in a general way, volume change behavior of soils when used as foundations for houses. In order to characterize volume change behavior the term Potential Volume Change (PVC) is introduced. The PVC of a soil refers to the maximum possible volume change that the soil could undergo from water content changes due to changes in the "moisture conditions" around the soil (i.e., swelling of dry soils or shrinkage of wet soils).

In order to accomplish this objective, it is first necessary to determine what property (or properties) of a soil can be measured by a device within two hours to arrive at a PVC. The second step is the correlation of the PVC, as measured by a device, with the expected behavior of the soil in the field in order to set up categories of behavior which can be used as guides for ascertaining the desirability of using the soil as foundation material.

There are several types of tests that might be used to obtain a PVC, such as classification tests or tests which measure volume changes due to swelling or pressures exherted by swelling soils. The latter approach was adopted by the Contractor. A preliminary test program was initiated which consisted of swelling magnitude and swell pressure measurements on several soils. This program showed that the amount of heave measured in a "standard" consolidation apparatus (sample thickness of 1/2 to 1 inch) at the end of two hours was not necessarily a good indication of the final amount of heave. On the other hand, it was found that the swell pressure at the end of two hours was generally a good indication of the final swell pressure. Consequently, a device to measure some sort of swell pressure appeared promising.

Two aporoaches could be used to correlate the PVC with field behavior: 1) obtain PVC measurements on soils of known field behavior and 2) correlate PVC measurements with some measurable property (or properties) of the soil which in turn can be correlated with field behavior from data presented in the literature. The Contractor had to follow the latter approach since no soils were obtained on which there was extensive field data.[15]

---

15. There were no conclusive data on the field behavior of the soils supplied by the FHA, although Mr. Henry furnished a general idea of the behavior of some of the soils.

B. Testing Procedures

1. Soil Samples

Table V lists the name, supplier, location and Unified Soil Classification designation of 13 soils tested at M.I.T. A complete set of tests were run on 11 of the 13 soils; however, all the data are reported on only 10 soils since the swelling data on the kaolinite-bentonite mixture did not appear relevant.

The FHA supplied 6 of the 10 soils for which extensive data are reported, four samples being highly plastic, one fairly plastic and one nonplastic. The remaining four soils ranged from a plastic clay to a relatively nonplastic silt.

2. Classification Test (See Appendix A for test details)

The specific gravity, Atterberg Limits (Liquid, Plastic and Shrinkage), grain size distribution, Field Moisture Equivalent (FME), Free Swell, and water content at 50 and 100 per cent relative humidity (R.H.) were determined for almost every sample. The water contents at 30 and 70 per cent R.H. were also determined on several of the more plastic clays.

3. Heave, Swell Pressure and Swell Index Tests (See Appendix A for test details and Figs. 6 and 7 for diagrams of the apparatus).

Description of Tests: Three types of tests were run as follows:

1. Heave (H): The amount of heaving expressed as per cent volume increase, of a sample in a consolidometer is measured for a low confining pressure (200 lb./sq.ft.) after the sample is given access to water.

2. Swell Pressure (SP): The pressure required to maintain absolutely constant volume of a sample in a consolidometer is measured after the sample is given access to water.

3. Swell Index (S.I.): The pressure required to reduce the heave to a small amount of a sample is measured after the sample is given access to water. The Swell Index is the value of pressure at the end of two hours. (Whereas the Swell Pressure test requires constant manual control during testing in order to maintain a constant sample volume, the Swell Index test requires no adjustment and hence is a simpler test.)

The above tests were run on samples compacted in consolidation rings. Readings were usually taken until equilibrium was reached.

Preparation of Samples: The samples were compacted in the consolidometer rings at three different Relative water contents. The Relative water content of a sample of soil refers to the relation between the actual water content of the sample and the water contents (which might be called "standard" water contents) at which the soil exhibits certain well-defined properties (such as at the Liquid Limit, the Plastic Limit, the Shrinkage Limit, or the FME) or is in equilibrium with moist air having a certain R.H. (such as 100 per cent R.H. or 50 per cent R.H.). The meaning of Relative water content is illustrated in Fig. 8, which shows the values of some "standard" water contents of a given soil (w_l, FME, w_p, w_s, w_{100} and w_{50}) of which, FME, w_p, w_{100} and w_{50} are of particular interest, and the water content values of four samples (u, x, y, z) of this soil. Sample u has a Relative water content halfway between w_p and the FME, etc.

The three Relative water contents at which samples were tested are termed Dry, Moist, and Wet and are equal to the following:

1. Dry = w_{50} = water content at 50 per cent R.H.

2. Moist = w_{100} = water content at 100 per cent R.H.

3. Wet = w_p = Plastic Limit.

Dry, Moist and Wet samples were chosen to represent approximately an air-dried water content, a very low field water content and a very high field water content respectively. Thus samples were compacted in the laboratory at water contents which are thought to cover the range of water contents from which a field sample might swell.

The samples were compacted dynamically at compactive efforts of approximately Modified AASHO, one half of Modified AASHO, and Standard AASHO for Dry, Moist and Wet[16] samples respectively. The compactive effort was increased with decreasing water content since: 1) it requires more effort to compact dry samples to the same density as wet samples and 2) dry samples are likely to have high densities in the field due to the shrinkage stresses set up during drying.[17]

C. Test Results

The results of the classification tests are presented in Table VI, which also include values of activity and per cent volume change, (calculated from Eq. 2) $\Delta V/V$, from drying a saturated sample

16. The Plastic Limit approximates optimum water content for Standard AASHO compaction.

17. The Moist samples usually had densities equal to 80 to 90 per cent of the density at the Shrinkage Limit.

from the F.M.E.[18] to the Shrinkage Limit. The activity has been used to indicate the expansiveness of soils. $\Delta V/V$ approximates the largest volume change likely to occur in the field from shrinkage. The Atterberg Limits have also been plotted on a Plasticity Chart in Fig. 9.

The results of the Heave (H), Swell Pressure (SP) and Swell Index (SI) tests are summarized in Table VII for ten soils. The H, SP, and SI (and heave during SI tests) values, the ratio of the two hour to final values of H and SP, and the average molded water contents and dry densities are presented for Dry, Moist and Wet molded water contents.[19] Correlations among some of the above values are shown in the following figures:

Figure 10 - Plasticity Index (PI) vs. Volume Change - Drying from F.M.E. to Shrinkage Limit $\Delta V/V$ and Final Heave (H).

Figure 11 - H vs. $\Delta V/V$.

Figure 12 - H vs. Water Content at 100 per cent R.H. (w_{100}) and Free Swell.

Figure 13 - H vs. Final Swell Pressure (SP).

Figure 14 - Swell Index (SI)[20] vs. PI.

Figure 15 - SI vs. $\Delta V/V$.

Figure 16 - SI vs. H.

Figure 17 - SI vs. SP.

---

18. The F.M.E. is a measure of the largest water content that an unloaded soil would normally attain in the field. Lightly loaded soils might reach about 75 per cent of the F.M.E. (Wooltorton, 1954). Out test data show that the water content of initially moist samples after swelling under 200 lb. sq.ft. averaged 93 per cent of the F.M.E.

19. The measured values of H, SP, and SI have been adjusted to correspond to the average molded water content for each group of tests to eliminate scatter due to nonuniform water contents. The average water content was almost always within one per cent of the desired Relative water content.

20. The <u>two</u> hour reading is always reported.

Typical plots of time versus heave and pressure are presented in Figs. 18 and 19 for the Iredell and Keyport soils.

D. Discussion of Test Results

1. Soils

Tables V and VI and Fig. 9 show that the soils tested:

(1) Range from nonplastic silts to very plastic clays, with the latter predominating.

(2) Represent many geographical locations (5 states and 1 foreign country).

(3) Usually fell close to, but above, the A line on the Plasticity chart (Fig. 9).

At least two of the soils, Iredell clay and Houston Black clay, are known to cause considerable trouble as foundation soils.

2. Heave and Swell Pressure Tests

The data in Tables VI and VII and Figs. 10 through 13, 18 and 19 show:

(1) The time to reach equilibrium is far greater for Heave than Swell Pressure tests (Figs. 18 and 19). The ratio of the two hour to the final value for the Heave tests ranged from 45 to 100 per cent, and was commonly 50 to 75 per cent for the more plastic soils. On the other hand, the ratio ranged from 85 to 100 per cent (except for one test) for the Swell Pressure tests. (Table VII.)

(2) The amount of Heave of a given soil almost always increases with decreasing Relative water content.[21] The Swell Pressure values always increased in going from Wet to Moist, but remained approximately constant in going from Moist to Dry. (Table VII.)

---

21. It is interesting to note that for the more plastic soils the water content approximately doubles in going from the Dry to the Moist and from the Moist to the Wet Relative water contents.

(3) The Plasticity Indices of soils are well correlated with volume changes due to shrinkage (Fig. 10). A good correlation also exists between PI and Heave, for comparable Relative water contents, at least for the lower range of PI's (Fig. 10). Consequently, volume changes due to shrinkage and swelling are well correlated (as shown in Fig. 11), i.e., high swelling soils can also be high shrinking soils.

(4) Heave values can also be related to the water content at 100 per cent R.H. and Free Swell (Fig. 12), although not as well as with PI.

(5) An excellent correlation exists between Heave and Swell Pressure, independent of of molded water content (Fig. 13), at the lower values of Heave, but there is considerable scatter at high values.

3. Swell Index Tests

The data in Tables VI and VII and Figs. 14 through 19 show:

(1) The time to reach equilibrium is somewhat greater for Swell Index than Swell Pressure tests, but considerable smaller than for Heave Tests (Figs. 18 and 19). However, only the two hour value of Swell Index is of interest.

(2) The Swell Index value of a soil always increases in going from a Wet to a Moist Relative water content. (Table VII.) The value can either increase or decrease in going from a Moist to a Dry Relative water content. (A decrease is noted if the dry density for a Dry sample is lower than that for a Moist sample, such as occurs for the Enon Silt Loam, the Keyport soil, Boston Blue clay, and the Vicksburg Loess.) The values of Swell Index for Dry and Moist samples have been treated as one in the same when establishing correlations, although these values are sometimes quite different.

(3) The Swell Index Test yields a good indication of the PI and $\Delta V/V$ of a soil (Figs. 14 and 15) up to values of about 35 per cent if the Relative water content of the sample tested is known.

(4) The Swell Index test can be used to estimate within about 50 per cent accuracy the Heave of a sample compacted at a comparable water content and density (Fig. 16), independent of the Relative water content.

(5) The Swell Index test can estimate with reasonable accuracy the Swell Pressure of a sample compacted at a comparable water content and density (Fig. 17), independent of the Relative water content, up to Swell Pressures of about 3000 lb./sq.ft. There is noticeable scatter at higher Swell Pressure values.

(6) Properly run Swell Index tests (with apparatus shown in Plates 1 through 4) are very reproducible as evidenced by the following data from tests on identical samples of 3 soils compacted at "air dried" water contents (which approximate w_{50}):

Soil	No. of Tests	Swell Index (lb./sq.ft.)
Iredell clay	3	6425 ± 225
Houston Black clay	3	6075 ± 75
Keyport soil	2	2350 ± 75

In summary, the data indicate that a Swell Index value can be well correlated with PI, $\Delta V/V$, Heave and Swell Pressure if: 1) the Relative water content of the sample tested is known and 2) the value of Swell Index for Dry or Moist samples does not exceed about 3000 lb/sq.ft. (for Wet samples, the scatter is still tolerable for the highest values measured). Hence, for the less plastic soils, good correlations exist; whereas for the most plastic soils there is some noticeable scatter. In other words, the Swell Index test can note differences among the properties of various lean to nonplastic clays but cannot distinguish with great accuracy differences among very highly plastic clays (PI of over 35 per cent).

E. Correlation of Swell Index with Potential Volume Change

Each soil was classified into one of four categories (non-critical, marginal, critical and very critical) based on the following correlations reported in the literature:[22]

1. Heave from air-dried condition (Holtz and Gibbs, 1956 - data changed to compare to average of Dry and Moist Heave).

2. Plasticity Index (Holtz and Gibbs, 1956; Holtz, 1959).

3. Water content at 85 per cent RH (De Bruyn Collins and Williams, 1956).

4. Plasticity Index and Activity (Williams, 1957).

5. Linear shrinkage, FME to Shrinkage Limit (Altmeyer, 1956 - data changed to compare to $\Delta V/V$, FME to w_s).

Since a given soil may fall into different categories for the different criteria of classification, a weighted average should determine the final classification. The resulting classifications are shown in Table VIII. A numerical PVC classification system has been set up as follows:

Category	Rating
Noncritical	< 2
Marginal	2 - 4
Critical	4 - 6
Very Critical	> 6

Table VIII shows that there are 3, 1, 3 and 3 soils respectively in categories of Noncritical through Very Critical.

Finally, Swell Index values were established that would best divide the measured Swell Index values for the soils into their proper category. The resulting plot of Swell Index versus Potential Volume Change rating is shown in Fig. 20.

The results of "reclassifying" the soils from the measured Swell Index values using Fig. 20 are presented in Table IX. The

---

22. It is interesting to note that the per cent clay size and the Shrinkage Limit did not correlate at all well with the other methods of classification.

ratings by tests at Dry, Moist and Wet Relative water contents are shown, along with the average rating and how it compares with the ratings obtained in Table VIII. The data in Table IX show an excellent agreement between the average of the measured ratings and those obtained from correlation with Heave tests, PI, etc. Moreover, 22 of the 27 Swell Index tests on the 10 soils yielded values falling in the correct category.

In summary, the Contractor feels that Swell Index values can yield a considerable amount of valuable information about a soil, such as an indication of the PI, the potential heave from a given water content and density, and, of greatest interest to the FHA, a PVC rating. It must be cautioned, however, that the above correlations are based on tests that were carefully executed. Furthermore, and of utmost importance, a knowledge of the Relative water content of the sample tested is required for many of the correlations, including the PVC rating.

VII. UTILIZATION OF SWELL INDEX DEVICE

A. General

The main purpose of the Swell Index device is to aid personnel in the identification of soils that might be potentially troublesome because of volume changes due to swelling and/or shrinking if used as foundation soils for houses. A rating system, called the Potential Volume Change (PVC), has been developed which classifies soils according to four categories, as follows:

PVC Rating	Category
< 2	Noncritical
2 - 4	Marginal
4 - 6	Critical
> 6	Very Critical

The PVC of a soil is obtained by running a Swell Index test on the soil and then entering a plot which relates values of Swell Index with PVC. The Swell Index test is essentially a measurement of the pressure exerted by a sample of compacted soil when it tries to swell against a restraining force after being wetted. A schematic drawing of the device is shown in Drawing 1. Photos of the apparatus are shown in Plates 1 through 4. The value of pressure (in lb./sq.ft.) 2 hours after adding the water is the Swell Index of the sample.

The Swell Index device, in addition to yielding PVC values, can be used to estimate the Plasticity Index and shrinkage behavior of soils. It may also be used to obtain the relationship between water content and the swelling and swell pressure characteristics of a soil.

In order to correlate Swell Index with PVC, Plasticity Index, etc., the operator must know the Relative water content of the sample tested, i.e., how wet or dry is the sample? This is required since the behavior of soil depends to a large degree upon the amount of moisture in the soil.

A summary of the steps in running a Swell Index test for determining the PVC of a soil is presented below:

1. Determine the Relative water content of the sample to be tested. If it does not equal one of three Relative water contents for which a correlation between Swell Index and PVC has been established, it is necessary to adjust the water content of the sample until it does. (In many cases, this adjustment can be most easily made by air drying the sample.)

2. Compact the sample in the Swell Index device and assemble the device.

3. Add water to the sample.

4. At the end of 2 hours read the dial which records the force exerted by the sample. (It may be more convenient to make a plot of dial reading versus time.)

5. Obtain Swell Index from the known relationship between dial reading (at 2 hours) and pressure (lb./sq.ft.).

6. Obtain PVC from a figure relating Swell Index and PVC.

B. Sample Preparation and Testing

1. Log of Test Information

A data sheet, such as illustrated in Table XI, should be used to tabulate the information obtained from the Swell Index test and other pertinent information, such as location of sample, visual classification of soil, etc.

2. Determination of Relative Water Content

The Relative water content of the sample to be tested must be established in order to choose the appropriate compaction procedure and the appropriate correlation between Swell Index and PVC.[23]

Table X suggests methods for estimating the Relative water content of clay samples. The correlation between Swell Index and PVC has only been established for three Relative water contents (called Dry, Moist and Wet). It is therefore necessary to bring the sample to be tested to one of these three Relative water contents. Best results will be obtained if the sample is brought to the Dry-Moist Relative water content range. This can be easily accomplished by simply air-drying the soil (after breaking the soil into small, 1/4 to 1/2 inch, lumps). The time required for drying will depend upon the relative humidity of the air, the amount of moisture initially in the soil and the plasticity of the soil. It should not take more than several hours to one day in most cases. Before testing, any stones larger than 1/8 inch should be removed and all dried clay lumps completely broken down (with the compaction hammer or comparable impliment).

---

23. This is also true for correlations with Plasticity Index (Fig. 14) and shrinkage (Fig. 15).

It is emphasized that the Relative water content of the sample must be closely controlled for best results. For example, if a sample at an Intermediate Relative water content (Table X) were tested and either the Dry-Moist or Wet correlation used, the PVC rating could easily be in error by a factor of 2. It is also important to have the sample at an uniform moisture content.

The above comments concerning the Relative water content do not have to be considered when using the Swell Index test to estimate Heave and Swell Pressure values. In this case, the operator should place the sample at water content and density values comparable to those which will be used in the field. (See Appendix B for Compaction procedures using compactive efforts ranging from Modified AASHO to Standard AASHO).

3. Compaction and Placement of Sample in Device

A detailed, step by step, procedure for sample compaction and placement of the sample in the Swell Index device is presented in Appendix B.

4. Determination of Swell Index

After starting the test (i.e., a pressure of 200 lb./sq.ft. applied to the sample and water then added to the sample), the operator must only determine the proving ring dial reading at the end of <u>two</u> (2) hours. The device should not be jarred during the two hour interval, as this may alter the dial reading. The two hour dial reading is converted to pressure (lb./sq.ft.), which equals the Swell Index, from Fig. 21.

Although only the two hour dial reading is required, it may be advantageous to take several readings in order to obtain a plot of dial reading versus time, (to log scale) as shown in Fig. 22. (Readings at time intervals about double or triple the preceding time yield satisfactory plots, (e.g., at 1, 2, 5, 10, 30, 100, 200, etc. minutes). From the time plot, the operator can:

(1) Interpolate to find the two hour reading if it was not taken.

(2) Check the progress of the test to see if it is working properly.

(3) Obtain the Swell Index value in less than two hours for samples which reach an equilibrium pressure quickly, such as often occurs with samples of the less plastic clays (see, for example, Fig. 19 or 22).

(4) Obtain a dial reading in less than two hours which indicates a Critical or Very Critical PVC rating for samples of very highly plastic soil.

Thus a time plot may reduce the required testing time, help check the reliability of the test, and give the operator a "feel" for how the pressure varies with time for different moisture conditions and soil types. The time-pressure relationship can be used in a relative way to indicate the rapidity with which volume changes may occur in the field.

C. <u>Interpretation of Swell Index Values</u>

1. To Obtain PVC Ratings

(The sample tested must have a Dry, Moist, or Wet Relative water content.)

The PVC of the soil is obtained from Fig. 20. From the value of Swell Index on the vertical axis, move horizontally to the appropriate Relative water content line (Dry and Moist or Wet), then down vertically to obtain the PVC rating. Note the PVC value (to the nearest 0.1 units) and the category (Marginal, Critical, etc.).

2. To Obtain Estimates of Plasticity Index and Shrinkage

Same procedure as for obtaining the PVC, except use Figs. 14 and 15.

3. To Obtain Estimates of Heave and Swell Pressure

Possible swelling and swell pressure values for a field sample can be estimated from a Swell Index value if the sample tested has the same water content (very important) and density (less important) as that in the field. In other words, if a sample of soil in the field <u>obtains access to water</u>, the resulting amount of heave or swell pressure of the sample can be estimated by running a Swell Index test.

To estimate the swell pressure, use Fig. 17 which shows experimental data for ten soils relating Swell Index to Swell Pressure (this correlation can be assumed to hold for all water contents).

To estimate the heave, use Fig. 16 which shows experimental data for ten soils relating Swell Index to Heave (per cent change in height of sample) under a 200 lb./sq.ft. surcharge. This yields the potential heave if the sample is confined by a pressure of only 200 lb./sq.ft. In order to obtain an estimate of the amount of heave for larger confining pressures, the following empirical approach may be used for a given Swell Index value (this method is illustrated in Fig. 23).

(1) Construct a graph of heave (%) on vertical axis versus pressure (lb./sq.ft.) on horizontal axis.

(2) On the graph, plot the value of heave for a 200 lb./sq.ft. pressure that corresponds to the Swell Index value, i.e., from Fig. 16.

(3) On the horizontal axis, (zero heave) plot the value of Swell Pressure (lb./sq.ft.) that corresponds to the Heave under a 200 lb./sq.ft. surcharge by using Fig. 13 (or the Swell Pressure may be obtained from the Swell Index value and Fig. 17).

(4) At a pressure equal to one-half (0.5) of the Swell Pressure, plot a heave value equal to one-quarter (0.25) of the Heave under the 200 lb./sq.ft. surcharge.

(5) At a pressure equal to one-quarter (0.25) of the Swell Pressure, plot a heave value equal to one-half (0.50) of the Heave under the 200 lb./sq.ft. surcharge.

(6) Draw a curve through the four points which have been plotted. This plot represents the relationship between heave and the confining pressure that has been applied during swelling.

The above procedure to correct for increased confining pressure will usually yield values of heave that are slightly high.

D. <u>Comments</u>

1. Test Procedure

a. A very careful control of the Relative water content is required for a reliable estimate of the PVC of a soil. Dry or Moist samples will probably yield the best results.

b. The compaction procedure should be followed accurately, since a density variation will cause a change in the Swell Index value. For example, if the height of the compacted sample is not 1/8 to 1/4 inch above the compaction ring before trimming, the sample should be taken out and recompacted. Careful trimming of the sample is also necessary so that the bottom of the sample fits tightly against the bottom porous stone.

c. It is a good idea to check the progress of the test and to ensure that the device does not get jarred.

d. Check the PVC rating from the Swell Index test with your visual classification of the soil. Does the PVC rating look reasonable?

e. Swell Index tests at Dry, Moist and Wet Relative water contents might be run on a soil for which a more accurate PVC rating is desired.

2. Reliability of Test Results

(Assuming test run properly):

a. The correlations that have been established between Swell Index and PVC, Plasticity Index, Heave, etc., based on test data on 10 soils, ranging from sandy silts to very plastic clays, which fall close to the A-line. The reliability of the correlations for soils which do not fall close to the A-line is not known. Henceforth, discussion is restricted to soils similar to those tested.

b. PVC ratings from a <u>single</u> test on a soil should be reliable: to within about 0.5 units for Marginal and Noncritical soils; to within about 1 unit for Critical soils; and to within about 2 units for Very Critical soils. PVC ratings from the average of Dry and Moist samples should reduce the above errors by a factor of 2. (See Table VIII.)

c. Heave values from Swell Index test may be as much as 50 to 100% in error due to the scatter in experimental data (Fig. 16). However, the Swell Index test should indicate the order of magnitude of potential field swell (if corrections for confining pressures are made) and it should show a reliable indication of the variation in swelling behavior of a soil with changes in water content.

d. Swell Pressure values from Swell Index tests should be reasonably accurate (Fig. 17) up to Swell Index values of 3000 lb./sq.ft. (Swell Pressure of 4500 lb./sq.ft.), but less reliable for higher values.

VIII. RECOMMENDATIONS

The Contractor makes the following recommendations in order to: improve the Swell Index test procedure; check and, if required, improve the correlation between Swell Index and PVC, Plasticity Index, Heave, etc., check and, if required, improve and/or modify the correlation between PVC and the volume change behavior of soils in the field; and strengthen our knowledge of the factors which influence soil volume changes and how one may deal with or modify such volume changes.

1. Run a set of tests comparable to those which have been performed (i.e., classification, Heave, Swell Index, etc.) on several soils which do <u>not</u> fall close to the A-line, such as OH and MH soils.

2. Run a series of Heave, Swell Pressure and Swell Index tests on these 10 soils already tested and on several OH and MH soils (from above) at the Intermediate Relative water content (Table X) to see if Swell Index tests can be run at any known Relative water content between Dry and Wet (by means of interpolation).

3. Accurate PVC data (from 2 or more Swell Index tests at 2 or more Relative water contents) should be obtained on soils of <u>known</u> field behavior throughout the United States in order to test the reliability of the PVC rating system. At each site, at least two or three years after the building has been erected, the following information should be gathered: 1) soil profile and PVC values, condition of soil at time of construction and present moisture and density conditions and amount of volume change, 2) type of building and foundation, condition of building (i.e., cracks, differential movements, etc.), and 3) climatic date (particularly rainfall and evaporation patterns), pedologic data, and hydrologic data (location of water table, presence of leaky water mains, occurrence of lawn watering, etc.). Ten to twenty well selected "case studies" should yield a positive check on the accuracy of the PVC rating system. A record should also be kept in the future of the condition of the buildings, etc., on jobs where the PVC rating system was used.

4. Maintain records of the difficulties which arise during the performance of Swell Index tests and the suggestions operators offer so that the testing procedures and/or equipment can be modified to yield better results. Modifications might include: better methods for estimating Relative water contents, a longer or shorter testing period, modification of the device to run Swell Pressure tests, and a more elaborate method for estimating heave values.

5. In areas where the Swell Index device is likely to be extensively used, at least one man should be trained to operate the device and he should thoroughly study this report so that he is familiar with the various facets of the problems encountered with expansive soils.

6. A long range research program on the general subject of expansive soils should include the following topics (among others).

   a. Means of dealing with expansive soils by treatment (e.g., with additives or by proper placement), by control of water movements (e.g., placement of drains or moisture barriers), and by special foundations or structural framing.

   b. Factors governing the rate of swelling and shrinking and development of theories for predicting the rate of swelling and shrinking.

   c. Investigation in a quantitative manner of the various means by which moisture moves.

   d. Collection and correlation of data on the effect of climate on soil volume changes.

APPENDIX A

TEST PROCEDURES FOR LABORATORY TEST PROGRAM

A. Classification Tests

1. Soil Preparation. Soil was air-dried and ground (with mortar and pestle if sandy, with corn grinder if no sand) and scalped on a No. 40 sieve for all tests except the grain size distribution.

2. Specific Gravity, Atterberg Limits and Grain Size Distribution. The above tests were run in accordance with Lambe (1951).

3. Field Moisture Equivalent (F.M.E.). The F.M.E. equals the water content of the soil at which a drop of water will just disappear from the surface of the moist soil in 30 seconds. The test was run in accordance with ASTM D426-39.

4. Free Swell Test. Ten cc of air-dried minus No. 40 sieve soil is slowly poured into the top of a 50 or 100 cc graduate filled with water and the swelled volume of the soil measured after it comes to rest at the bottom.

$$\text{Free Swell } (\%) = \frac{\text{Final Volume} - \text{Initial Volume}}{\text{Initial Volume (10 cc)}} \times 100.$$

5. Water Content at Various Relative Humidities. Sample of air-dried (or oven-dried, if necessary) soil was placed in an enclosure kept at approximately constant relative humidity and at room temperature. Sample weighted over a period of several days until constant weight recorded. Then water content of equilibrated sample measured. The relative humidities were obtained by placing the following saturated chemical solutions at the bottom of the enclosure:

   a. 100 Per Cent - pure water

   b. 70 Per Cent - $SrC\ell_2$-$6H_2O$

   c. 50 Per Cent - $Ca(NO_3)_2 \cdot 4H_2O$

   d. 30 Per Cent - $CaC\ell_2 \cdot 2H_2O$

B. Heave, Swell Pressure and Swell Index Tests

1. Soil Preparation. Soil was air-dried, ground, and scalped on a No. 10 sieve. Desired amount of distilled water hand-mixed with air-dry soil and mixture equilibrated in closed container for at least 24 hours.

2. Compaction. Samples were compacted dynamically by a 5.5 pound hammer (Standard AASHO hammer) falling 12 inches into 2.75 inch diameter by 0.85 inch high (or sometimes, 2.50 inch diameter by 1.0 inch high) consolidation rings with collars in the following manner.

Water Content	Nominal Compactive Effort*	No. of Layers	Blows per Layer
Dry	Modified AASHO	3	7-7-8
Moist	1/2 Modified AASHO	3	4
Wet	Standard AASHO	1	5

After compaction, samples were leveled off and weighted and then either: partially extruded from the ring such that a sample height of about 0.6 inches remained, or left as is in cases where an insert was placed at one end of the ring during compaction so that a sample height of about 0.6 inches was already obtained. Samples were then set in consolidometer units with top and bottom porous stones (so that water can enter both ends of the sample) as shown in Fig. 6.

3. Heave Tests. Samples in the consolidometer units are placed on platform type consolidation units (Fig. IX-3, Lambe, 1951), a pressure of 200 lb./sq.ft. applied, the sample height measured, and a .0001 inch/division extensometer dial attached in order to measure changes in the height of the sample. Water added to the sample and readings of time and change in sample height taken until equilibrium reached (few hours to several days). Plot of per cent Heave (change in height divided by initial height) versus log time in hours is made.

4. Swell Pressure Tests. Samples in the consolidometer units are placed in an apparatus as shown in Fig. 7. A pressure of 200 lb./sq.ft. is applied to the top of the sample by means of the hand operated screw jack control, the height of the sample measured, and a .0001 inch/division extensometer dial (Dial B) attached to the top of sample in order to measure changes in the height of the sample. Water is

---

* For samples without an insert during compaction, the compactive efforts were actually about 75% of Modified AASHO, Standard AASHO, etc.

added to the sample and the pressure P is then continuously adjusted by the screw jack control to maintain a constant sample height as indicated by Dial B. Measurements of the pressure required to keep constant volume versus time are taken until equilibrium is reached (usually a few hours). Plot of Swell Pressure (lb./sq.ft.) versus log time in hours is made.

The pressure P is obtained by measuring the radial deflection of a calibrated proving ring (approximately 4.5 inch diameter, 0.25 inch thick, and 1.5 inch wide) by means of Dial A (.0001 inch/division). The sensitivity of the ring is about 5lb./dial division (i.e., the vertical diameter of the ring decreases .0001 inch per 5 lb. of load). The operation of the screw jack control to maintain constant sample volume is required to compensate for the deflection of the proving ring and the expansion of the frame which attaches the jack and proving ring to the base containing the sample.

5. Swell Index Test. The Swell Index test is the same as the Swell Pressure test with one exception: The screw jack is not adjusted. Consequently, as the sample pushes against the top porous stone, the resultant deflection of the proving ring and expansion of the frame allow a small expansion of the sample. This expansion (approximately 1-2 per cent per 5000 lb./sq.ft. pressure) reduces the maximum pressure exerted by the soil.

   Measurements of pressure versus time are taken until equilibrium is reached. The amount of sample expansion has also been recorded (but this is not necessary). The time required for equilibrium is slightly longer than that required by the Swell Pressure tests. A plot of pressure versus log time is made. The Swell Index is the pressure at the end of two hours.

APPENDIX B

PROCEDURES FOR SAMPLE COMPACTION AND TESTING WITH THE SWELL INDEX DEVICE

A. Equipment (Drawings 1-5 and Plates 1-4

1. Swell Index Device
2. Compaction Hammer and Cylinder
3. Knife and Straight Edge
4. Water (squirt bottle)
5. Two Dry Porous Stones
6. Brush (optional)
7. 1/8 Inch Dia. Pin
8. Teaspoon
9. No. 10 Sieve (optional)
10. Wrenches

B. Preparation for Compaction

() Refers to Part No., Drawing 1.

1. Disassemble Swell Index Device, with exception of Rods (7) which can remain screwed into the Base (1). Place Proving Ring (13) and Top Bar (17) where it will not be jarred during compaction.

2. Place Compaction Ring (3), with letters CO at the top, on the Base and so that the No. 1 on the Base is aligned with No. 2 on the Compaction Ring. Place Spacer Ring (2), on Compaction Ring with letters CO at the bottom (radial grooves are on top) and aligned with the letters CO on the Compaction Ring. Note: In Swell Index Devices wherein the holes for the bolts attaching the Compaction and Spacer Rings to the Base are prefectly symetrical, this careful alignment is not necessary). Insert the 3 Bolts (5) and tighten firmly.

3. The Relative water content of the sample must be ascertained (see Table X). If the PVC of the soil is required, the Relative water content must be adjusted (if needed) to a Dry, Moist or Wet Relative water content.

4. The sample should not contain any stones or dried clay lumps that will not pass through a No. 10 sieve (.07 inch diameter).

C. Compaction

1. The compaction procedures to follow are shown below, along with the compactive efforts and their relation to Standard and Modified AASHO compaction.

Relative Water Content	No. of Layers	No. of Blows Per Layer	Compactive Energy* (ft.lb./cu.ft.)
Dry	3	7	55,000 (Modified AASHO)
Moist	3	4	31,000 (1/2 Modified AASHO)
Wet	1	5	13,000 (Standard AASHO)

* Compactive Energy (ft.lb./cu.ft.)

$$= \frac{\text{No. Layers} \times \text{No. Blows/Layer} \times 5.5 \text{ lb} \times 1 \text{ ft.}}{.00215 \text{ ft}^3}$$

$$= 2.55 \times \text{No. Layers} \times \text{No. Blows/Layer}$$

2. Place the apparatus on a sturdy support or on the floor for compaction.

3. For the compaction procedure requiring one layer, press the soil with the Hammer into the Rings until the top of the soil is about 1/8 inch below the top of the Spacer Ring before applying the blows. During compaction space the blows evenly over the surface by shifting the location of the hammer after each blow. Make sure that the Sleeve for the Compaction Hammer rests inside the Spacer Ring so that the Hammer does not strike the Spacer Ring. Make sure that the top of the compacted sample after compaction is 1/8 to 1/4 inch above the Compaction Ring.

4. For the Compaction procedure requiring three layers, add 2 to 3 heaping teaspoons per layer and press the soil with the Hammer to smooth and firm-up the surface before applying the blows. (This reduces the amount of soil "jumping" out of the mold during compaction.) Each layer of soil after compaction should have a thickness of about 1/4 inch so that the final compacted thickness before trimming is 1/8 to 1/4 inch above the Compaction Ring. (If the soil is below the level of the Compaction Ring, remove soil and recompact.) The top of the first and second layers should be scarified (draw knife across top several times to loosen the top 1/16 inch of soil) to ensure a good bond between successive layers.

5. Disassemble. Remove the 3 Bolts. Rotate Spacer Ring (to break bond between Ring and soil) and remove.

6. Trim the top of the sample. Start by trimming the edges of the sample first, gradually working toward the center of the sample. When the sample is almost level, do the final leveling by drawing a metal straight edge over the sample.

The final surface must be level and firm ("holes" in the surface of the sample should be filled again with soil with light tamping).

7. Rotate Compaction Ring (to break bond between Base and soil) and remove. Clean soil from Base and from holes in Compaction and Spacer Rings.

D. Assembly and Start of Test

1. Place Spacer Ring on Base with No. 2 on the Ring (radial grooves are on top) aligned with No. 1 on the Base. Place a dry Porous Stone (the stone must be dry throughout) in the Spacer Ring (top of Porous Stone should be level with top of Spacer Ring). Place Compaction Ring, with recessed soil on top, on Spacer Ring and Porous Stone with letter S on Compaction Ring lined up with letter S on Spacer Ring (see note under B, 2 with regard to alignment). Insert 3 Bolts and tighten firmly.

2. Place O-Ring (20) and screw Lucite Container (6) firmly down onto O-Ring to ensure water tightness.

3. Place a dry Porous Stone on top of sample inside Compaction Ring. Place the Cover (4), with the radial grooves on the bottom, on the Porous Stone.

4. Place Top Bar (17) with Proving Ring on the steel Rods. (Be sure that Adjustable Rod (8) does not strike the Cover, as jarring of the Proving Ring Dial may be harmful), add Washers (19) and Nuts (18) and tighten firmly.

5. Push up on Proving Ring Dial to see that it appears to work properly. The Dial should move about one division (.0001 inch) per 5-6 pounds/force.

6. Set Proving Ring Dial to zero by moving the face of the Dial, then clamp face to dial. Turn Adjustable Rod down into groove on top of the Cover until a Dial reading corresponding to 200 lb./sq.ft. (about one division, see Fig. 21) is attained. Tighten Adjustable Nut (9) on Adjustable Rod so that Adjustable Rod has no play. Check to see that pressure on top of sample is still 200 lb./ sq. ft. as indicated by the Dial reading.

7. Record the time and Proving Ring reading on the data sheet. Add water to the sample by squeezing water from the squirt bottle into one of the three vertical (0.14 inch diameter) holes located at the top of the Compaction Ring until the water level in the Lucite Container has reached the Cover. (This procedure is used to reduce the amount of air entrapped in the Porous Stones and thus to ensure that the sample has access to water over its entire top and bottom surfaces.)

APPENDIX C

SPECIFICATIONS AND DRAWINGS FOR SWELL INDEX DEVICE

Photographs of the Swell Index device are shown of Plates 1-4. Drawings 1-5 show detailed shop drawings from which a Swell Index device can be constructed. Table XII specifies the materials to be used for each part and lists possible suppliers (the addresses of these suppliers are listed in Table XIII). The following comments are also pertinent:

1. The advantage of D-alum over brass lies in its reduced weight. However, brass is probably more durable. Brass or stainless steel might be preferable for the Compaction and Spacer Rings since they will receive the toughest use.

2. The Proving Ring and Dial and related connections may be designed by a manufacturer if they meet the following criteria:

    a. The Proving Ring deflects .0001 in per 5 to 6 lb. applied force (i.e., .0017 to .0020 in. deflection for 100 lb. force) and must have a capacity of at least 1000 lb.

    b. The Dial reads .0001 in. per division.

    c. Connections to Top Bar must be tight and provisions for Adjustable Nut and Rod made.

BIBLIOGRAPHY

1. Aitcheson, G. D. (1956). "The Strength of Quasi-Saturated and Unsaturated Soils in Relation to the Pressure Deficiency in the Pore Water," Proc. 4th International Conf. Soil Mech. and Foundation Engr., London, pp. 135-139.

2. Allen H. and Johnson, A. W. (1936). "The Results of Tests to Determine the Expansive Properties of Soils," Proc. HRB, Vol. 16, pp. 220-233.

3. Altmeyer, W. T. (1956). Discussion of Paper 2814. Trans. A.S.C.E., Vol. 121, p. 666.

4. Baracos, A. and Bozozuk, M. (1957). "Seasonal Movements in Some Canadian Clays," Proc. 4th International Conf. Soil Mech. and Foundation Engr., Vol. I, p. 264.

5. Barber, E. S. (1956). Discussion of Paper 2814. Trans. A.S.C.E., Vol. 121, p. 669.

6. Building Research Advisory Board (1959). "Design Criteria for Residential Slabs-On-Ground," National Academy of Sciences, National Research Council, Pub. 657, Washington, D.C.

7. Boardman, V. R. (1958). "Design, Construction and Behavior of a Reinforced Brick Building on Heaving Clay," S. African Inst. of Civil Engineers, "Symposium on Expansive Clays," Johannesburg.

8. Bolt, G. H. (1956). "Physico-Chemical Analyses of the Compressibility of Pure Clays," Geotechnique, Vol. 6, No. 2.

9. Bozozuk, M. (1958). "Volume Changes Measured in the Leda Clay," 12th Canadian Soil Mech. Conf., National Research Council.

10. Bozozuk, M. and Burn, K. N. (1960). "Vertical Ground Movements Near Elm Trees," Geotechnique, Vol. X, No. 1, March.

11. Collins, L. E. (1958). "Some Observations on the Movement of Buildings on Expansive Soils in Vereeniging and Odendaalsrus," S. African Inst. of Civil Engineers, "Symposium on Expansive Clays," Johannesburg.

12. Dawson, R. F. (1953). "Movement of Small Houses Erected on Expansive Clay Soil," Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. I, p. 346.

13. Dawson, R. F. (1959). "Modern Practices Used in the Design of Foundations for Structures on Expansive Soils," Quarterly of the Colorado School of Mines, Vol. 54, No. 4, p. 67.

14. de Bruyn, C. M. A., Collins, L. E. and Williams, A. A. B. (1956). "The Specific Surface, Water Affinity and Potential Expansiveness of Clay," Clay Minerals Bulletin, Vol. 3, No. 17.

15. de Wet, J. A. (1957). "The Time-Heave Relationship for Expansive Clays," S. African Inst. of Civil Engineers, Symposium on Expansive Clays, p. 20.

16. Federal Housing Administration (1959. "Engineering Soil Classification for Residential Developments," FHA, No. 373, Washington, D. C.

17. Felt, E. J. (1953). "Influence of Vegetation on Soil Moisture Contents and Resulting Soil Volume Changes," Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 24.

18. Grim, R. E. (1953). Clay Mineralogy, McGraw-Hill Book Co., Inc., New York.

19. Highway Research Board (1958). "Water and Its Conduction in Soils," Special Report 40, Pub. 629.

20. Hilf, J. W. (1956). "An Investigation of Pore-Water Pressure in Compacted Cohesive Soils," Technical Memo 654, Bureau of Reclamation, Denver, Colorado, October.

21. Holtz, W. G. (1959). "Expansive Clays-Properties and Problems," Quarterly of the Colorado School of Mines, Vol. 54, No. 4, p. 89.

22. Holtz, W. G. and Gibbs, H. J. (1956). "Engineering Properties of Expansive Clays," Trans. A.S.C.E., Vol. 121, p. 641.

23. Jennings, J. E. (1950). "Foundations for Buildings in the Orange Free State Goldfields," Journal S. African Inst. Civil Engineers, Vol. 49, No. 4.

24. Jennings, J. E. (1953). "The Heaving of Buildings on Desiccated Clay," Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 390.

25. Jennings, J. E. (1955). "The Phenomenon of Heaving Foundations," S. African Inst. Civil Engineers, Vol. 5, No. 9, pp. 264-266.

26. Jennings, J. E. and Knight, K. (1957). "The Prediction of Total Heave from the Double Oedometer Test," S. African Inst. Civil Engineers, Symposium on Expansive Clays, p. 13.

27. Johns Hopkins University (1954). "Estimating Soil Tractionability from Climatic Data," Publication in Climatology, Vol. 7, No. 3.

28. Kantey, B. A. and Brink, A. B. A. (1952). "Laboratory Criteria for the Recognition of Expansive Soils," S. Africa National Building Research Institute, Bulletin No. 9, p. 25.

29. Komornick, A. (1957). Discussion of Collins (1957), S. African Inst. Civil Engineers, "Symposium on Expansive Clays," p. 52, Johannesburg.

30. Ladd, C. C. (1960). "Mechanisms of Swelling by Compacted Clay," H.R.B. Bulletin 245, Pub. 731, Washington, D.C.

31. Lambe, T. W. (1951). Soil Testing for Engineers, John Wiley and Sons, Inc., New York.

32. Lambe, T. W. (1953). "The Structure of Inorganic Soil," ASCE Separate No. 315.

33. Lambe, T. W. (1958). "The Structure of Compacted Clay" and "The Engineering Behavior of Compacted Clay," ASCE Journal of Soil Mechanics, Vol. 84, No. SM 2.

34. Lambe, T. W. (1960). "A Mechanistic Picture of Shear Strength in Clay," ASCE Research Conf. on Shear Strength of Cohesive Soils, Boulder, Colorado, June.

35. Lambe, T. W. and Whitman, R. V. (1959) "The Role of Effective Stress in the Behavior of Expansive Soils," Quarterly of the Colorado School of Mines, Vol. 54, No. 4, p. 33.

36. Lange, C. P. (1958). "Methods Used to Overcome Foundation Difficulties in Heaving Clays," S. African Inst. Civil Engineers, "Symposium on Expansive Clays," p. 33 Johannesburg.

37. Low, P. F. (1959). Discussion of Taylor (1959), ASCE Journal of Soil Mechanics, Vol. 85, No. SM 2.

38. Martin, R. T. (1959). "Water Vapor Sorption on Kaolinite: Entropy of Adsorption," 8th National Clay Conf., Norman, Oklahoma, October.

39. McDowell, C. (1956). "Interrelationship of Load, Volume Change and Layer Thicknesses of Soils to the Behavior of Engineering Structures," HRB Proc. Vol. 35, p. 754.

40. McDowell, C. (1959). "The Relation of Laboratory Testing to Design for Pavements and Structures on Expansive Soils," Quarterly of the Colorado School of Mines, Vol. 54, No. 4, p. 127.

41. Means, R. E. (1959). "Building on Expansive Clay," Quarterly of the Colorado School of Mines, Vol. 54, No. 4, p. 1.

42. Road Research Laboratory (1954). Soil Mechanics for Road Engineers, Her Majesty's Stationery Office, London.

43. Salas, J. A. J. and Serratosa, J. M. (1957). "Foundations on Swelling Clays," Proc. of 4th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 424.

44. Seed, H. B. and Chan, K. (1959). "Structure and Strength Characteristics of Compacted Clays," ASCE Journal of Soil Mechanics, Vol. 85, No. SM 5, October.

45. Seed, H. B., Lundgren, R. and Chan, K. (1954). "Effect of Compaction Method on Stability and Swell Pressure of Soils," HRB Bulletin 93, Publication 341, Washington, D.C.

46. Skempton, A. W. (1954). "A Foundation Failure Due to Clay Shrinkage Caused by Poplar Trees," Proc. Inst. Civil Engr., Vol. 3, Part 1, p. 66, London.

47. Taylor, A. W. (1959). "Ion Exchange Phenomena," ASCE Journal of Soil Mechanics, Vol. 85, No. SM-2.

48. Taylor, D. W. (1948). Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York.

49. Taylor, K. V. (1954). "Foundation Failures in Residences and Small Structures," ASCE Separate No. 561, December.

50. Templer, C. F. (1958). "Notes on the Design of Structures Founded on Heaving Soil," S. African Inst. Civil Engineers, "Symposium on Expansive Clays," p. 27, Johannesburg.

51. Terzaghi, K. (1950). Discussion of Jennings (1950). Journal of S. African Inst. Civil Engineers, Vol. 49, No. 4.

52. Terzaghi, K. and Peck, R. B. (1948). Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York.

53. Tschebotarioff, G. P. (1953). "A Case of Structural Damages Sustained by One-Story High Houses Founded on Swelling Clays," Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 473.

54. Ward, W. H. (1953). "Soil Movement and Weather," Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 477.

55. Williams, A. A. B. (1957). Discussion of Jennings and Knight, (1957), S. African Inst. Civil Engineers, Symposium on Expansive Clays, p. 57.

56. Wooltorton, D. (1936). "A Preliminary Investigation into the Subject of Foundations in the 'Black Cotton' and 'Kyatti Soils of the Mandalay District, Burma," Proc. 1st International Conference on Soil Mechanics and Foundation Engineering, Vol. III, pp. 242-256, Cambridge, Mass.

57. Wooltorton, F. L. D. (1950). "Movements in the Desiccated Alkaline Soils of Burma," Proc. A.S.C.E., Vol. 76, p. 63.

58. Wooltorton, F. L. D. (1954). The Scientific Basis for Road Design, Ed. Arnold, London.

59. Wooltorton, F. L. D. (1956). Discussion of McDowell (1956), H.R.B., Proc., Vol. 35, p. 770, Washington, D. C.

60. Youssef, M. S., Sabry, A. A. and Tewfik, M. M. (1957). "Substantial Consolidation and Swelling of Clay Cause Two Interesting Cases of Serious Damage to Hospital Buildings in Egypt," Proc. 4th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 462.

TABLE I

EXAMPLES OF PROBLEMS DUE TO SWELLING IN THE UNITED STATES

Reference	Location	Initial Soil Condition	Cause of Water Movement	Nature of Damage
Holtz and Gibbs (1956)	Malheur River Siphon, eastern Oregon	-	Seepage from nearby canal.	Pipeline rockers rose by up to 1 foot.
	Friant-Kern Canal, Central Valley, Calif.	Proctor Compaction	Seepage through canal lining.	Concrete lining on clay cracked badly.
Barber (1956)	Coastal Plain near Dist. of Columbia	-	-	Stair well of apartment building rose by 1 inch.
Allen and Johnson (1936)	Kansas	Compacted	-	Severe cracking of rigid pavements.
Dawson (1953, 1959)	Southwestern USA particularly Texas	Often Desiccated	Lawn watering, reduced evaporation, capillary rise, etc.	Cracking of residential houses causes tremendous damage.
Means (1959)	Oklahoma	Often Desiccated	Reduced evaporation, leakage of water mains, wet periods.	Rise of rigid pavements of 2 in., numerous examples of several inches of movement in building.
Terzaghi (1950)	San Antonio, Texas	Desiccated	Heavy rainfall.	Ground heave of up to 24 in.
Holtz (1959)	Colorado Springs	-	Watering lawns and roof runoff during storms	Severe cracking and uplifting of a motel.
	Boulder City, Nevada	-	Seepage of water through cracks in floor.	Severe movements of auto servicing garage floor slab.
McDowell (1956, 1959)	Texas	Usually Desiccated or Compacted	Reduced evaporation, etc.	Movement of roads and damage to buildings.

TABLE II

EXAMPLES OF PROBLEMS DUE TO SWELLING IN FOREIGN COUNTRIES

Reference	Location	Initial Soil Condition	Cause of Water Movement	Nature of Damage
Tschebotarioff (1953)	Eastern Cuba	-	Leakage from water system, among other factors.	Up to 4 in. rise of 1 storey house with severe cracking.
Baracos and Bozozuk (1957)	Winnipeg, Canada and Vernon, B.C.	Often Desiccated	Floods, interception by houses of rain runoff, etc.	Extensive damage to foundations.
Collins (1957)	Leeuhof, South Africa	Fairly Desiccated	Reduced evaporation and capillary rise.	Severe cracking of houses due to differential movements of 2 inches.
Youssif, et al, (1957)	Egypt	-	Leakage of water.	Severe cracking in hospital walls, etc., with heaving of floors of up to 20 inches.
Jennings (1950, 1955)	Union of South Africa	Often fairly Desiccated	Reduced evaporation and capillary rise, possibly flow due to differential temperatures.	Houses often rise 1-2 inches with resultant cracking.
Salas and Serratosa (1957)	Bornos, Spain	Usually Desiccated	-	Structural damage to buildings.
Wooltorton (1936)	Burma	-	Rainy periods.	Severe cracking in buildings with footing pressures less than 2000 lb./sq.ft.

TABLE III

EXAMPLES OF PROBLEMS DUE TO SHRINKAGE

Reference	Location	Cause of Shrinkage	Nature of Damage
Barber (1956)	Coastal Plain near Dist. of Columbia	Roots of trees contributed to shrinkage.	Differential movements along walls and fences.
Ward (1953)	Southeast England	Few dry weeks at beginning of summer and nearby trees.	Cracking of shallow building foundations.
Baracos and Bozozuk (1957)	Ottawa, Canada	Abnormally dry summer and presence of trees.	Differential vertical movements of 3 to 4 inches common under buildings.
Taylor (1954)	Kansas City	Two year drought, 1952-1953..	$20,000,000 to $40,000,000 damage due to cracking in houses and light structures.
Holtz (1959)	Denver, Colorado	Heating of soil from basement.	Settlement of interior footings.

TABLE IV

EXAMPLES OF PROBLEMS DUE TO CYCLIC MOVEMENT

Reference	Location	Cause of Cyclic Movement	Nature of Damage
Wooltorton (1956)	East Africa	(Presumably climatic variations).	Roadway moved sinusoidally with 3 in. amplitude.
Wooltorton (1950)	Mandalay District, Burma	Swelling during rainy periods. shrinkage during dry periods.	Severe cracks (up to 3 in.) in buildings.
Baracos and Bozozuk (1957)	Winnipeg and Ottawa, Canada	Normal Seasonal climatic changes.	Cyclic movements of 1 in. common. Damage to water mains noted. Seasonal water content changes to depth of 12 feet.
Komornick (1957)	Israel	Seasonal climatic changes.	Movements down to 20 feet.
Jennings (1950)	Orange Free State, South Africa	Seasonal climatic changes.	Ground movement in open fields of up to 2 inches.
Dawson (1953)	Austin, Texas	-	Cyclic variations of 0.5 in. noted for two houses

TABLE V

SOILS TESTED

Name	Supplied By	Location	Classification[1]	Remarks
50-50 Kaolinite-Bentonite Mixture	M.I.T.	Georgia Kaolin and Wyoming Bentonite	CH	Tests run but all not reported.
Iredell Clay	F.H.A.	Fairfax County, Va. (15-25" depth)	CH	Unsuitable for road construction or foundation[2]
Houston Black Clay	F.H.A.	6 Miles E. of L.W. Stasney Farm, Temple, Texas (0-24" depth)	CH	Known to cause considerable trouble[2]
Enon Silt Loam	F.H.A.	Fairfax County, Virginia	CH	
Vicksburg Buckshot Clay	M.I.T.	Vicksburg, Miss. (surface soil)	CH	
Texas Black Clay	F.H.A.	Temple, Texas	CH	Result of mixing Houston Black and Wilson clays.
Siburua Shale	M.I.T.	Siburua Dam, Siburua, Venezuela	CH	
Keyport Soil	F.H.A.	Norfolk County, Va. (15-22" depth)	CL	No known trouble[2]
Boston Blue Clay	M.I.T.	Cambridge, Mass.	CL	
Vicksburg Loess	M.I.T.	Vicksburg, Miss.	CL-ML	
Guelph Fine Sandy Loam	F.H.A.	NE 1/4 of NE 1/4 of Sec. 2, Washington Township, Sanilac County, Michigan	CL-ML	
Maury Soil	F.H.A.	Kentucky	CH	One lb. sample. Only classification tests run.
Carlton Soil	F.H.A.	Salem, Oregon	OH	One lb. sample. Only classification tests run.

1. Unified soil classification.

2. Information from Mr. Henry of the F.H.A.

1

TABLE VI

RESULTS OF CLASSIFICA

Soil	Atterberg Limits[1] (%) Liquid	Plastic	Shrinkage	Plasticity Index	M.I.T. Grain Size Classification (%) Sand	Silt	Clay	Sp Gr
50-50 Kaolinite-Bentonite Mixture	295	33	23 ?	262	0	30	70	2
Iredell Clay	81	34	14.5	47	15	50	35	2
Houston Black Clay	71	27	18	44	0	30	70	2.
Enon Silt Loam	69	27	17	42	25	30	45	2.
Vicksburg Buckshot Clay	65 (60-70)	27 (2[illegible]-29)	16 (14-18	38 (36-40)	5	60	35	2.
Texas Black Clay	58	24	-	34	5	45	50	2.
Siburua Shale	62 (60-63)	32 (30-33)	15	30	0	20	80	2.
Keyport Soil	44	23	17	21	20	50	30	2.
Boston Blue Clay	35	22 (21-21)	18.5 (17-20)	13 (11-14)	0	45	55	2.
Vicksburg Loess	33.5	23.5	21.5	10	5	80	15	2.
Guelph Fine Sandy Loam	21	14	13 ?	7	40	50	10	2.7
Maury Soil	52	28	17	24	-	-	-	-
Carlton Soil	55	36 ?	22 ?	19 ?	-	-	-	-

1. On minus No. 40 sieve fraction.
2. Activity = $\frac{\text{Plasticity Index}}{\text{\% Clay Size}}$
3. Field Moisture Equivalent
4. Per Cent volume change (based on to Shrinkage Limit (w_s).

Notes: Range of values reported in (

TABLE VI

...ULTS OF CLASSIFICATION TESTS

M.I.T. Grain Size Classification (%)			Specific Gravity	Activity[2]	F.M.E.[1,2] (%)	Free[1] Swell (%)	Water Content (%) at Per Cent R.H. of				$\frac{\Delta V}{V}$ (%)[4] F.M.E. to w_s
Sand	Silt	Clay					100	70	50	30	
0	30	70	2.75	3.8	~90	200	(12-32)	-	-	-	53 ?
15	50	35	2.87	1.3	47	95	~17 (12-18)	11	9.5	8.5	40
0	30	70	2.71	0.65	47	-	19	10.5	8	6.5	35
25	30	45	2.85	0.95	46	-	~17	6.5	5	4	36
5	60	35	2.70	1.1	48	75 (50-100)	~14 (11-16)	8.5	7	6.5	38
5	45	50	2.74	0.7	38	75	14	8	7	6.5	30 Estimate
0	20	80	2.85	0.4	34	135 (110-160)	~16	-	-	-	28
20	50	30	2.72	0.7	33	43 (39-45)	9 (8.7-9.5)	4	3	3	23
0	45	55	2.78	0.25	29	15 (10-25)	~5 (3.5-9.5)	-	1.5	-	16
5	80	15	2.74	0.65	29	-	~7	-	2.5	-	11.5
40	50	10	2.72	0.7	16	35 (30-40)	5 (4.5-6)	-	-	-	6
-	-	-	-	-	32	~55	~14	-	-	-	~22
-	-	-	-	-	~40	~45	~13	-	-	-	~25

...volume change (based on initial volume) if dry saturated sample from F.M.E. ...age Limit (w_s).

...of values reported in ( ). ? Denotes Questionable values.

2

TABLE VII

RESULTS OF HEAVE, SWELL PRESSURE

Soil	Final Heave, H_f (%)			Final Swell Pressure, SP_f (lb./sq.ft.)			Ratio: 2 Hour to Final Value				
							Heave			Swell Press	
	Dry	Moist	Wet	Dry	Moist	Wet	Dry	Moist	Wet	Moist	W
Iredell Clay	25	22.5	7	~10,500	11,600	2400	0.75	0.55	0.45	0.85	0.
Houston Black Clay	~20	15	5.5	~8,000	-	-	0.55	0.54	0.54	-	-
Enon Silt Loam	~16	13	4	-	-	1400	0.95	0.85	0.75	-	0.
Vicksburg Buckshot Clay	24	13	6.5	~8,000	7,000	2800	-	0.90	0.70	1.00	0.
Texas Black Clay	-	~17	4	-	5,400	1900	-	0.70	0.45	0.85	0.
Siburua Shale	-	17	4	-	10,000?	650	-	0.55	0.50	1.00	0.
Keyport Soil	7	8	~1	-	7,600	~400	0.95	0.90	0.90	1.00	1.
Boston Blue Clay	-	4.5	0.1	-	1,800	~200	-	0.95	0.6	1.00	-
Vicksburg Loess	~0	2.5	-1	~200	~900	< 200	1.0	1.0	1.0	1.0	1.
Guelph Fine Sandy Loam	-	0.5	-0.4	-	~500	< 200	-	1.00	-	1.00	1.

TABLE VII

SWELL PRESSURE, AND SWELL INDEX TESTS

Hour to Final Value for:			Swell Index, S.I. and Heave, H						Molding Conditions					
	Swell Pressure		Dry		Moist		Wet		Dry		Moist		Wet	
Wet	Moist	Wet	S.I. (psf)	H (%)	S.I. (psf)	H (%)	S.I. (psf)	H (%)	w_m (%)	γ_d (pcf)	w_m (%)	γ_d (pcf)	w_m (%)	γ_d (pcf)
0.45	0.85	0.85	7500	-	4400	0.8	1700	0.2	8.5	100	17	99	34	88
0.54	-	-	7000	-	5200	1.2	2200	0.4	9	99	17	99	28	91
0.75	-	0.95	5000	-	5800	1.2	1300	0.1	5.5	95	16	100	28	95
0.70	1.00	0.90	7000	1.3	5500	1.7	2400	1.1	7	98	14	97	27	93
0.45	0.85	0.5 ?	-	-	4800	0.8	900	0.1	-	-	14	102	24	98
0.50	1.00	0.95	-	-	3500	0.7	550	< 0.1	-	-	16	106	32	90
0.90	1.00	1.00	2200	0.4	3300	0.5	300	< 0.1	3.2	97	10	103	23	101
0.6	1.00	-	1200	-	1700	-	~200	~0	2	94	5.5	96	23	104
1.0	1.0	1.0	~200	0	~900	0.1	< 200	< 0	2.5	91	7	98	24	99
-	1.00	1.00	-	-	450	< 0.1	< 200	< 0	-	-	4.5	115	14	122

2

TABLE VIII

POTENTIAL VOLUME CHANGE CLASSIFICATION OF SOILS TESTED

(Based on correlations published in the literature)

PVC Classification				
Rating:	< 2	2 - 4	4 - 6	> 6
Category:	Noncritical	Marginal	Critical	Very Critical

Soil	RATING BASED ON (1) Average Heave* from Dry and Moist Water Content	(2) P.I.	(3) Water Content at 85% R.H.	(4) $\Delta V/V$ FME to w_s	(5) P.I. and Activity	(6) Average Rating**
Iredell Clay	7.8	8.1	5.0	7.6	7.5	7.3
Houston Black Clay	5.8	7.6	5.0	6.5	8.3	6.5
Vicksburg Buckshot Clay	6.1	6.5	4.0	7.2	7.0	6.2
Enon Silt Loam	5.1	7.2	3.8	6.7	8.0	6.0
Texas Black Clay	5.7	5.7	3.9	5.4	6.5	5.5
Siburua Shale	5.7	4.9	4.7	5.0	~ 2	4.7
Keyport Soil	2.9	3.1	3.2	3.8	3.5	3.2
Boston Blue Clay	2.0	1.5	1.8	2.2	~1.5	1.8
Vicksburg Loess	1.4	1.0	2.0	1.3	1.5	1.4
Guelph Fine Sandy Loam	0.8	0.4	2.0	0	1	0.8

* This value counted double.

** Sum of 2 x (1) + (2) through (5) divided by 6.

TABLE IX

RESULT OF POTENTIAL VOLUME CHANGE CLASSIFICATION OF SOILS TESTED BY SWELL INDEX VALUES

Soil	Rating for Test at Relative Water Content of: Dry	Moist	Wet	Average Rating	Deviation of Average from Column 6, Table VIII	No. of Tests in Correct Category
Iredell Clay	8.6	5.6	6.4	6.9	-0.4	2 of 3
Houston Black Clay	8.3	6.6	7.0	7.3	+0.8	3 of 3
Vicksburg Buckshot Clay	8.3	7.0	7.3	7.5	+1.3	3 of 3
Enon Silt Loam	6.4	7.3	5.8	6.5	+0.5	1 of 3
Texas Black Clay	-	6.1	5.0	5.6	+0.1	1 of 2
Siburua Shale	-	4.4	4.1	4.3	-0.4	2 of 2
Keyport Soil	2.6	4.1	3.0	3.2	0.0	2 of 3
Boston Blue Clay	1.3	2.0	2.0	1.8	0.0	3 of 3
Vicksburg Loess	0	0.9	1.5	0.8	-0.6	3 of 3
Guelph Fine Sandy Loam	-	0.4	~1	0.7	-0.1	2 of 2
					Total -	22 of 27

TABLE X

SUGGESTED METHODS FOR ESTIMATING THE RELATIVE WATER CONTENT OF A CLAY

(Tentative - subject to revision)

Relative Water Content: Designation	Relative Water Content: Equivalent Water Content	Properties (only exhibited by predominantly clayey soils)
Dry*	w_{50} (at 50% R.H.)	1) Soil looks "bone dry" and has the consistency of portland cement. 2) Can not mold soil with hand pressure into a coherent ball. 3) Soil is dusty if blown. If shake a handfull of soil in a quart jar, can see noticeable amounts of dust when stop shaking.
Moist**	w_{100} (at 100% R.H.)	1) Soil looks dry. 2) If shake a handfull of soil in a quart jar, can not see very much dust when stop shaking. 3) Can mold soil with considerable hand pressure into balls the size of small stones, but these will break if dropped several times on a hard surface from a height of 1 foot.
Intermediate***	$0.2(w_\ell) + 9\%$ (w_ℓ = Liquid Limit)	1) Soil looks as if it had some moisture. 2) Is not dusty. 3) Can mold soil with firm hand pressure into balls the size of small stones which will not break if dropped several times on a hard surface from a height of 1 foot. 4) Cannot roll into a 1/8" threat. 5) Drop of water will disappear in matter of a few seconds from the surface.
Wet	w_p (w_p = Plastic Limit)	1) Soil looks wet. 2) Can roll soil into a 1/8" threat, at which point the thread will just begin to crumble. 3) Can easily mold soil with hand pressure into large balls which will not break if dropped. 4) Drop of water will disappear in less than 30 seconds from the surface.
FME	Field Moisture Equivalent	1) Soil feels sticky. 2) Drop of water will disappear in just 30 seconds after being dropped on a smooth surface of the soil.

* This is equivalent to air drying a soil (assuming the relative humidity of the air is from 30 to 70%).

** Can be obtained by placing air-dried soil in closed container with water at the bottom and allowing to equilibrate for several days.

*** Suggested by McDowell (1956) as the minimum water content from which swelling soils start to swell when used as subgrade soil of a pavement.

TABLE XI

SWELL INDEX TEST - DATA SHEET
(Tentative)

PROJECT ____________ TEST NO. ____________

LOCATION ____________ DATE ____________

SAMPLE NO. ____________ NAME OF SOIL ____________ TESTED BY ____________

SAMPLE LOCATION ____________ DEPTH ____________

THICKNESS OF STRATUM ____________ CONDITION OF STRUCTURES AT LOCATION ____________

A. SOIL DESCRIPTION (from test results or estimated)

1. Grain Size ____________ 2. Dry Strength ____________
3. Feel When Wet ____________ 4. Dilatency ____________
5. Color ____________ 6. Atterberg Limits ____________
7. Unified Soil Classification* ____________

B. RELATIVE WATER CONTENT** (Dry, Moist, Intermediate, Plastic Limit, F.M.E.)

	Description**	Relative Water Content
1. Natural Soil:		______
2. As Received:		______
3. At Testing:		______

4. Adjustments*** to soil before testing ____________

C. SWELL INDEX TEST

THICKNESS OF STRATUM ________ CONDITION OF STRUCTURES AT LOCATION ________

A. SOIL DESCRIPTION (from test results or estimated)

1. Grain Size ________ 2. Dry Strength ________

3. Feel When Wet ________ 4. Dilatency ________

5. Color ________ 6. Atterberg Limits ________

7. Unified Soil Classification* ________

B. RELATIVE WATER CONTENT** (Dry, Moist, Intermediate, Plastic Limit, F.M.E.)

	Description**	Relative Water Content
1. Natural Soil:		________
2. As Received:		________
3. At Testing:		________

4. Adjustments*** to soil before testing ________

C. SWELL INDEX TEST

1. Compaction Procedure: No. Layers ________ No. Blows per Layer ________

2. Test Data

Elapsed Time (min.)	
Dial Reading (.0001 in.)	

Elapsed Time (min.)	
Dial Reading (.0001 in.)	

3. Dial Reading at 2 hours ________ 4. Swell Index (lb./sq.ft.) ________

5. PVC Rating ________ 6. PVC Category ________

7. Other ________

8. Remarks ________

* See FHA "Engineering Soil Classification for Residential Developments" (FHA, 1959).

** See Table X.

*** Air drying; water added; equilibration time; lumps broken, etc.

TABLE XII

MATERIALS LIST FOR SWELL INDEX D

Part No.	Name	Material and Comments
19	Washer	D-alum[1] or Brass[2]
18	Nut	Std. 5/8-11, Hex, Steel
17	Top Bar	D-alum[1] or Brass[2]
13	Proving Ring[3]	Tool Steel, Must Deflect .0001" per 5 to 6 lb. force
-	Proving Ring Dial[3]	.0001" per Division, .2 to .4" travel
10, 11, 12, 14	Assembly Parts[3]	D-alum[1] or Brass[2]
15, 16	For Proving Ring	
9	Adjustable Nut	D-alum[1] or Brass[2]
8	Adjustable Rod	D-alum[1] or Brass[2]
-	Top and Bottom Porous Stones	Refractory Porous Stones, P-260 Dia. = 2.740 - 2.745, Thickness = .410 -
4	Cover	D-alum[1] or Brass[2]
5	Bolts	Std. 5/16-18, Cap Screw, Brass[1] or St
3	Compaction Ring	D-alum[1], Brass[2], or Machinable Stain
2	Spacer Ring	D-alum[1], Brass[2], or Machinable Stain
1	Base	D-alum[1] or Brass[2]
20	O-Ring	4.5"I.D., 5.0"O.D. Parker No. 5427-
6	Lucite Container	Lucite
7	Rods	Machinable Stainless Steel, No. 304, o
H-1, H-2	Compaction Hammer[4]	Brass[2]
H-3, H-4	Sleeve for Compaction Hammer[4]	D-alum[1] or Brass[2]

1. Duraluminum 7075-T6 (755-T6). 2. Free Turning Brass. 3. Proving Ring ar purchased from a commercial firm, such as Soiltest, Inc., if Proving Ring meets
4. Compaction Hammer to weigh 5.5 lb. Sleeve to allow 12" drop of Hammer. Ham from a commercial firm, such as Soiltest, Inc. (This is a standard item of soil t

TABLE XII

ALS LIST FOR SWELL INDEX DEVICE

Material and Comments	Supplier
m[1] or Brass[2]	
5/8-11, Hex, Steel	
m[1] or Brass[2]	Whitehead Metals
Steel, Must Deflect " per 5 to 6 lb. force	
" per Division, .2 to .4" travel	B. C. Ames Company - Catalogue No. 212.2, .3, or .4
m[1] or Brass[2]	
m[1] or Brass[2]	
m[1] or Brass[2]	
ctory Porous Stones, P-260 2.740 - 2.745, Thickness = .410 - .420"	Norton Products
m[1] or Brass[2]	
/16-18, Cap Screw, Brass[1] or Stainless Steel	
m[1], Brass[2], or Machinable Stainless Steel, No. 304	
m[1], Brass[2], or Machinable Stainless Steel, No. 304	
m[1] or Brass[2]	
.D., 5.0"O.D. Parker No. 5427-88	Irving B. Moore Corporation
	Forest Products, Inc.
nable Stainless Steel, No. 304, or Brass[2] 2	Whitehead Metal Products, Inc.
n[1] or Brass[2]	

ing Brass. 3. Proving Ring and Dial and assembly parts may be
est, Inc., if Proving Ring meets stated deflection requirements.
llow 12" drop of Hammer. Hammer and Sleeve may be purchased
This is a standard item of soil testing equipment.)

2

TABLE XIII

LIST OF SUPPLIERS

1. Whitehead Metal Products, Inc.
   287-303 West 10th Street
   New York 14, New York

2. B. C. Ames Company
   131 Lexington Street
   Waltham, Massachusetts

3. Norton Products
   Worcester 6, Massachusetts

4. Soiltest, Inc.
   4711 West North Avenue
   Chicago 39, Illinois

5. Irving B. Moore Corporation
   65 High Street
   Boston, Massachusetts

6. Forest Products, Inc.
   131 Portland Street
   Cambridge, Massachusetts

STRESSES BETWEEN CLAY PARTICLES

$$\bar{\sigma} = \sigma - u = R - A \qquad \text{Equation 1.}$$

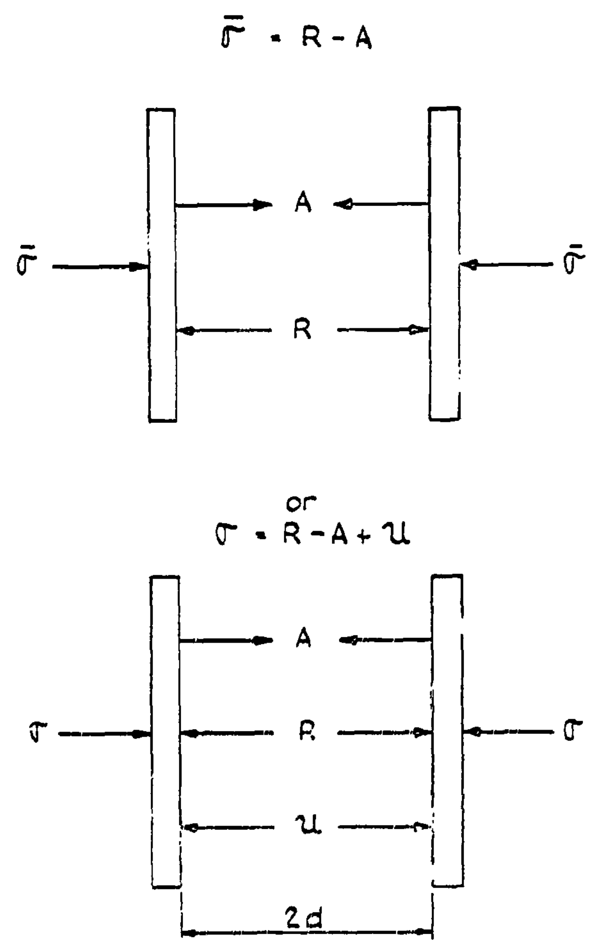


Fig. 1

SHRINKAGE BEHAVIOR OF CLAYS

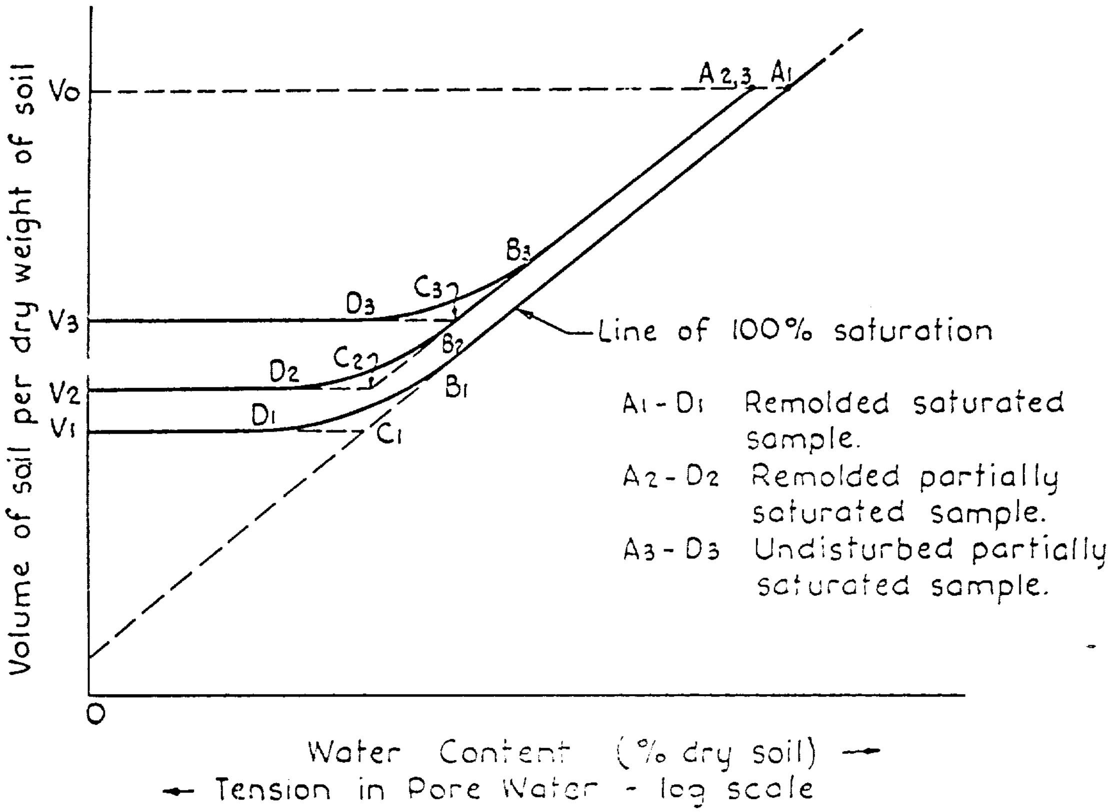


Fig. 2

EFFECT OF DENSITY AND WATER CONTENT ON SWELLING

(Holtz and Gibbs, 1956)

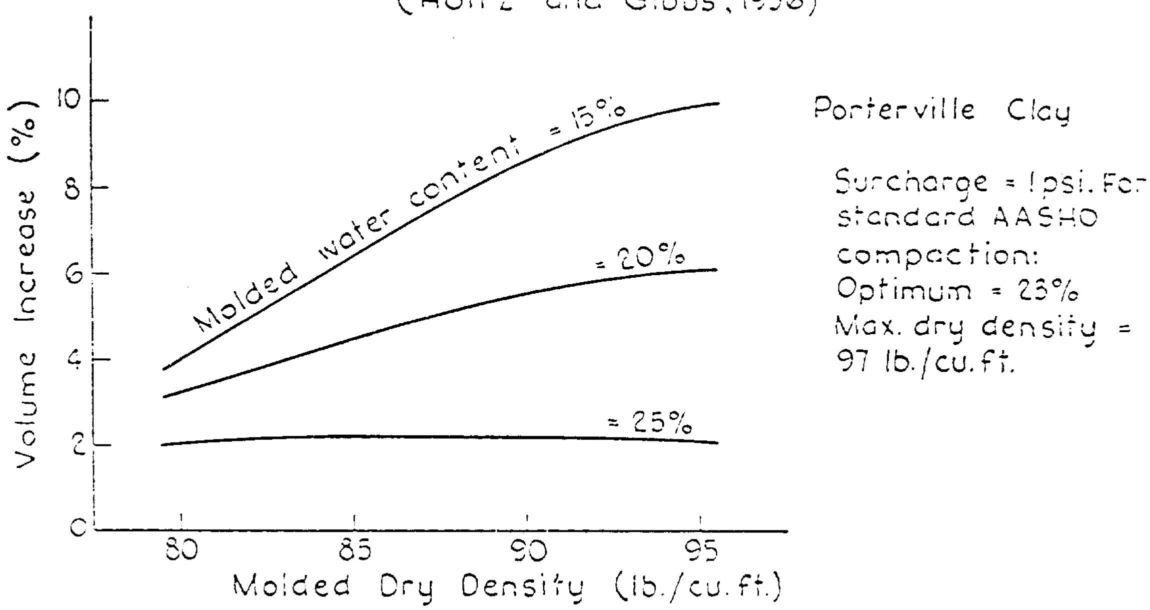


Fig. 3

Preceding Page Blank

CONFINING PRESSURE VS. SWELLING

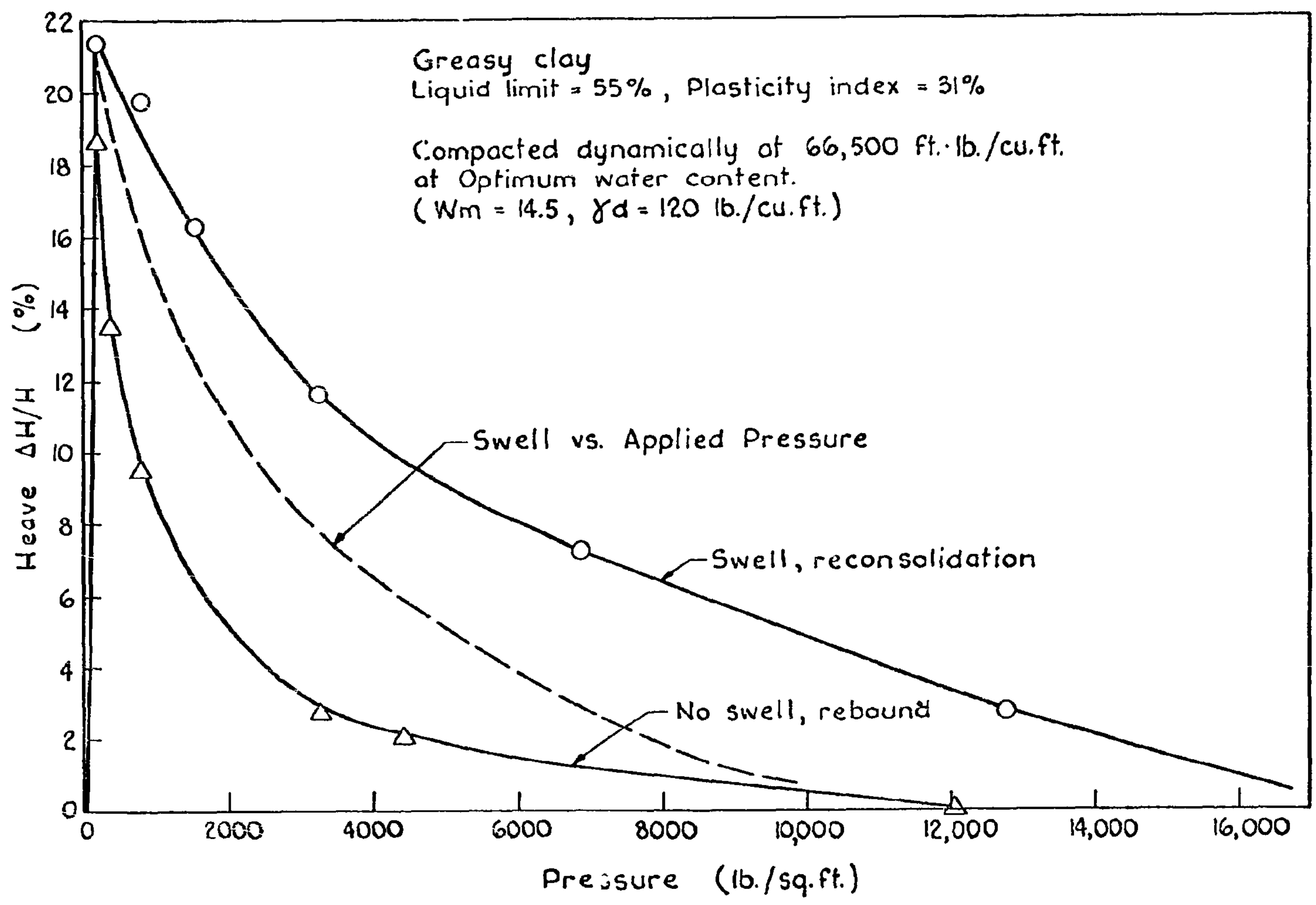


Fig. 4

ONE DIMENSIONAL SWELLING

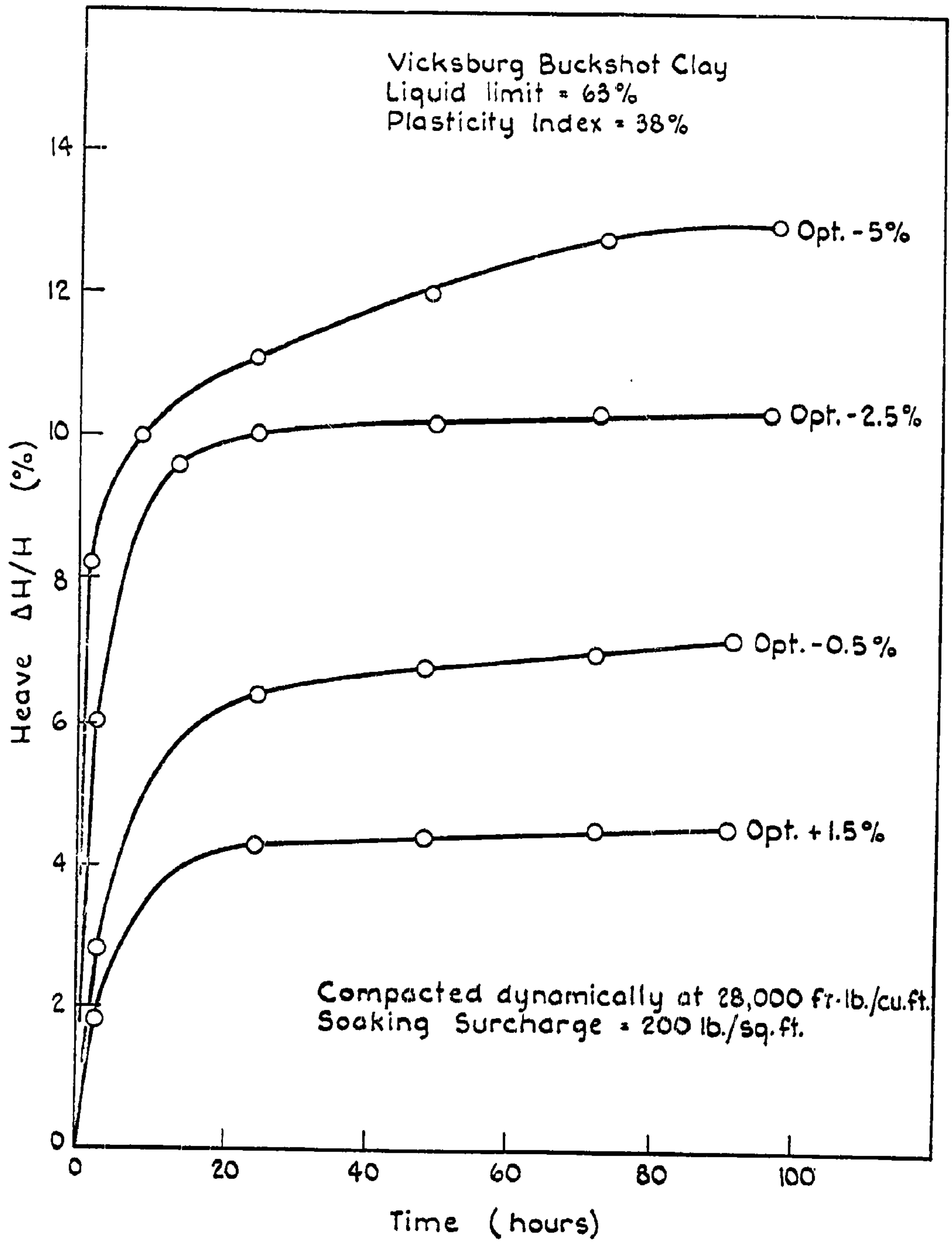


Fig. 5

TYPICAL SET-UP IN CONSOLIDOMETER UNITS

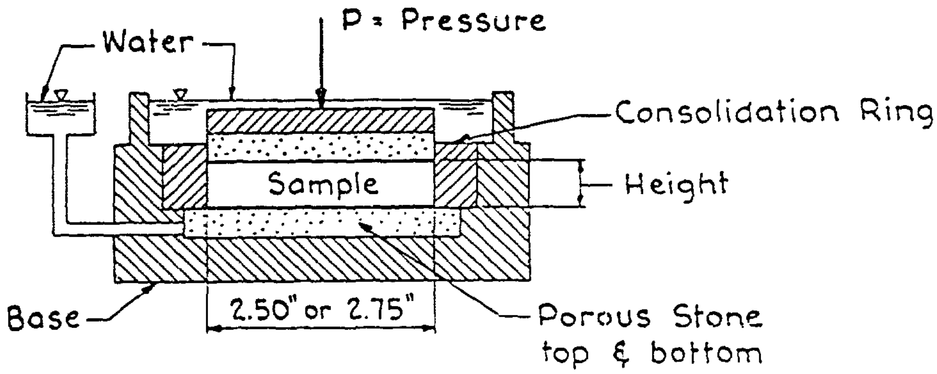


Fig. 6

APPARATUS FOR SWELL PRESSURE AND SWELL INDEX TESTS

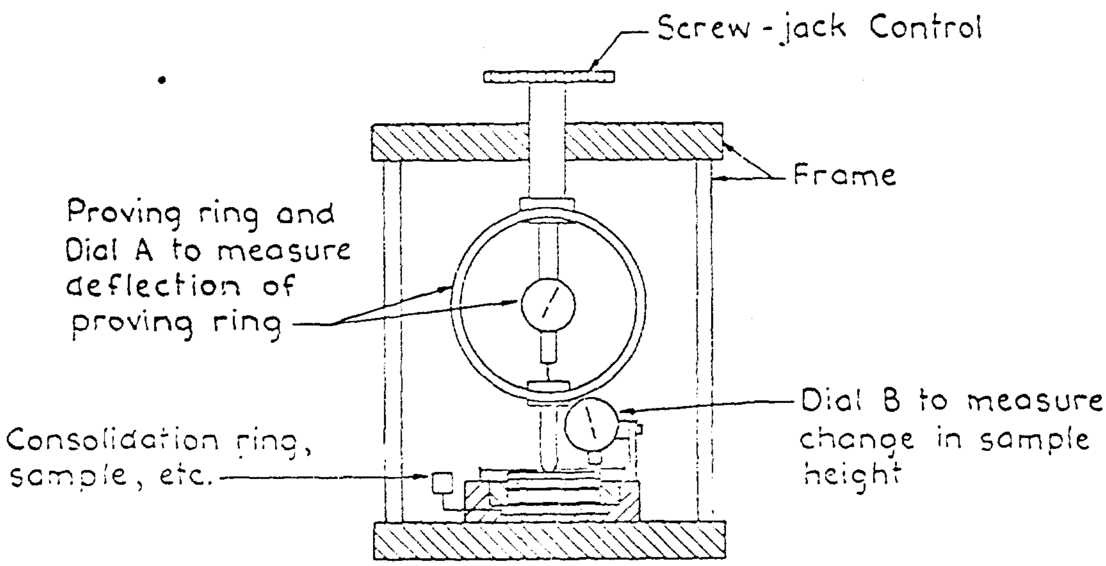
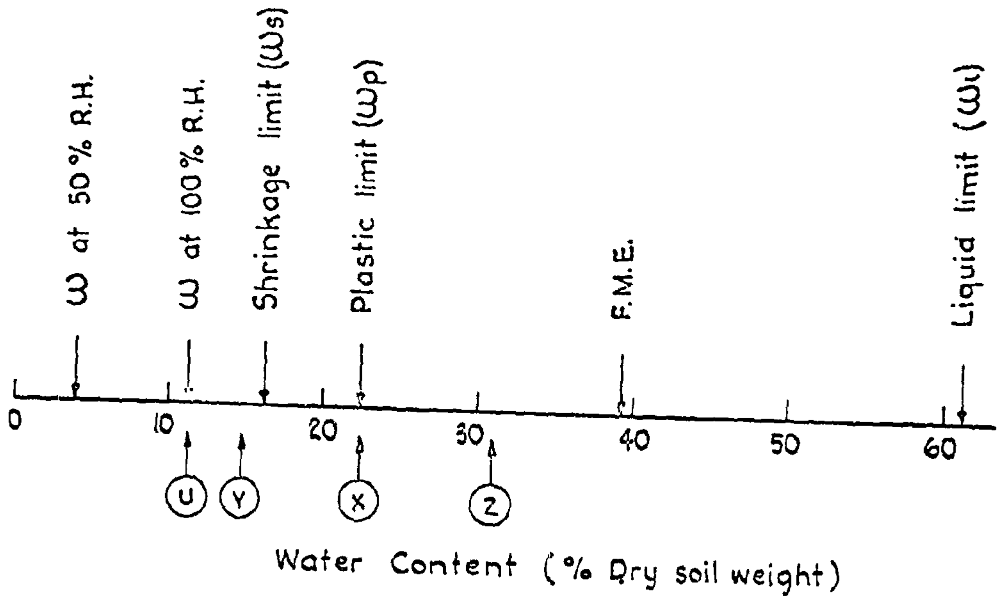


Fig. 7

Preceding Page Blank

RELATIVE WATER CONTENT



Sample	Water Content (%)	Relative Water Content
U	11	= ω at 100% R.H. (ω_{100})
X	22	= Plastic limit (ω_p)
Y	15	= $\frac{1}{3}$ of way from ω_{100} to ω_p
Z	31	= $\frac{1}{2}$ of way from ω_p to F.M.E.

Fig. 8

PLASTICITY CHART

Soils Tested

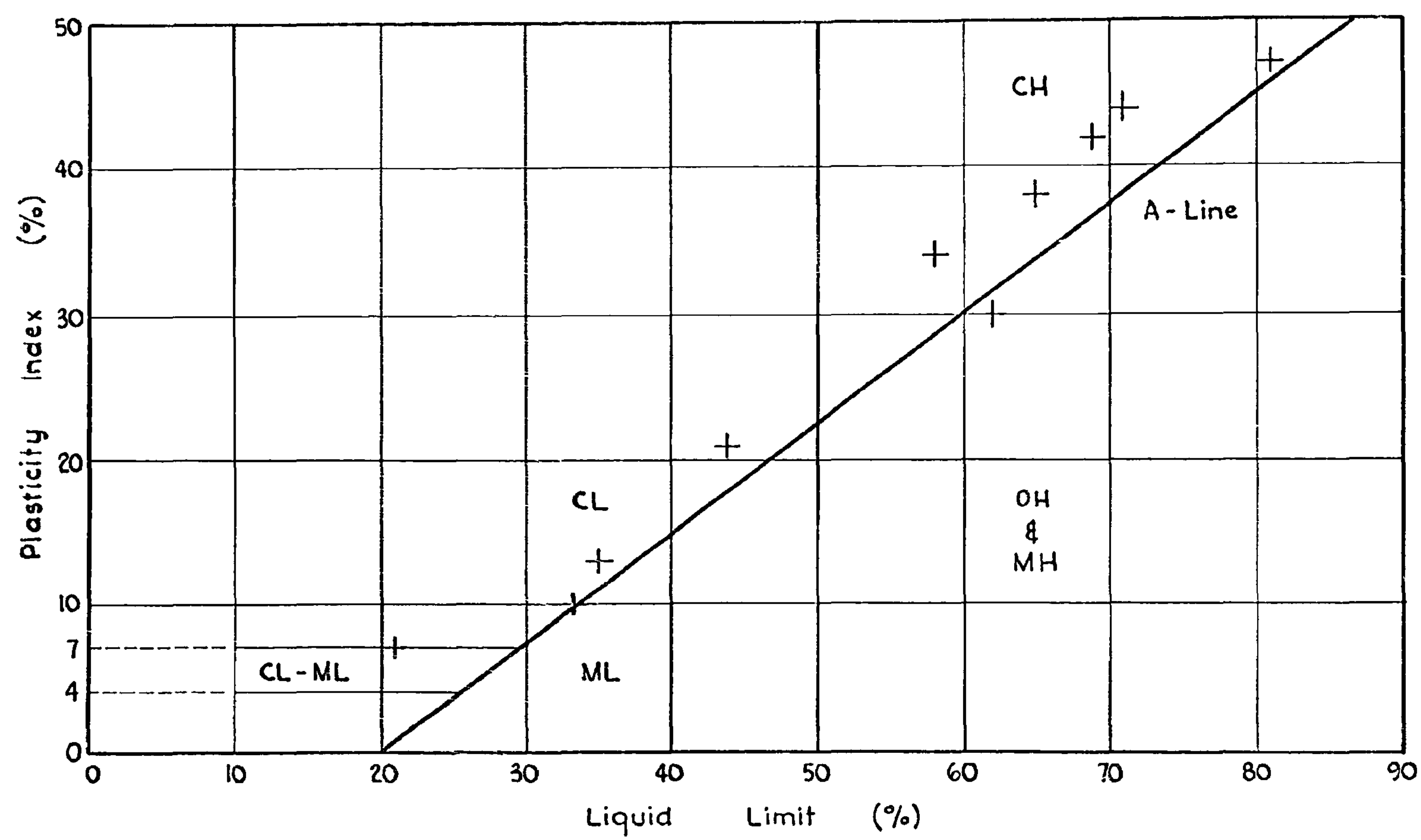


Fig. 9

PLASTICITY INDEX VS. VOLUME CHANGE - DRYING FROM F.M.E. TO SHRINKAGE LIMIT AND FINAL HEAVE

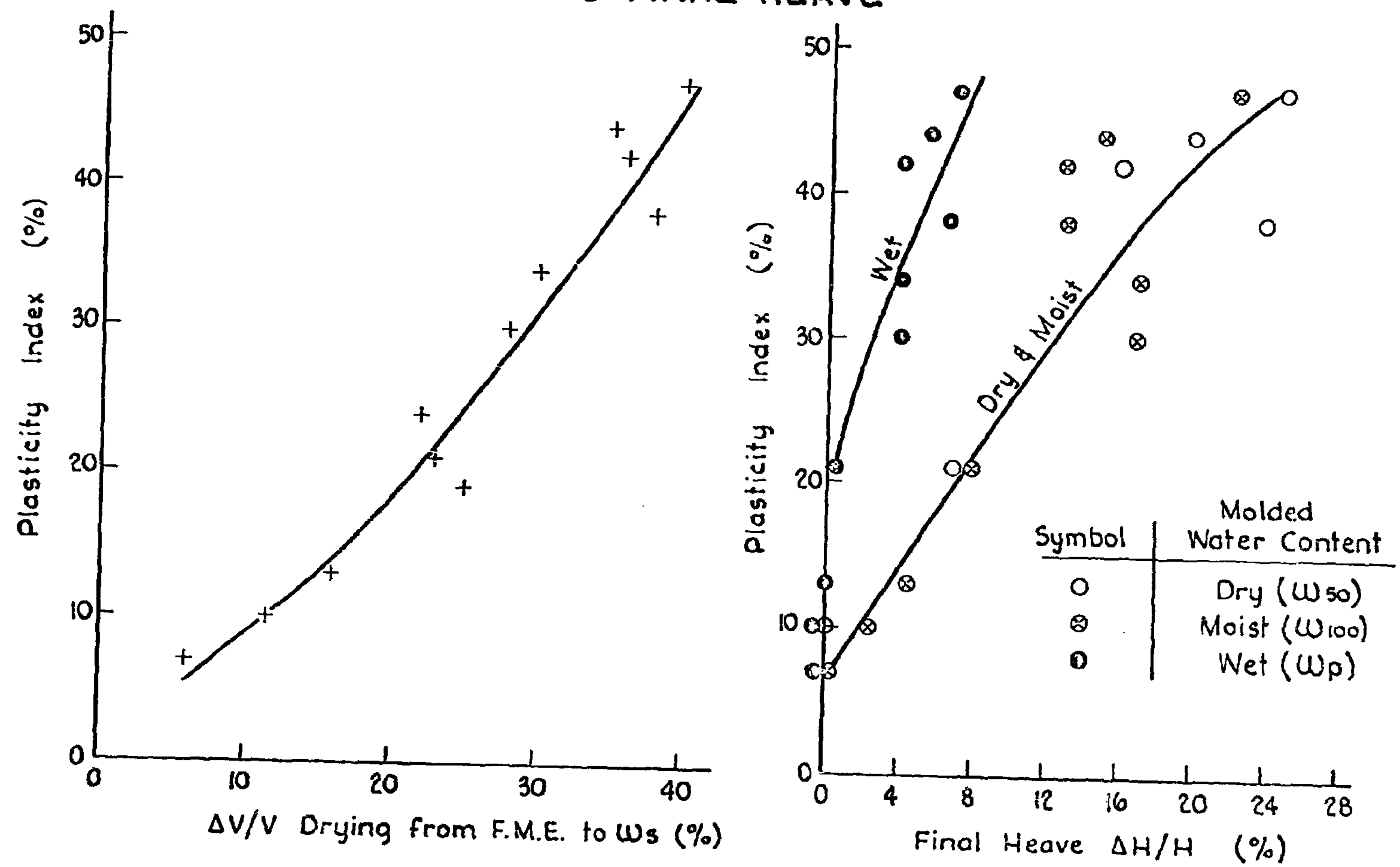


Fig. 10

FINAL HEAVE
VS.
VOLUME CHANGE - DRYING FROM F.M.E. TO
SHRINKAGE LIMIT

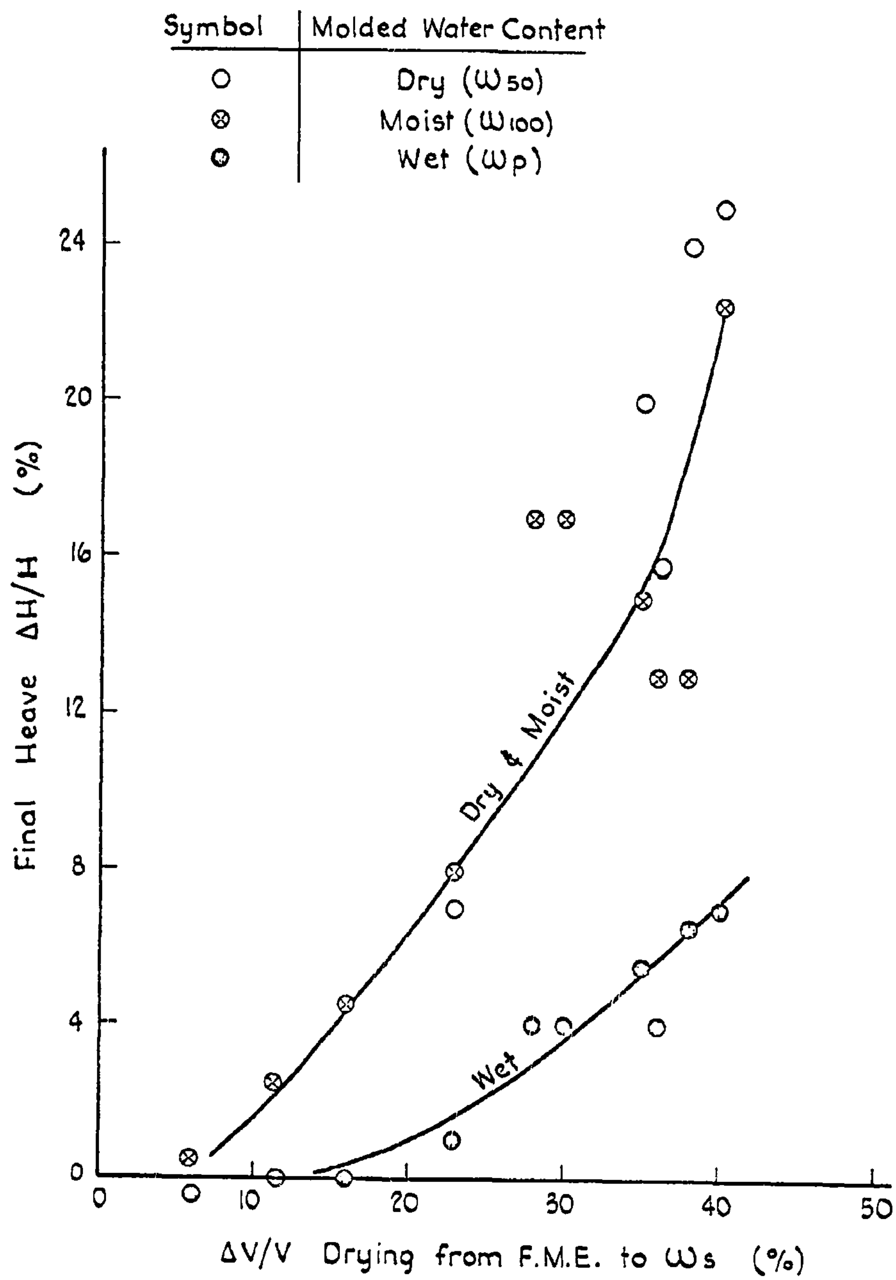


Fig. 11

FINAL HEAVE VS. WATER CONTENT AT 100% R.H. AND FREE SWELL

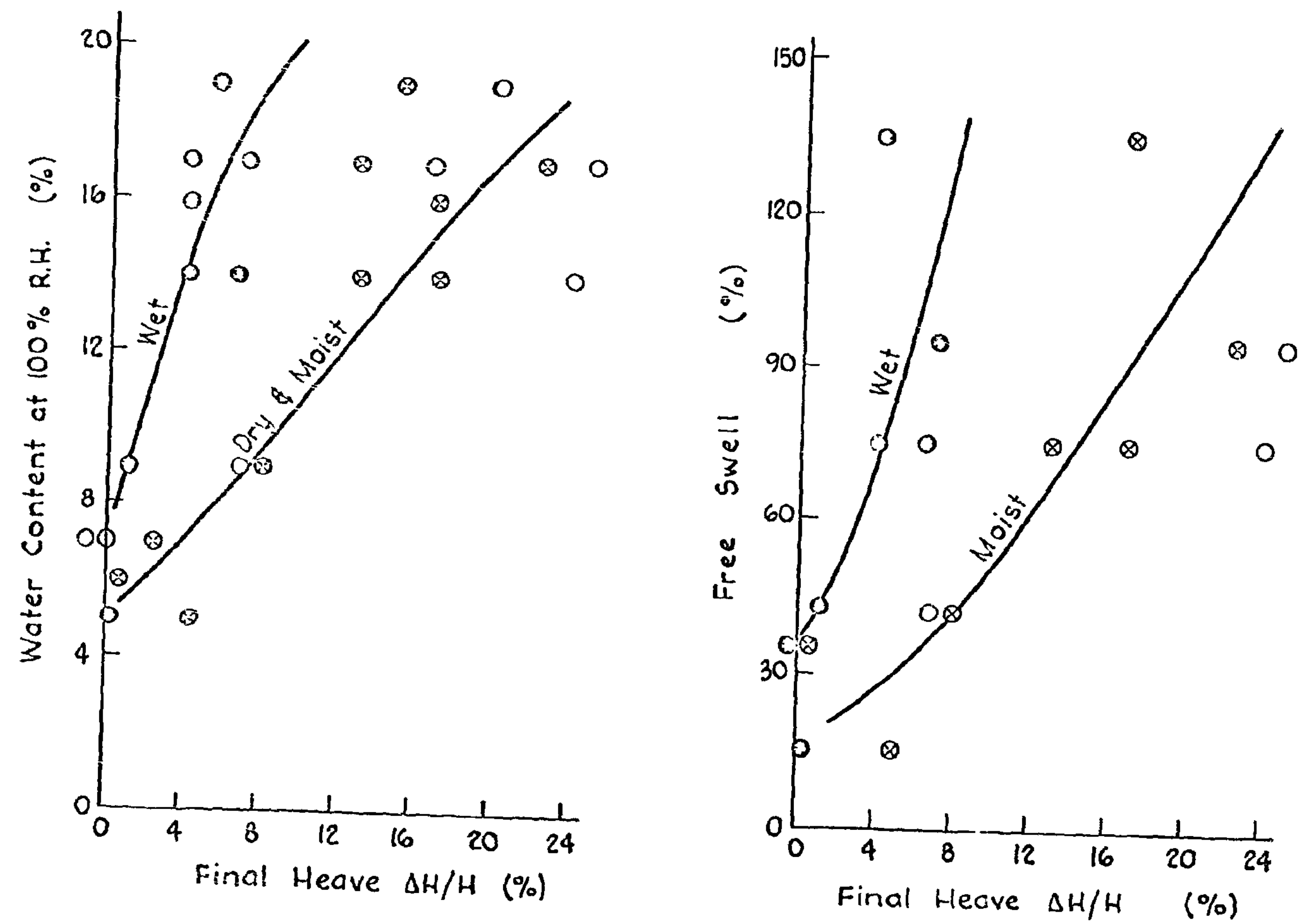


Fig. 12

FINAL HEAVE VS. FINAL SWELL PRESSURE

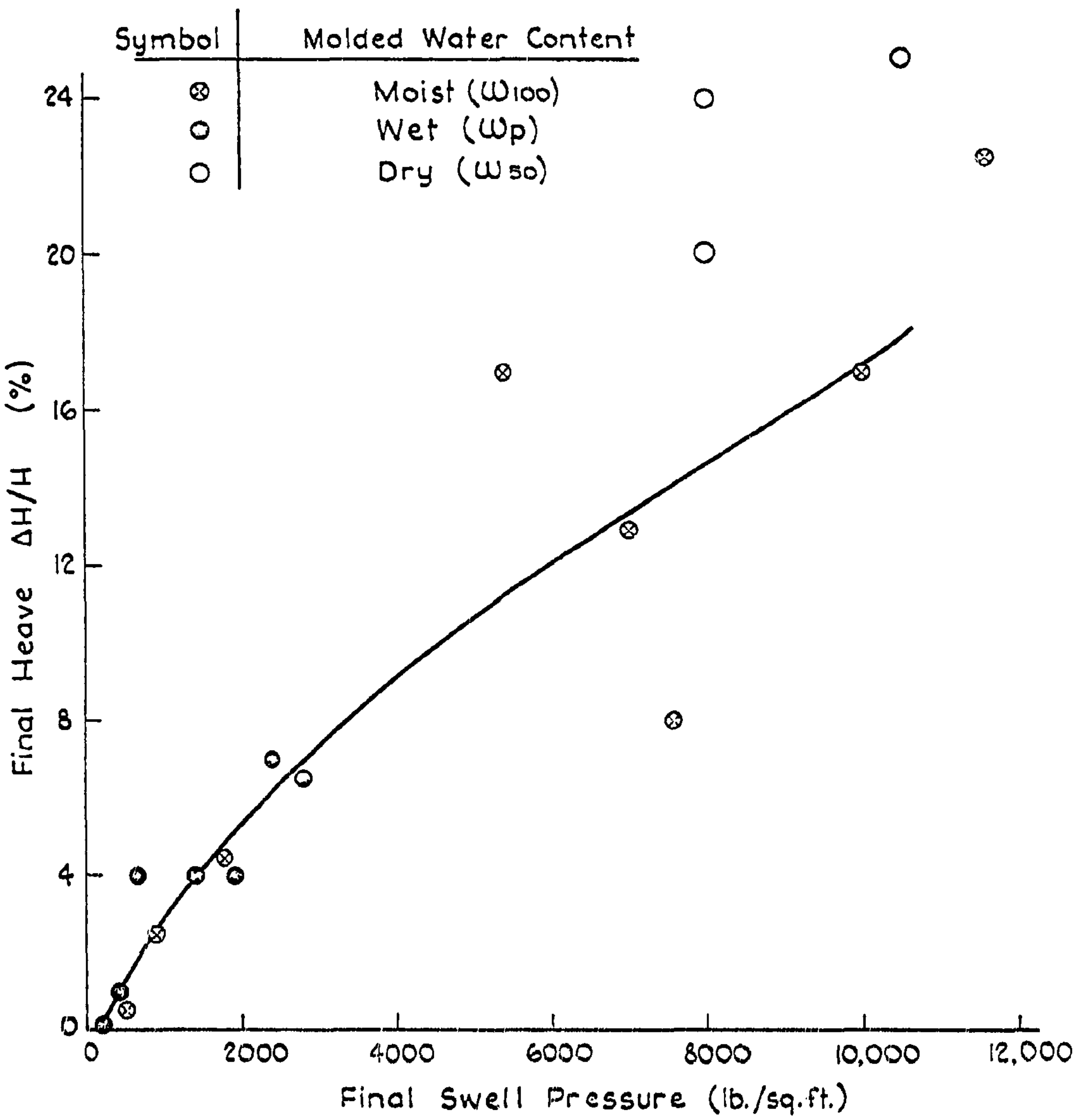


Fig 13

SWELL INDEX VS. PLASTICITY INDEX

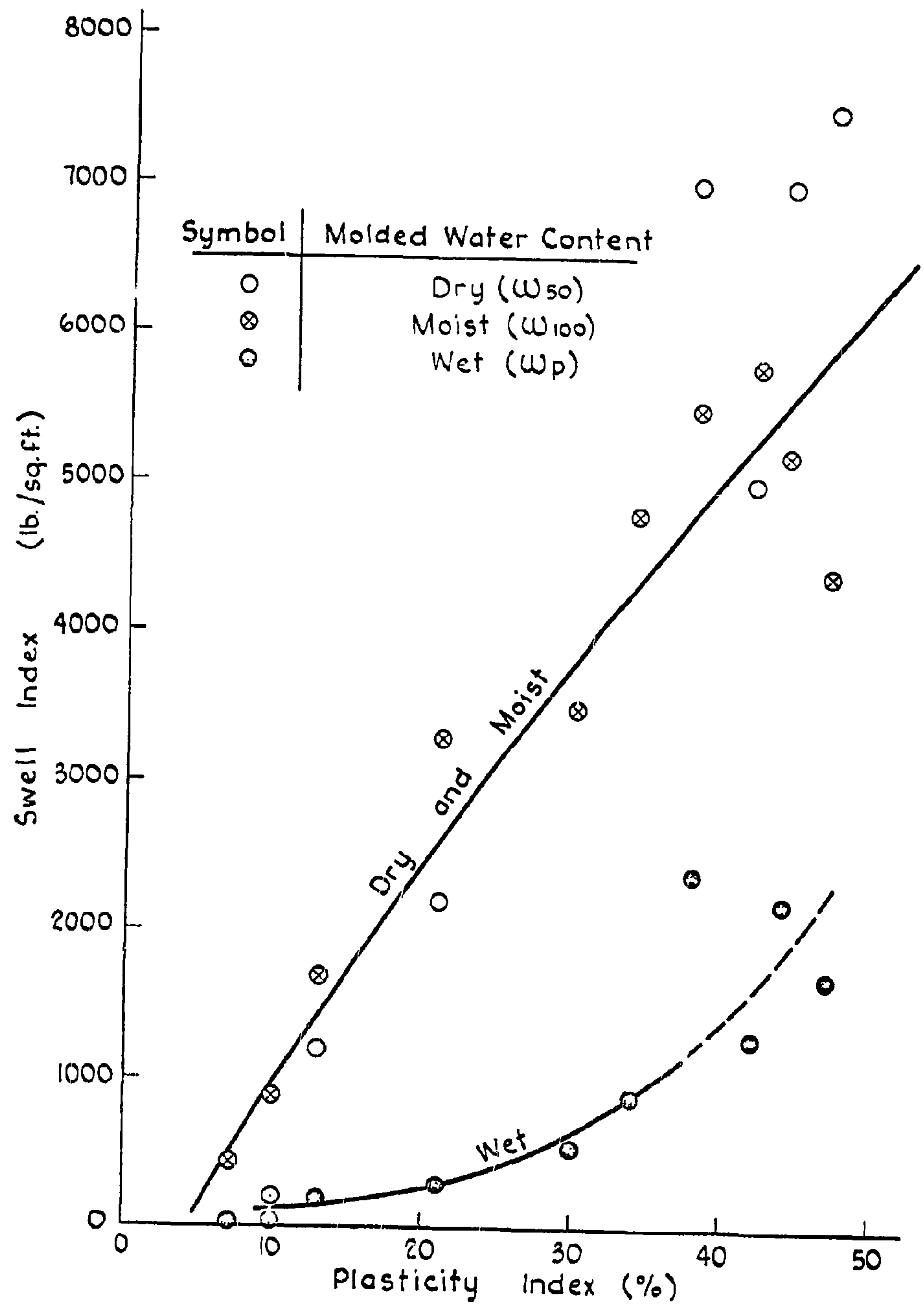


Fig. 14

SWELL INDEX VS. VOLUME CHANGE – DRYING FROM F.M.E. TO SHRINKAGE LIMIT

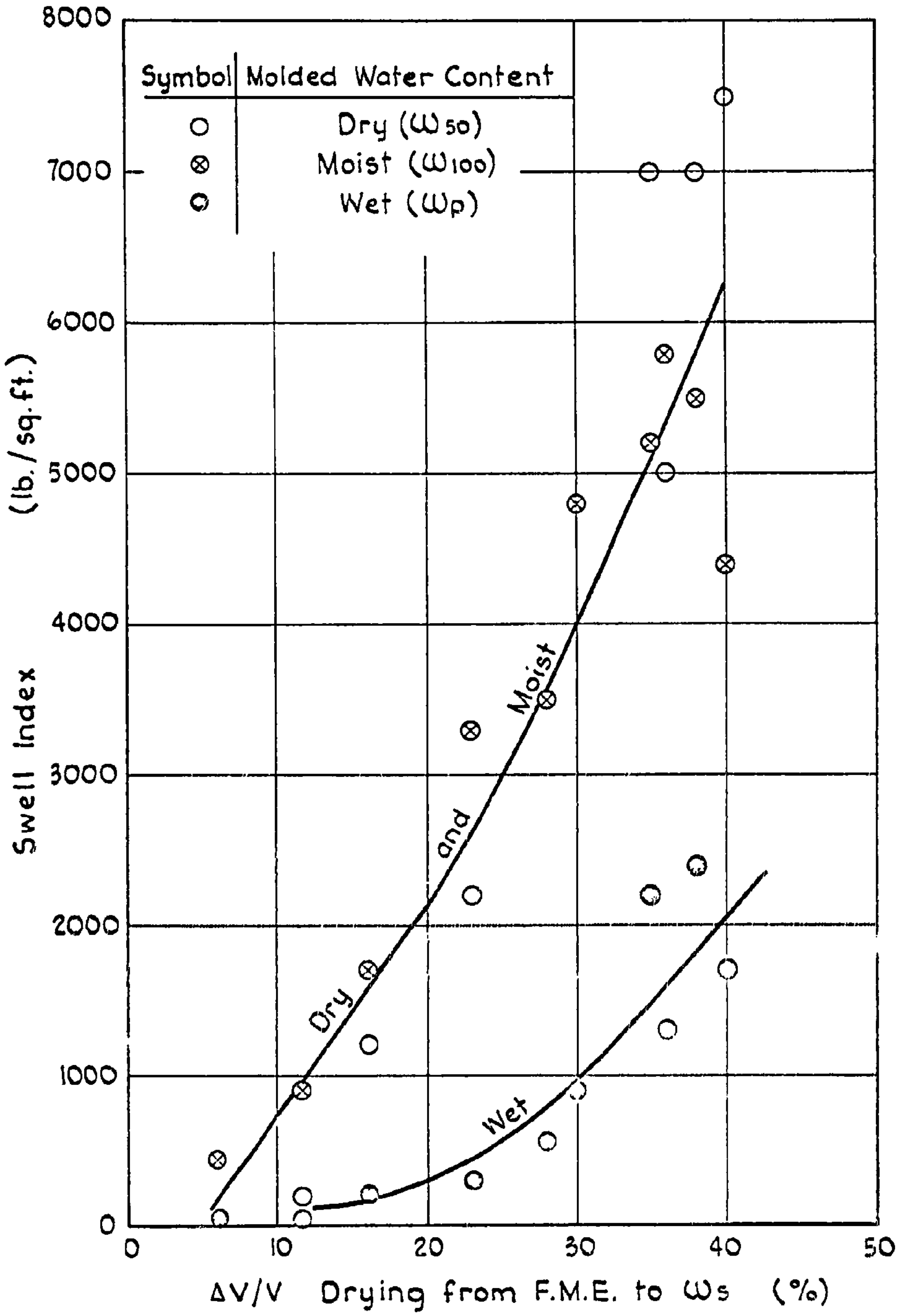


Fig. 15

SWELL INDEX VS. FINAL HEAVE

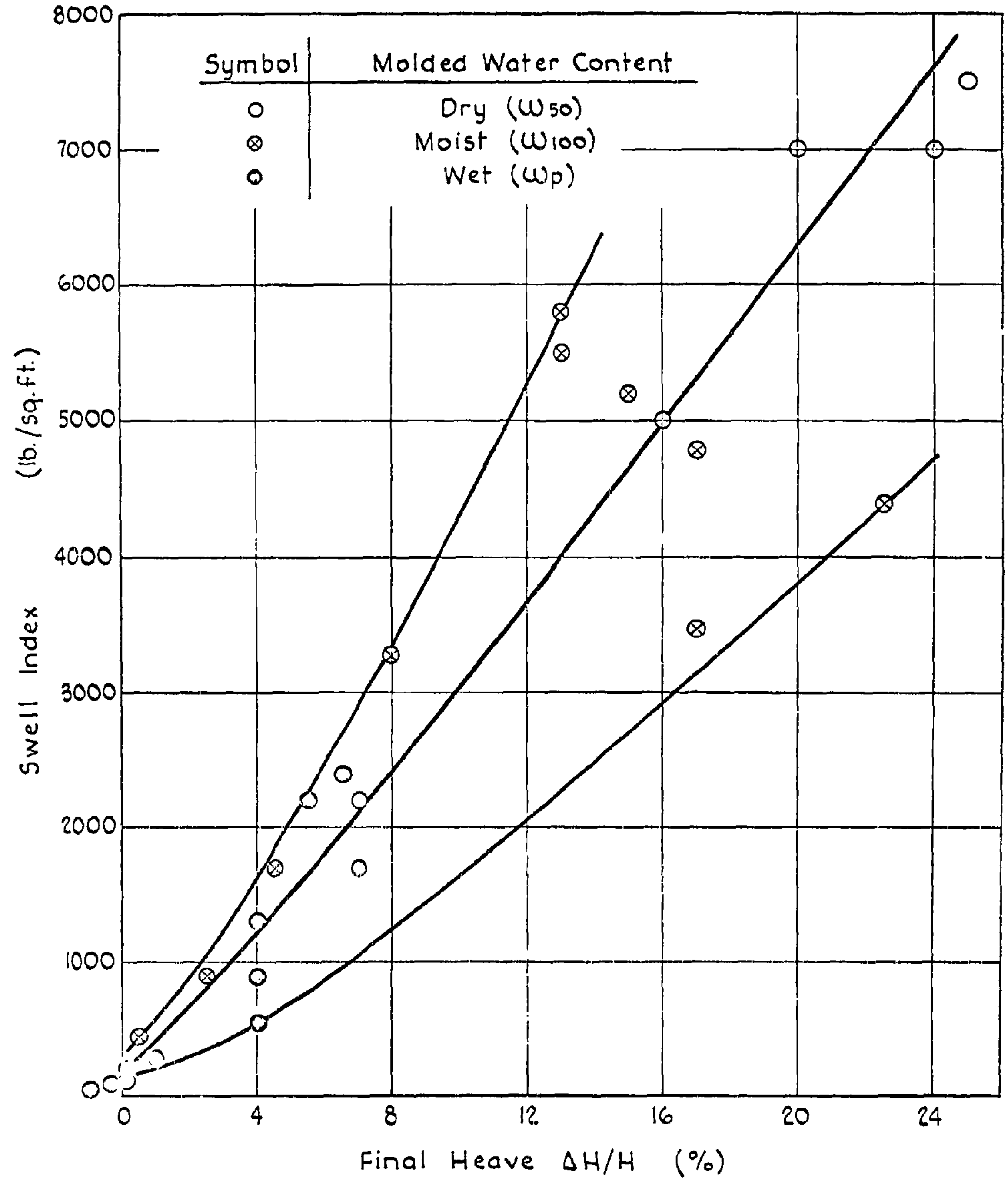


Fig. 16

SWELL INDEX VS. FINAL SWELL PRESSURE

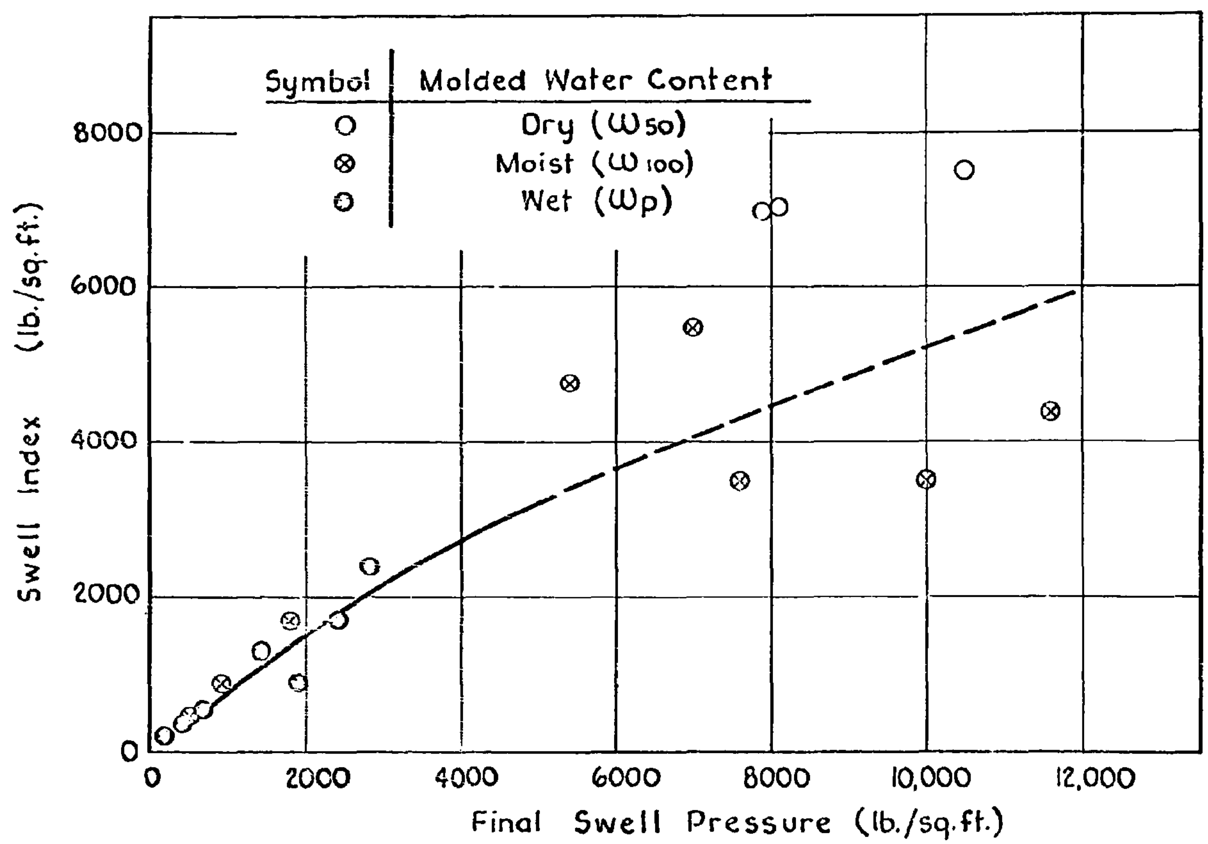


Fig. 17

TIME VS. HEAVE

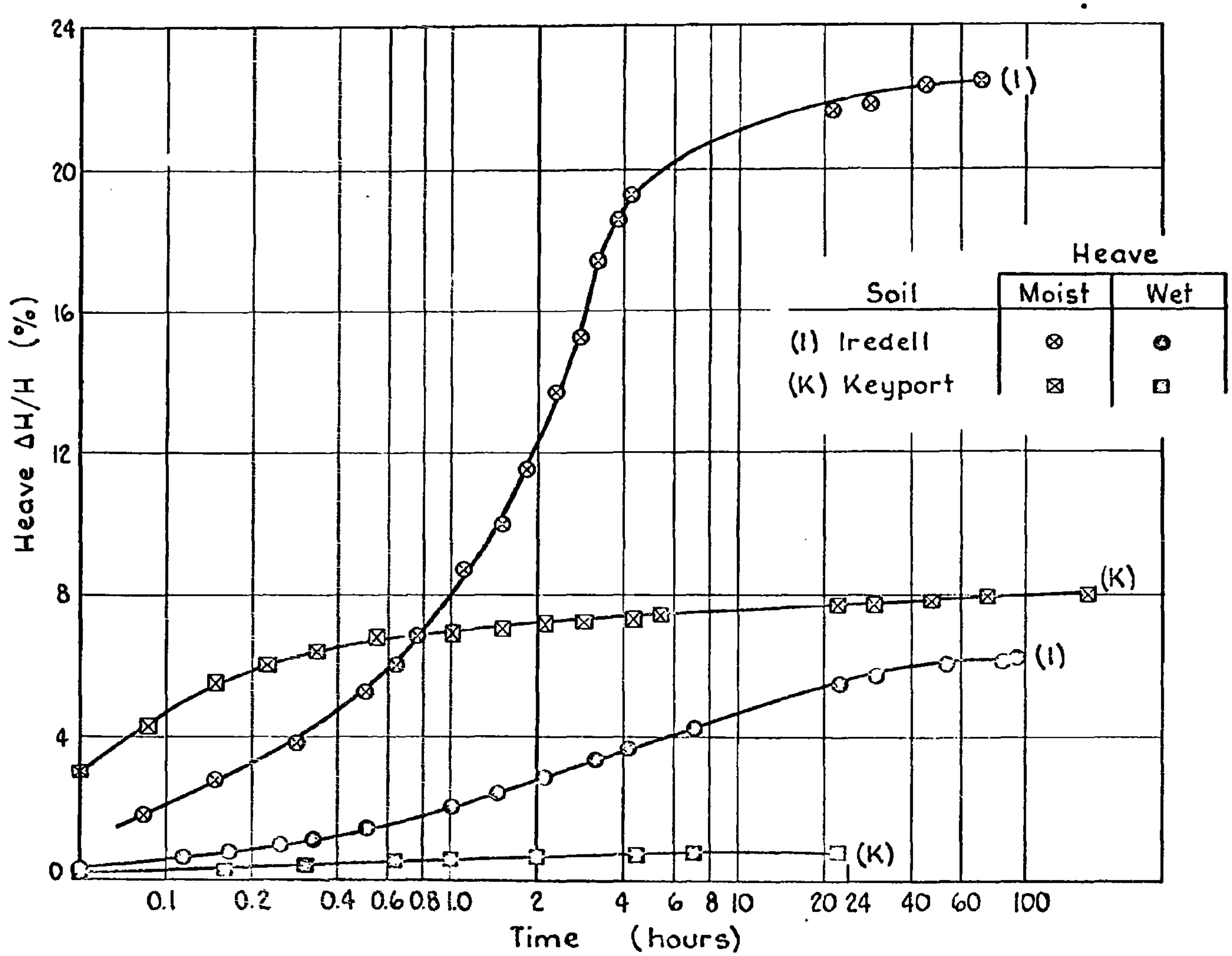


Fig.13

TIME VS. PRESSURE

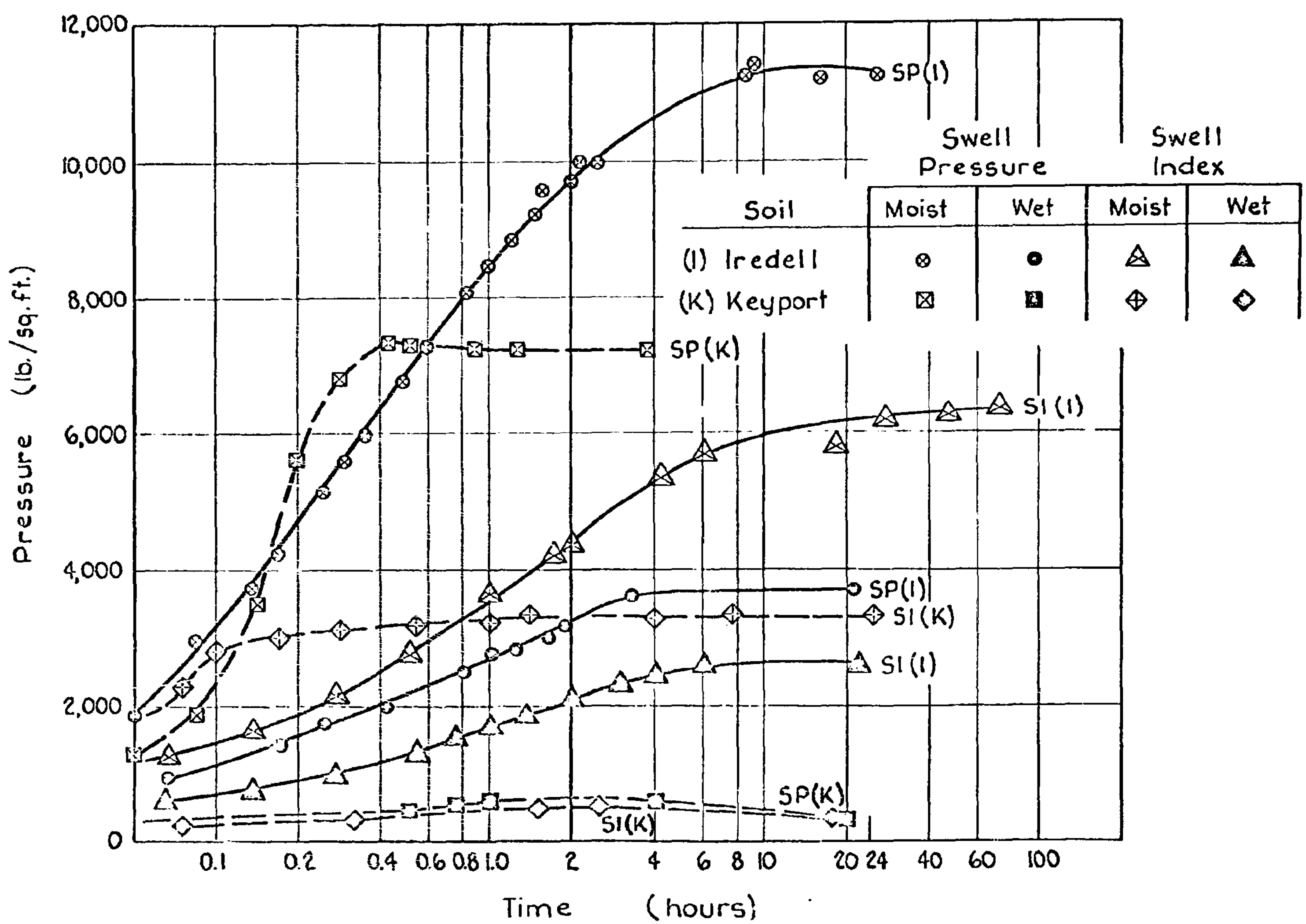


Fig.19

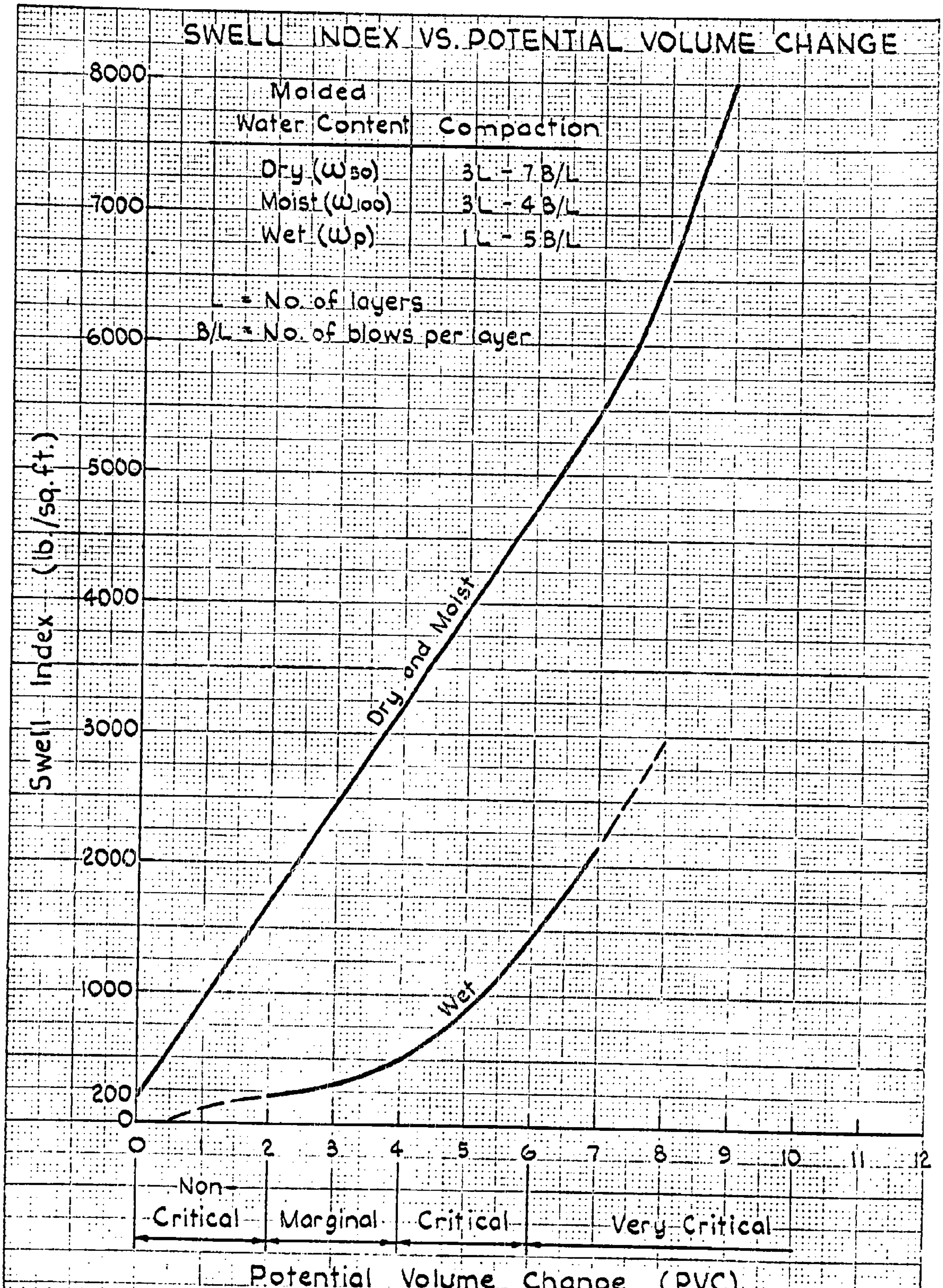
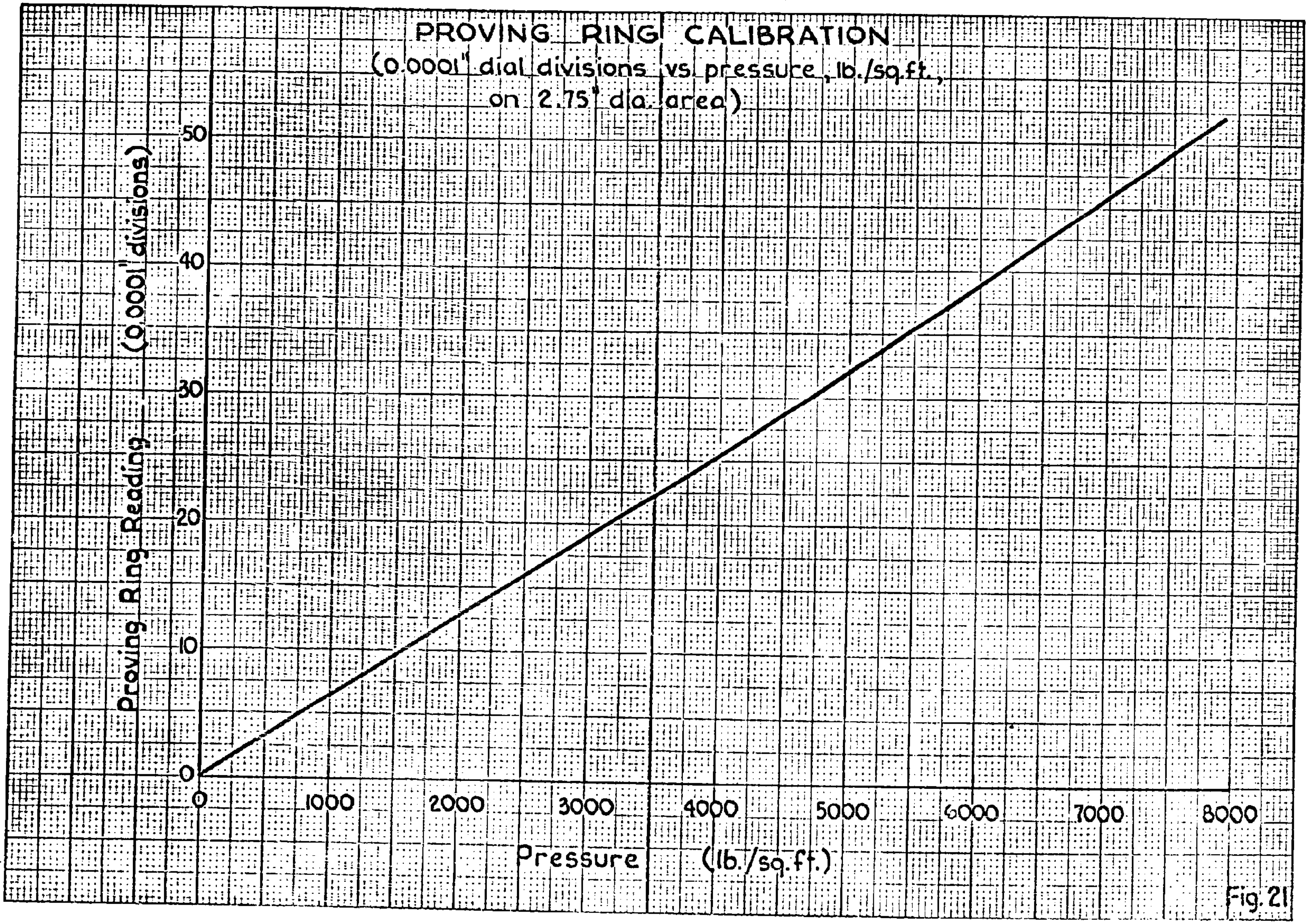

SWELL INDEX VS. POTENTIAL VOLUME CHANGE
Molded Water Content
Compaction
Dry (ω50)
3L - 7B/L
Moist (ω100)
3L - 4B/L
Wet (ωp)
1L - 5B/L
L = No. of layers
B/L = No. of blows per layer
Swell Index (lb./sq.ft.)
8000
7000
6000
5000
4000
3000
2000
1000
200
0
Dry and Moist
Wet
0
1
2
3
4
5
6
7
8
9
10
11
12
Non-Critical
Marginal
Critical
Very Critical
Potential Volume Change (PVC)

Fig. 20



PROVING RING CALIBRATION
(0.0001" dial divisions vs pressure, lb./sq.ft., on 2.75" dia. area)
Proving Ring Reading (0.0001" divisions)
0
10
20
30
40
50
0
1000
2000
3000
4000
5000
6000
7000
8000
Pressure (lb./sq.ft.)

Fig. 21

DIAL READING VS. TIME

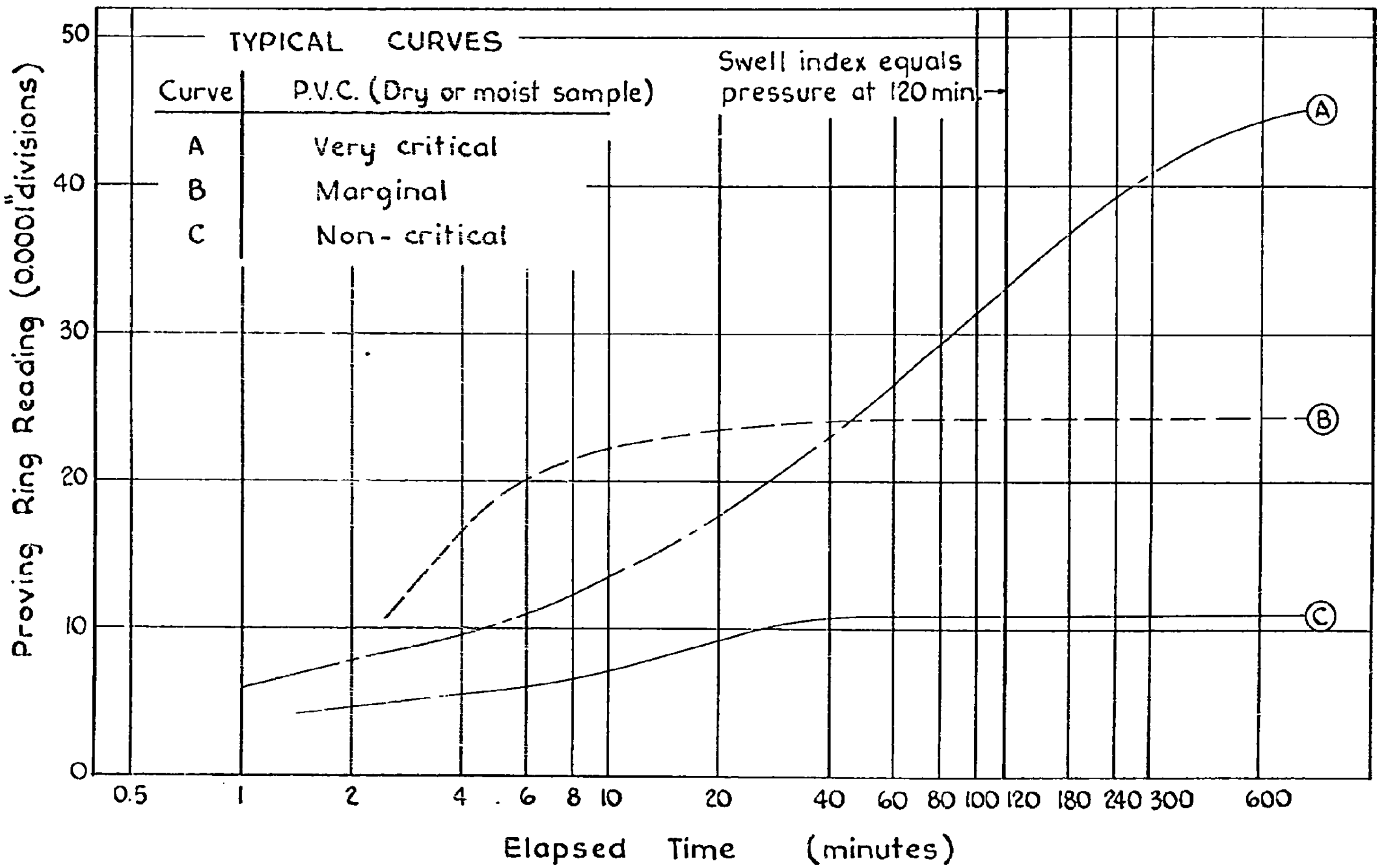


Fig. 22

EXAMPLE: MODIFICATION OF HEAVE VALUES TO ACCOUNT FOR PRESSURE VARIATIONS.

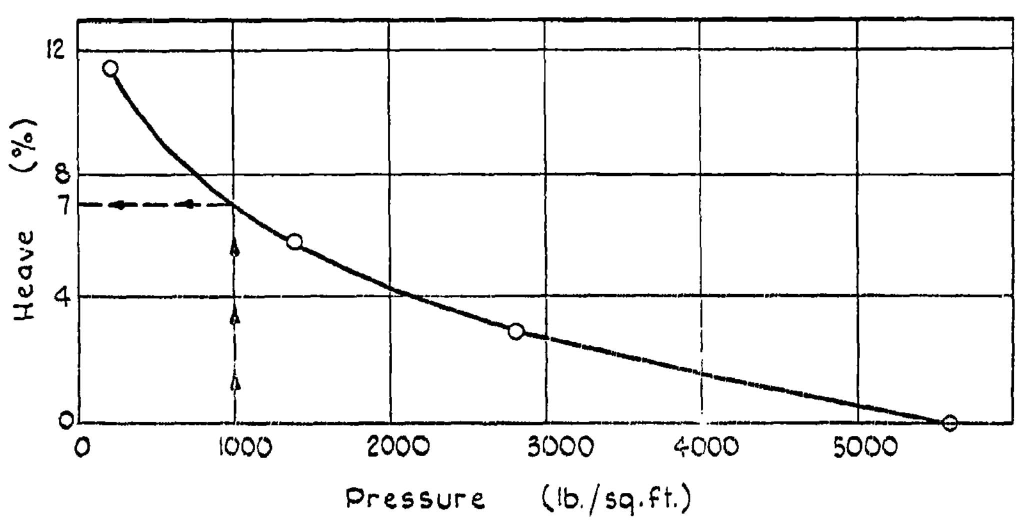
Wanted: Heave under 1000 lb./sq.ft. surcharge

Given: Swell index = 3500 lb./sq.ft.

Computations:

1. Heave under 200 lb./sq.ft. = 11.5 % (Fig. 16)
2. $11.5 \times \frac{1}{2}$ = 5.8 %
3. $11.5 \times \frac{1}{4}$ = 2.9 %
4. Swell Pressure = 5600 lb./sq.ft. (Fig. 13)
5. $5600 \times \frac{1}{2}$ = 2800 lb./sq.ft.
6. $5600 \times \frac{1}{4}$ = 1400 lb./sq.ft.

Plot :

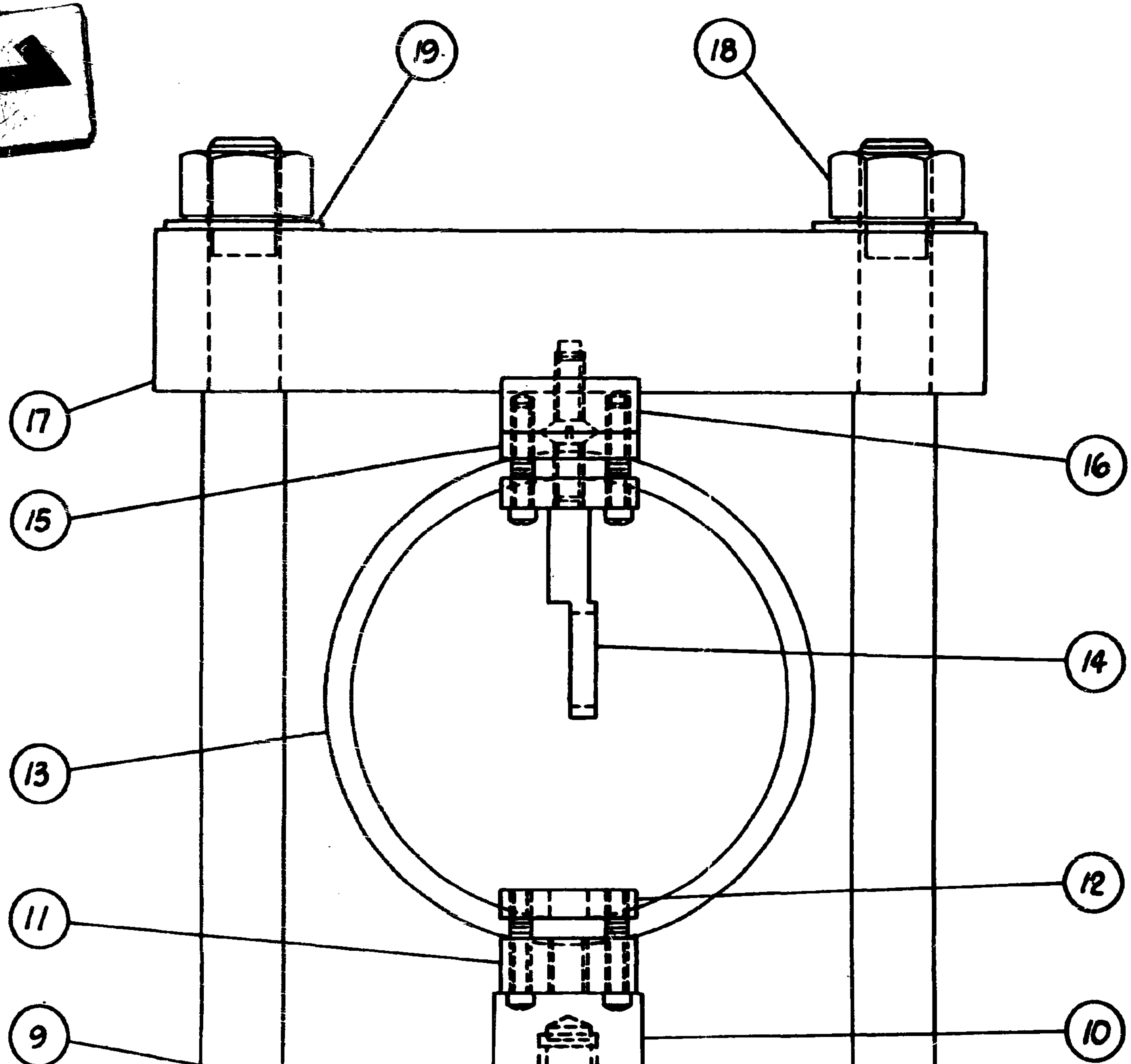


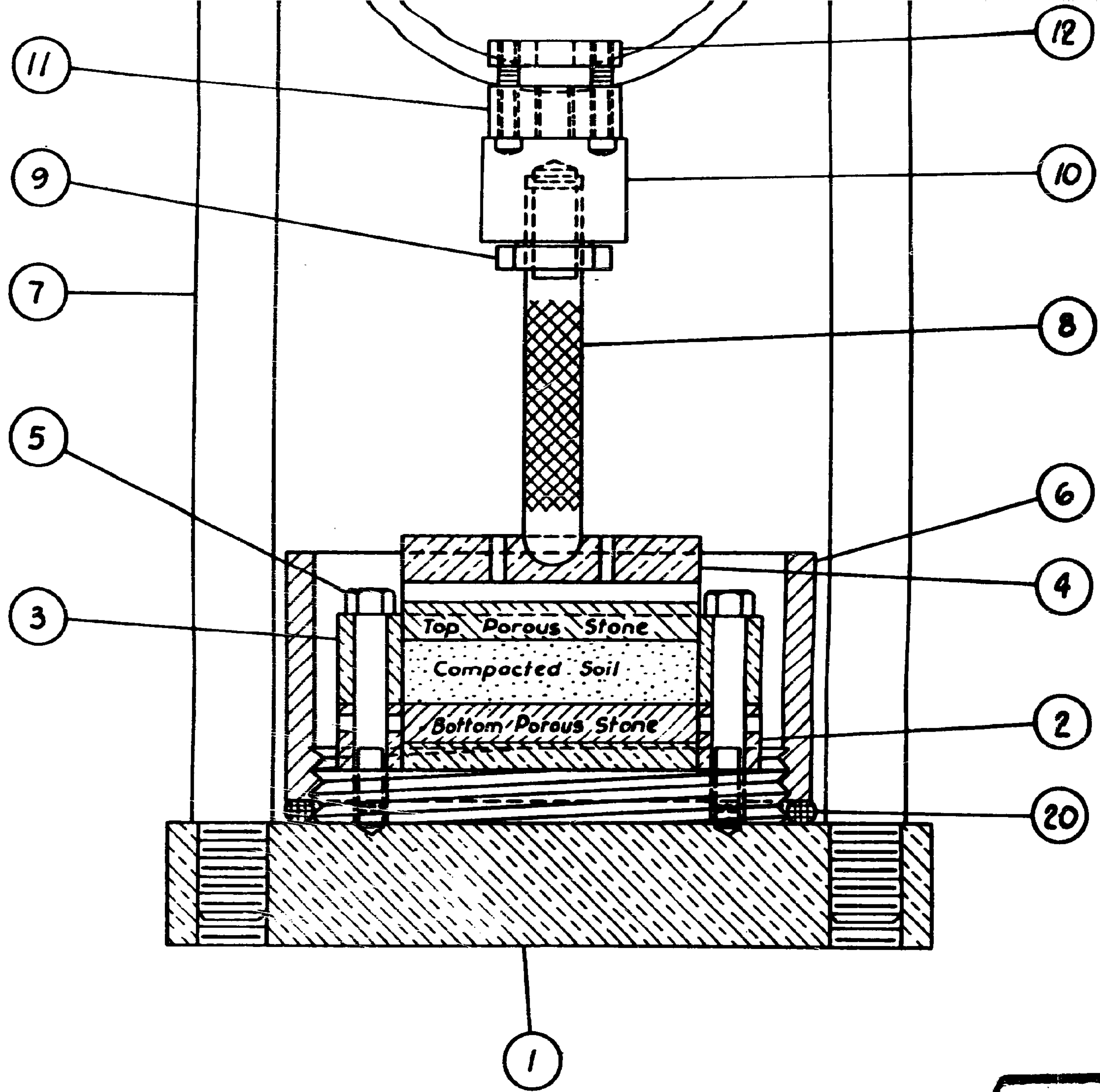
Answer: Heave under 1000 lb./sq.ft. surcharge = 7%

Note: Due to scatter in Figs. 13 and 16, above value could easily be in error by several percent heave.

Fig. 23

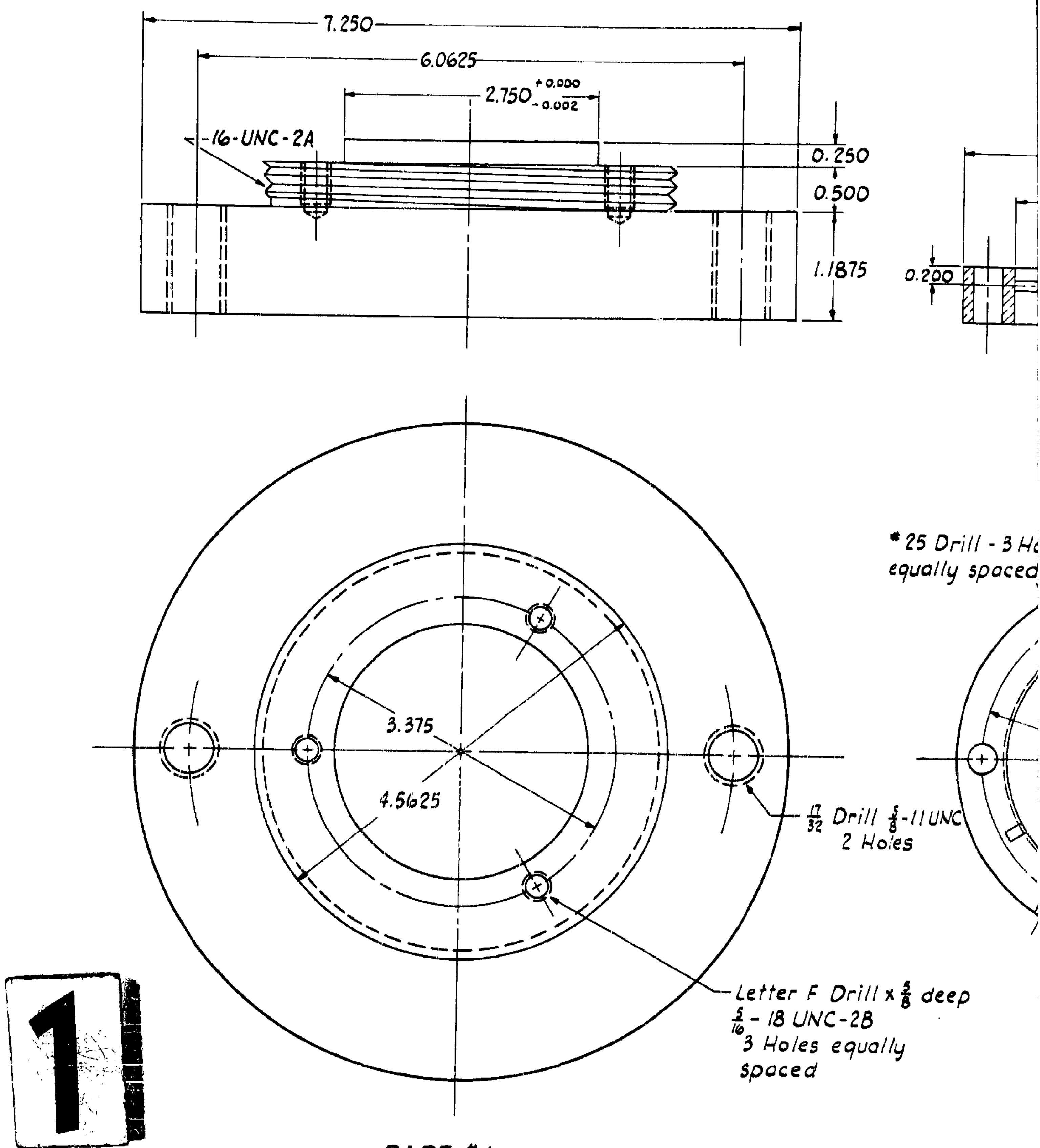
1



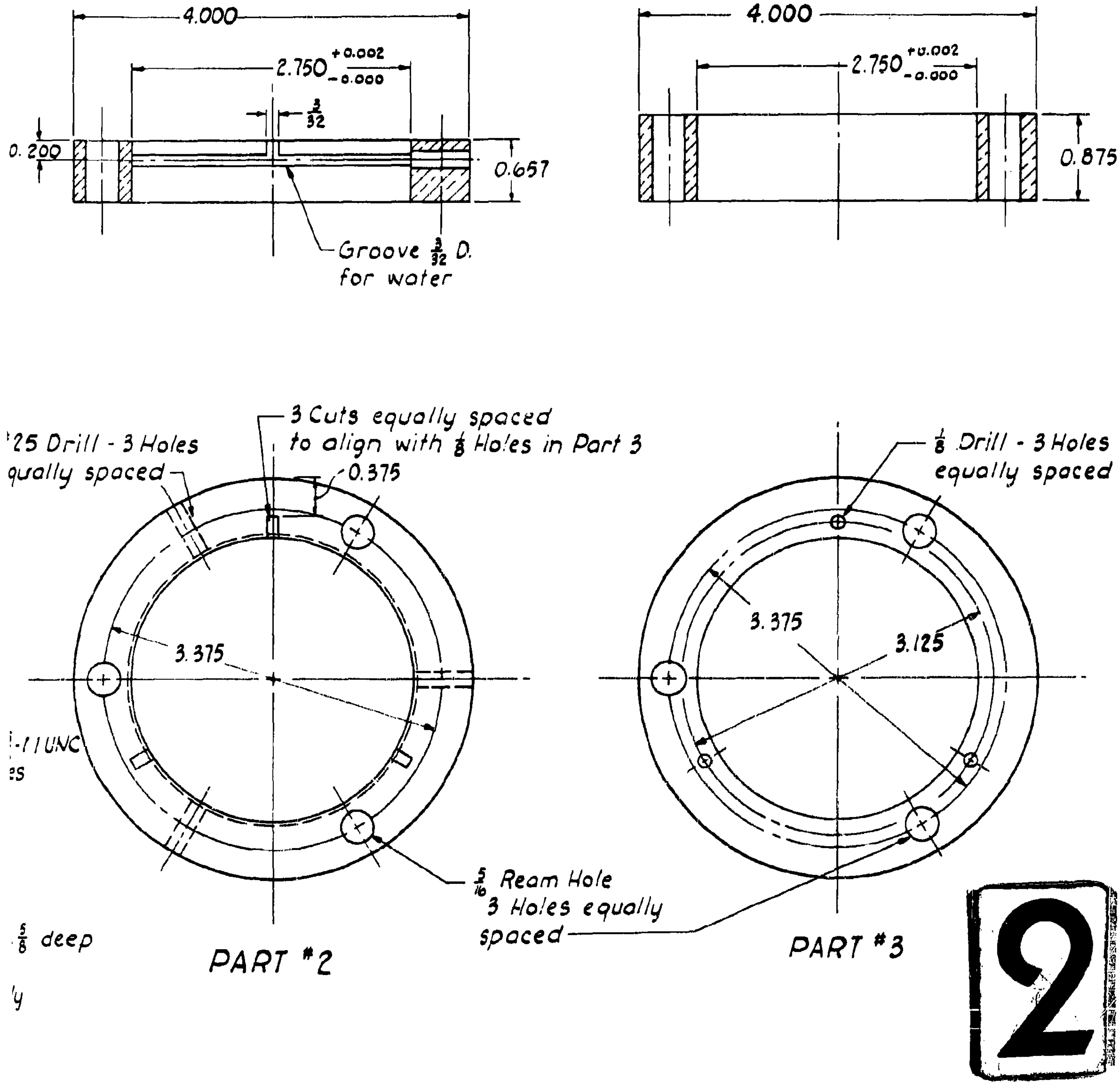


12
11
10
9
7
8
5
6
4
3
Top Porous Stone
Compacted Soil
Bottom Porous Stone
2
20
1

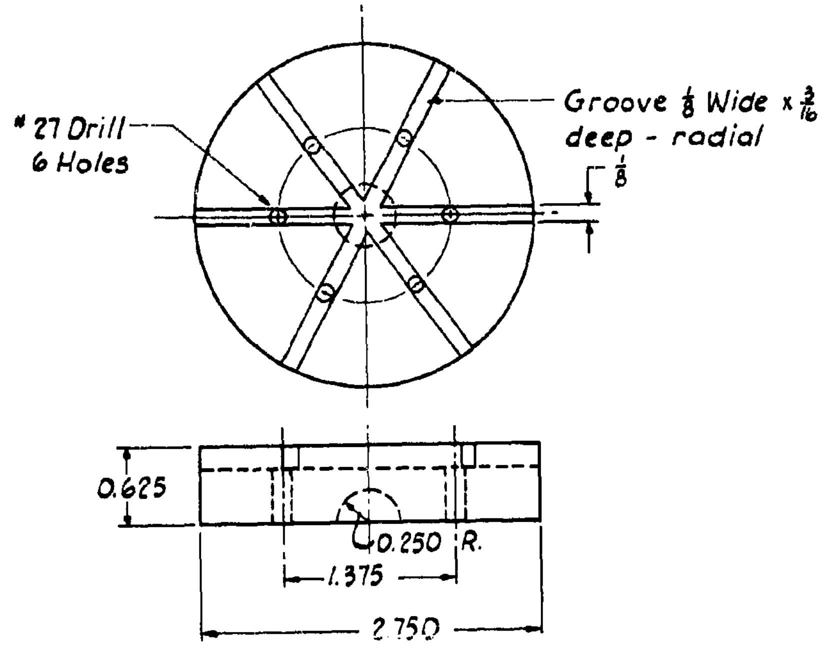
Drawing No. 1


7.250
6.0625
2.750 +0.000 -0.002
-16-UNC-2A
0.250
0.500
1.1875
0.200
3.375
4.5625
17/32 Drill 5/8-11 UNC
2 Holes
Letter F Drill x 5/8 deep
5/16 - 18 UNC-2B
3 Holes equally
spaced

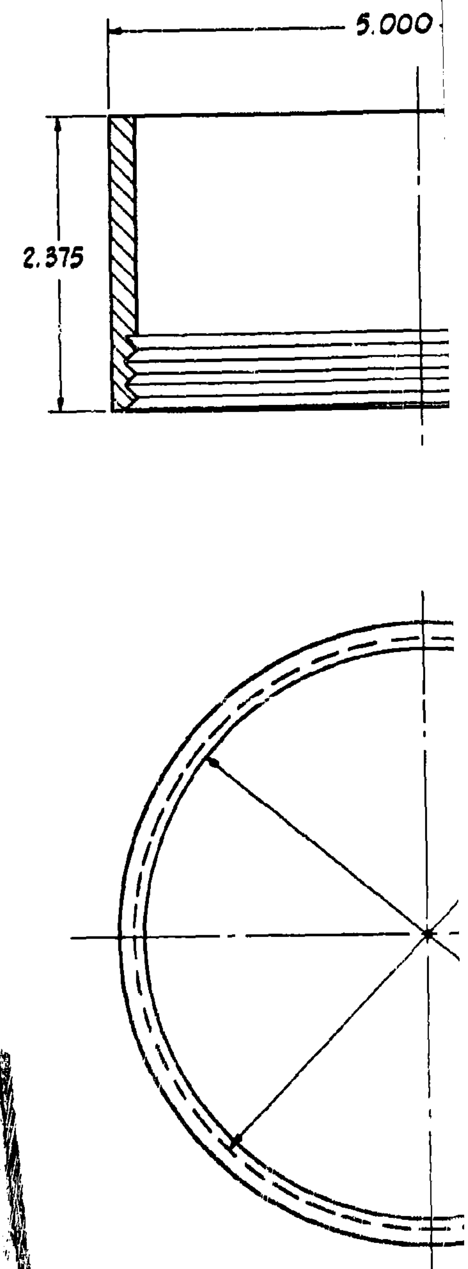
PART #1


4.000
2.750 +0.002 −0.000
3/32
0.200
0.657
Groove 3/32 D. for water
4.000
2.750 +0.002 −0.000
0.875
3 Cuts equally spaced to align with 1/8 Holes in Part 3
25 Drill - 3 Holes
qually spaced
0.375
3.375
-11 UNC
es
5/8 deep
y
5/16 Ream Hole 3 Holes equally spaced
PART #2
1/8 Drill - 3 Holes equally spaced
3.375
3.125
PART #3
2

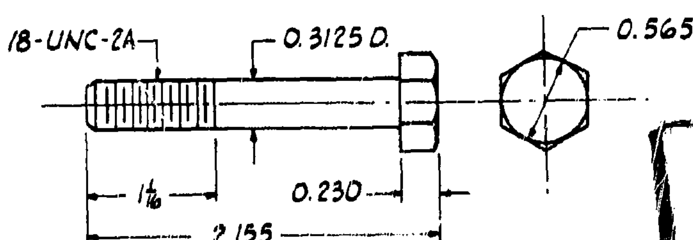
Drawing No. 2



PART #4

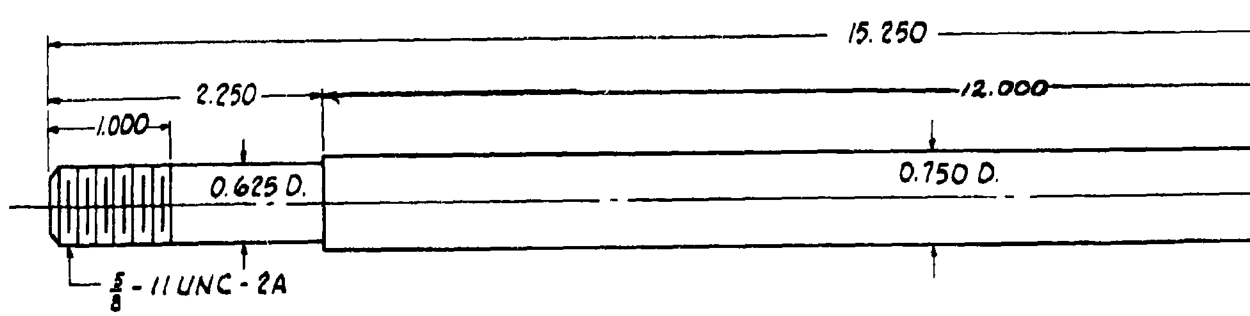


PART

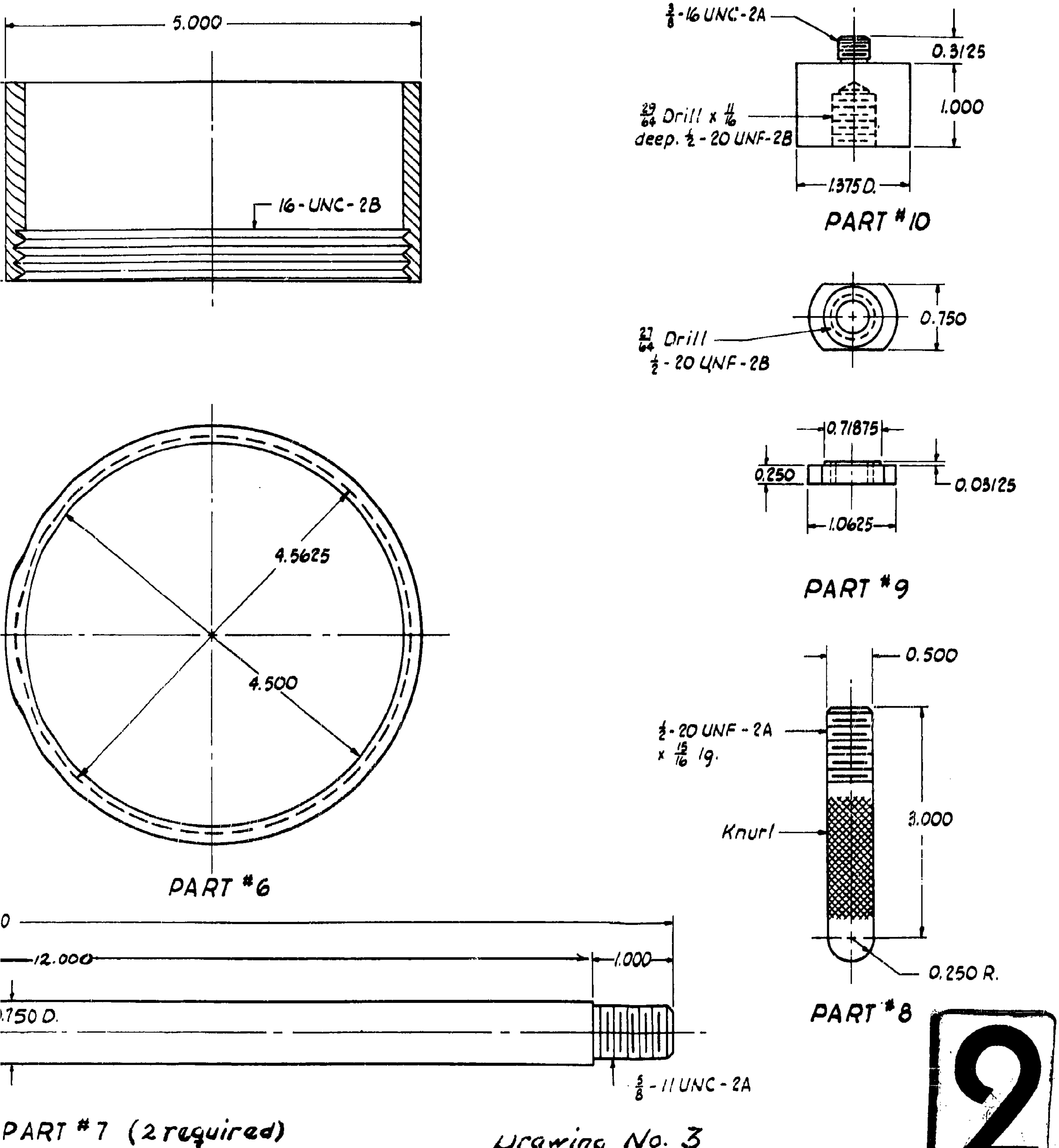


PART #5

1

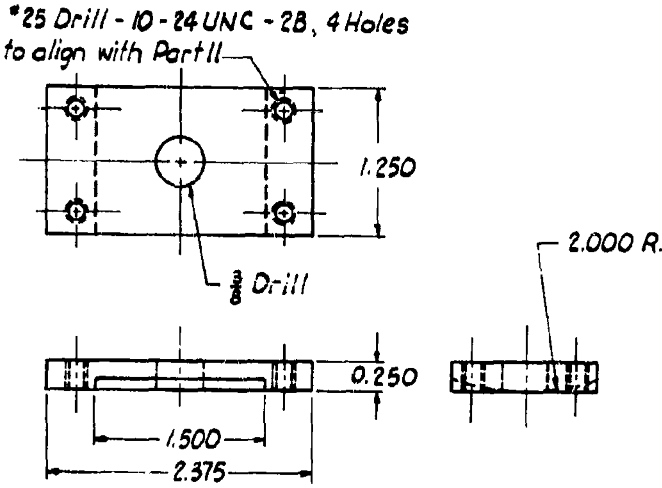


PART #7 (2 requir

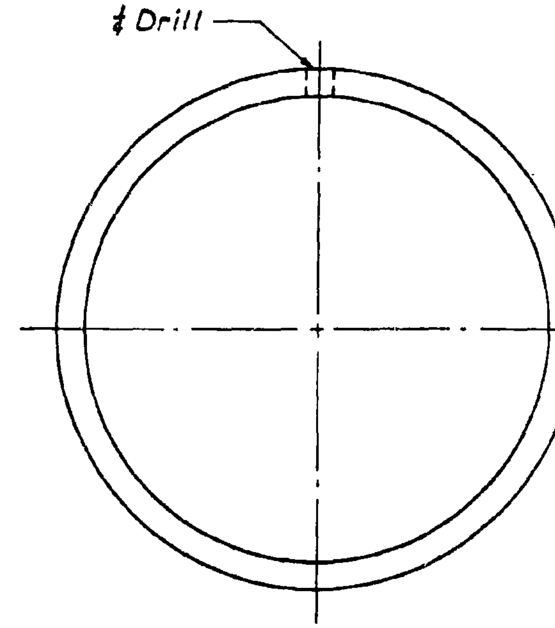


Drawing No. 3

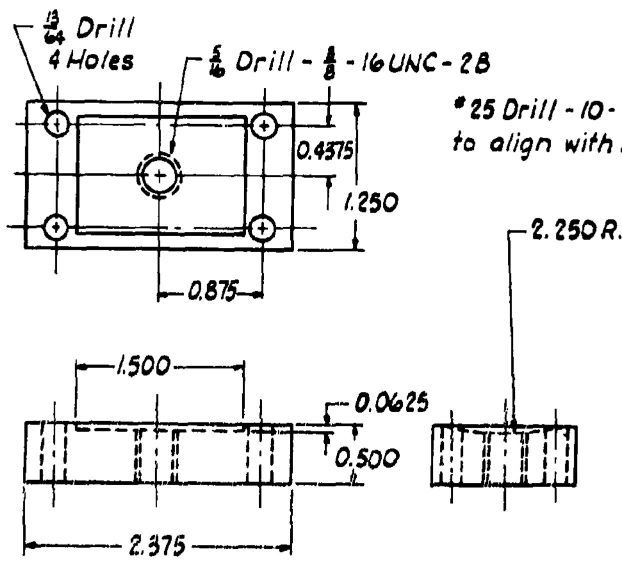
2



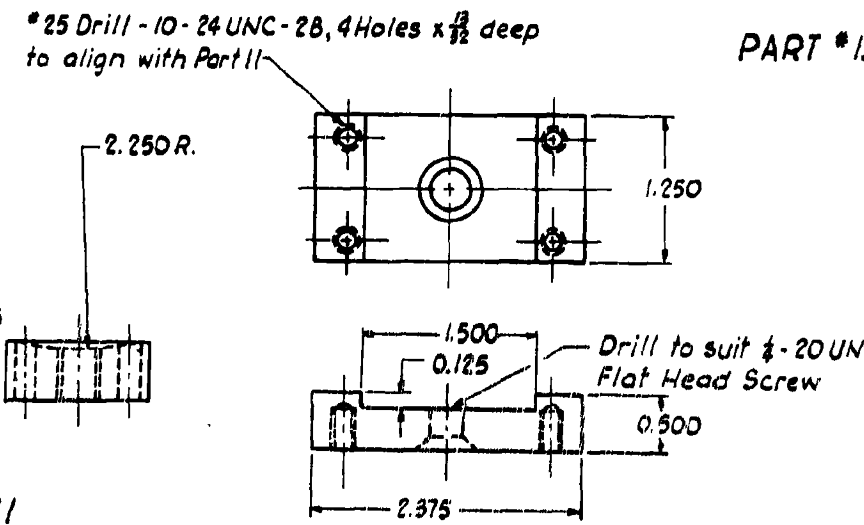
PART #12



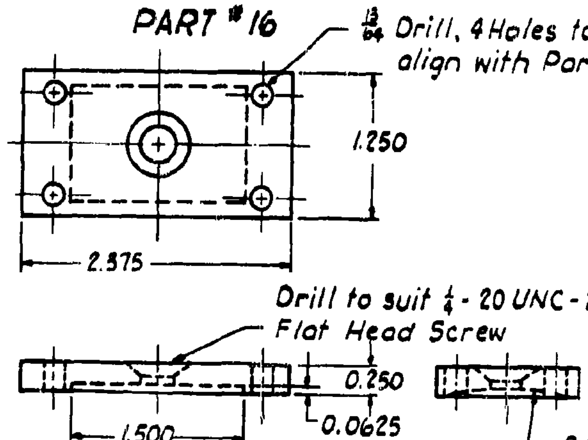
PART #13



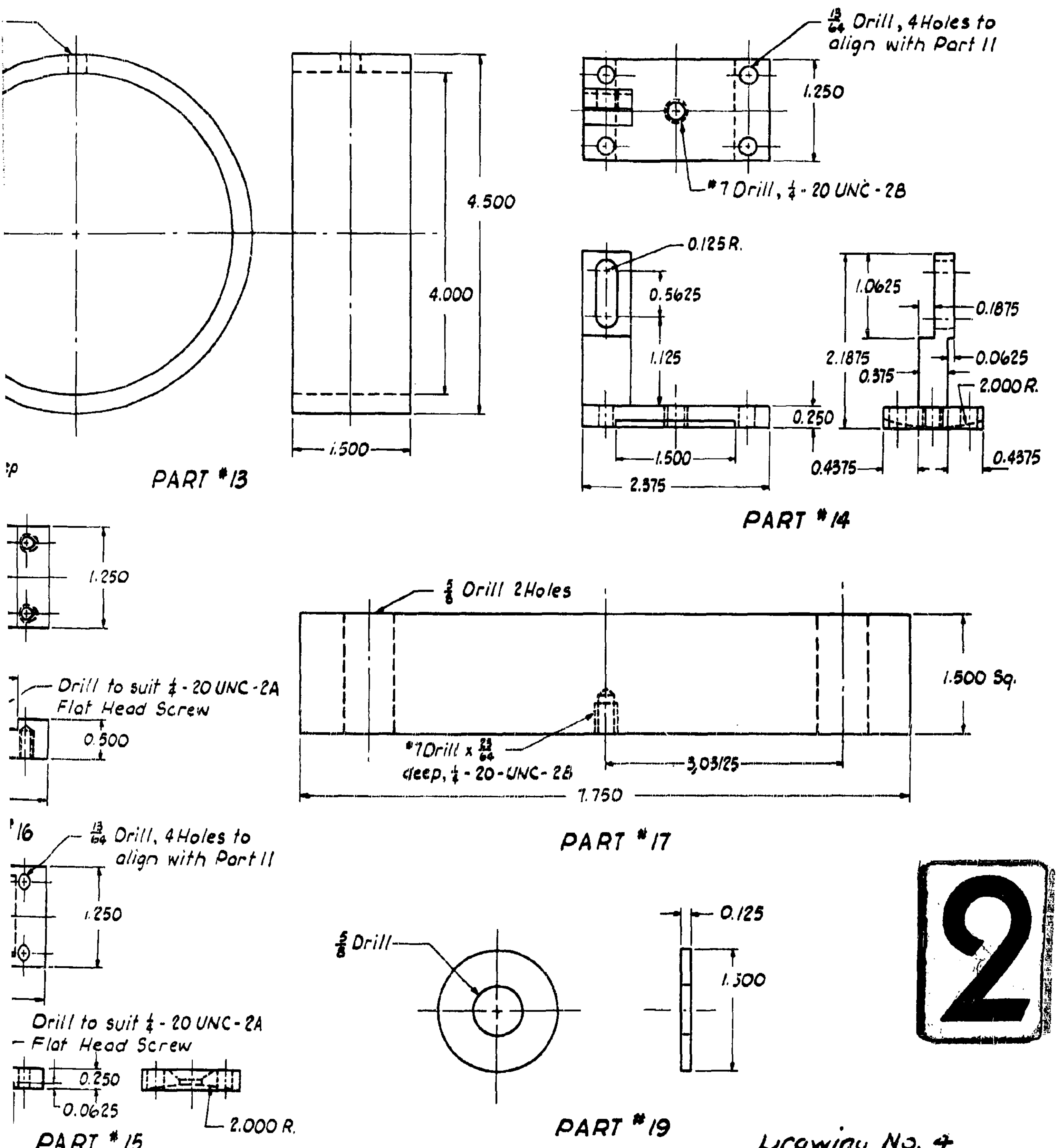
PART #11



PART #16

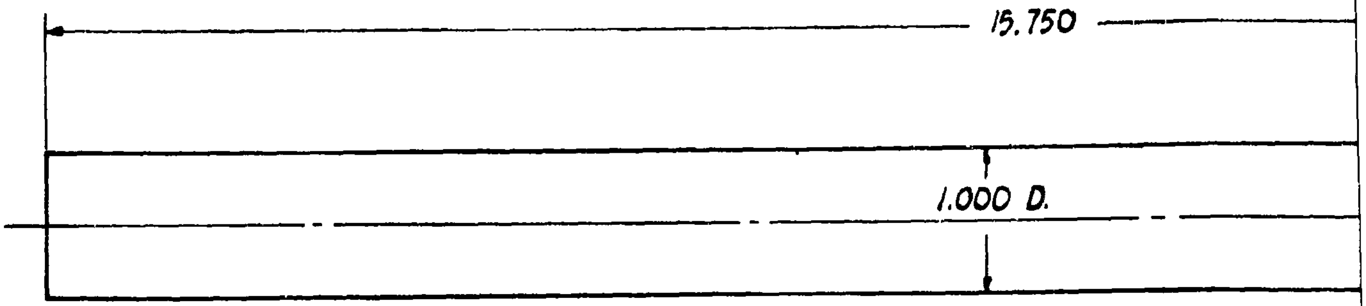


PART #15

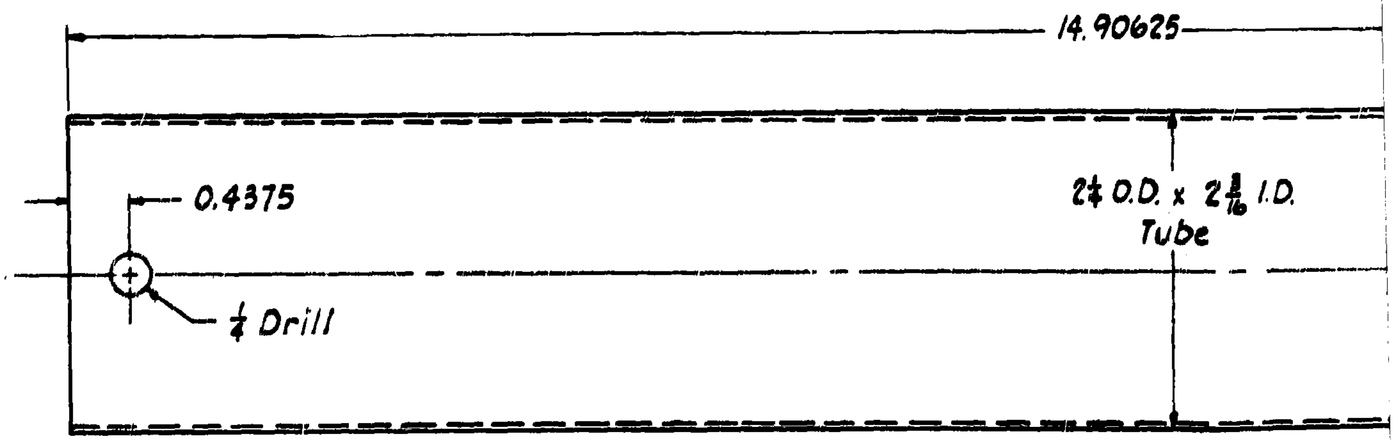


13/64 Drill, 4 Holes to align with Part II
1.250
#7 Drill, 1/4 - 20 UNC - 2B
4.500
4.000
1.500
PART #13
0.125 R.
0.5625
1.125
0.250
1.500
2.375
1.0625
0.1875
2.1875
0.0625
0.375
2.000 R.
0.4375
0.4375
PART #14
1.250
Drill to suit 1/4 - 20 UNC - 2A Flat Head Screw
0.500
5/8 Drill 2 Holes
1.500 Sq.
#7 Drill x 23/64 deep, 1/4 - 20 - UNC - 2B
3.03125
7.750
PART #17
16
13/64 Drill, 4 Holes to align with Part II
1.250
Drill to suit 1/4 - 20 UNC - 2A Flat Head Screw
0.250
0.0625
2.000 R.
PART #15
5/8 Drill
0.125
1.500
PART #19
2

Drawing No. 4



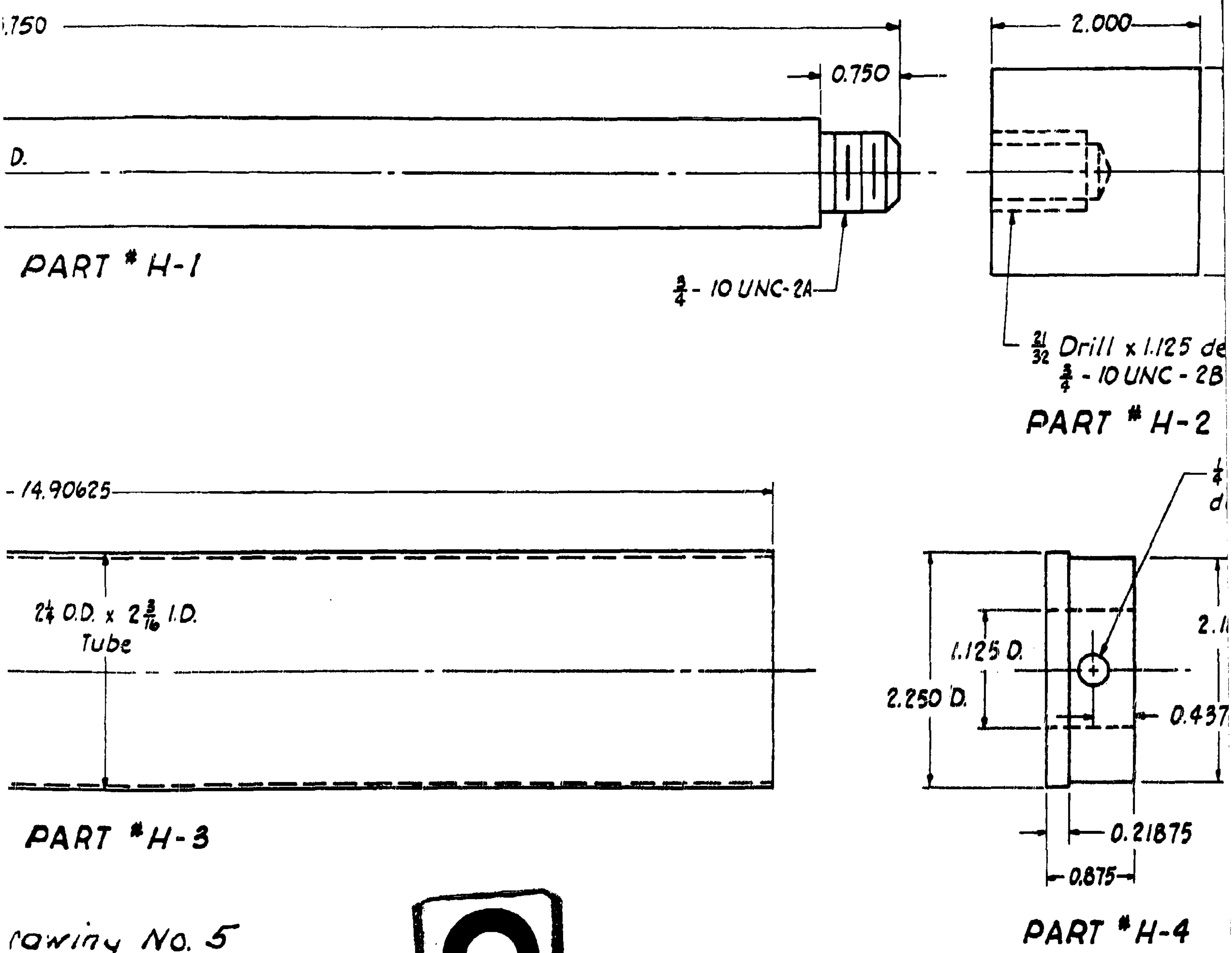
PART # H-1



PART # H-3

1

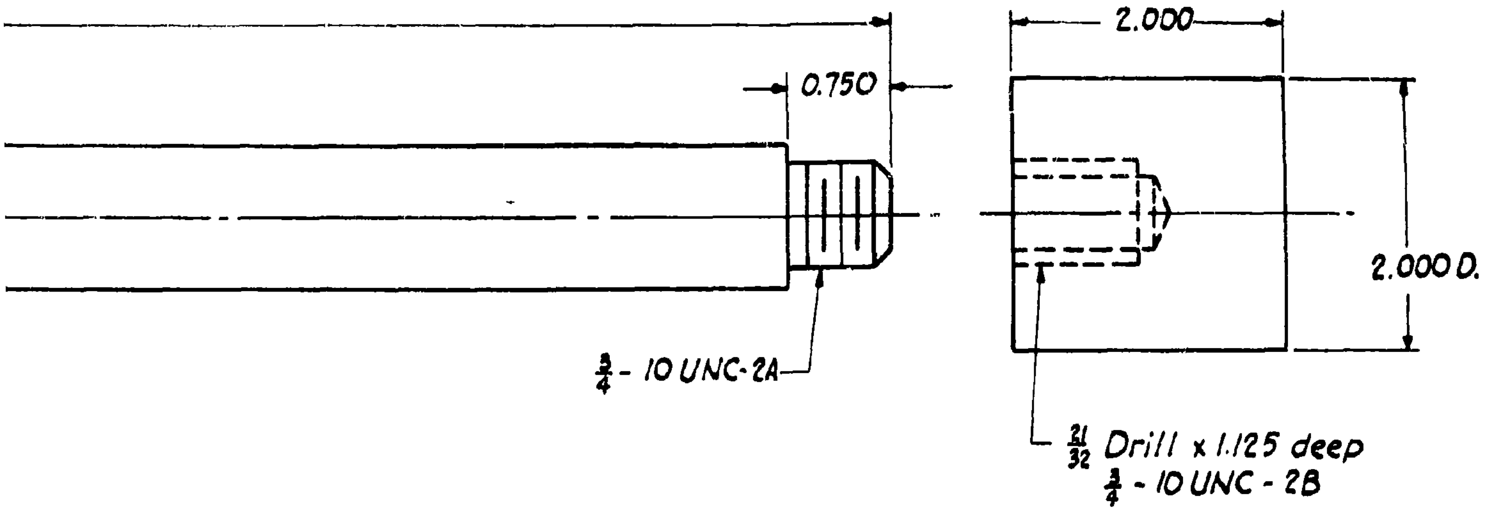
Drawing No. 5


.750
0.750
D.
PART # H-1
3/4 - 10 UNC-2A
2.000
21/32 Drill x 1.125 de
3/4 - 10 UNC - 2B
PART # H-2
- 14.90625
2 1/4 O.D. x 2 3/16 I.D.
Tube
PART # H-3
1/4
d
2.1
1.125 D.
2.250 D.
0.437
0.21875
0.875
PART # H-4

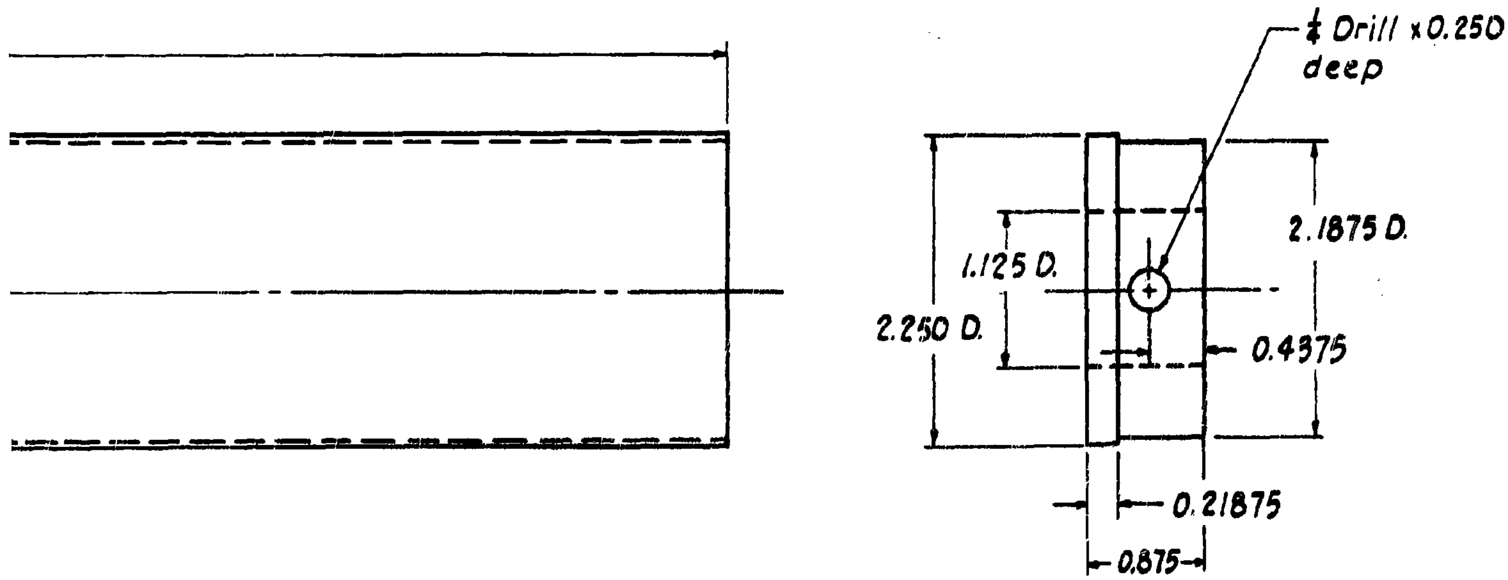
rawing No. 5

2

3



PART # H-2



PART # H-4

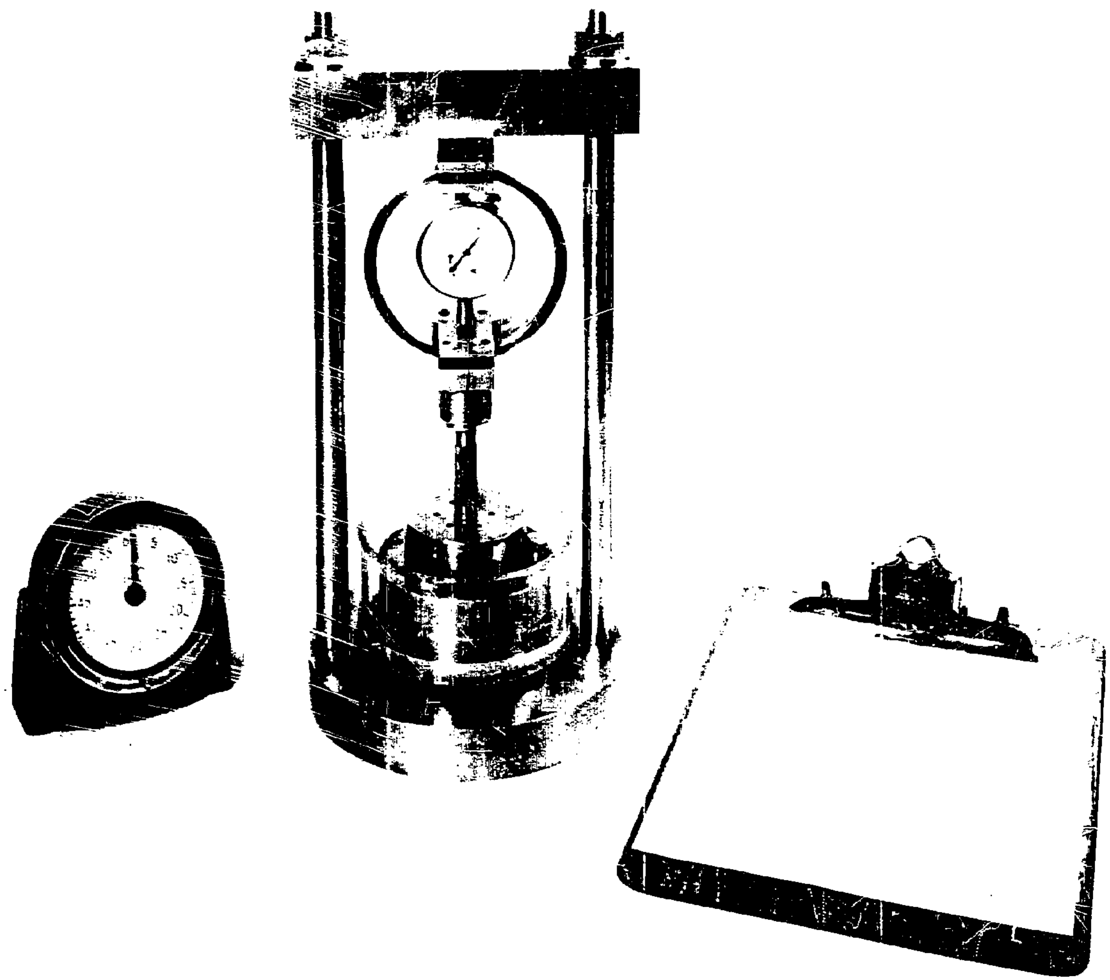


PLATE I

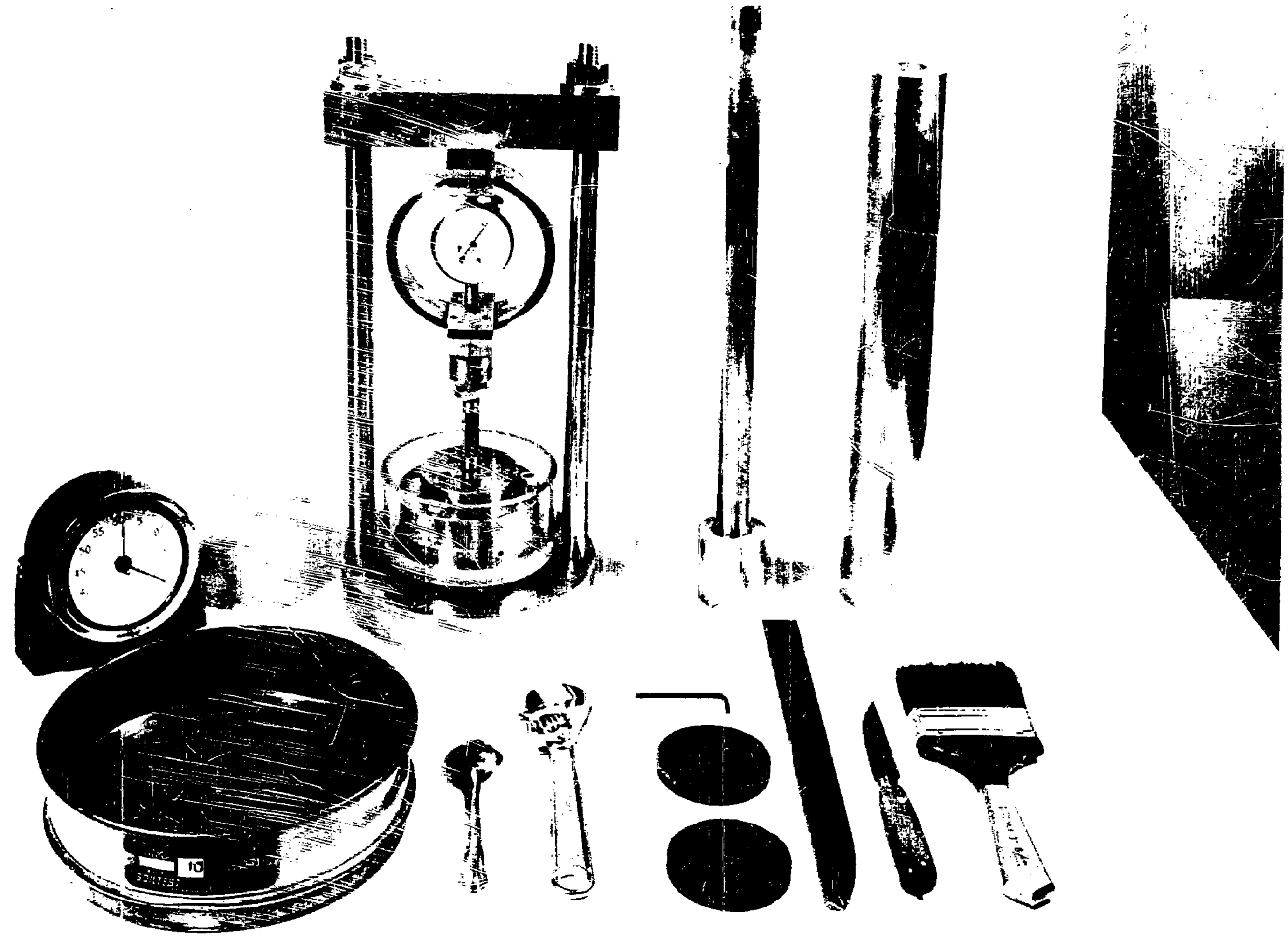


PLATE II

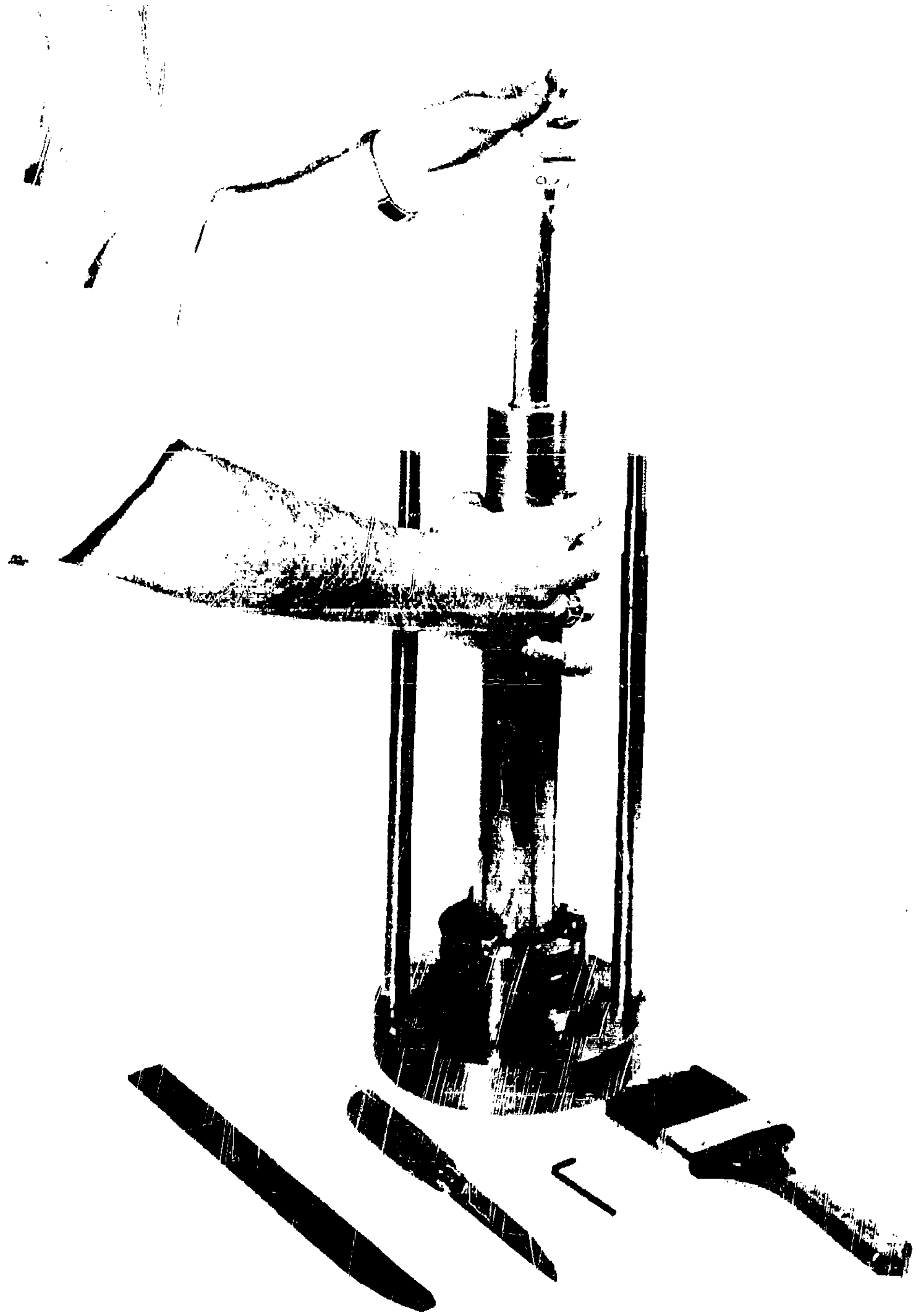


PLATE III

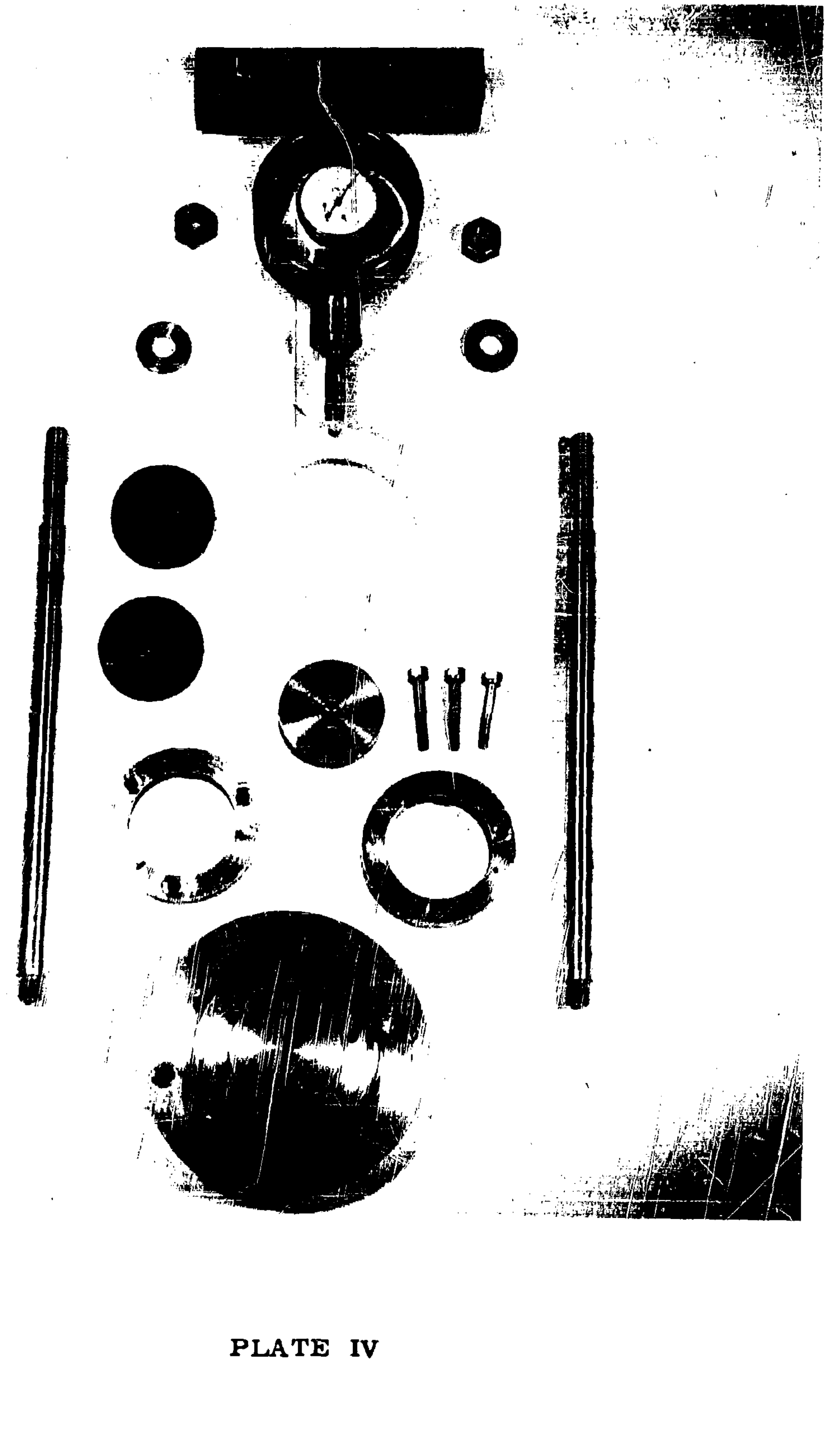


PLATE IV