A STUDY OF THE SWELLING CHARACTERISTICS OF EXPANSIVE SOILS UNDER CONTROLLED CONDITIONS OF LATERAL AND VERTICAL CONFINEMENT.

BY

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CONTENTS

List of Tables

LIST OF FIGURES

NOTATION

W_i	=	Initial moisture content.
W_f	=	Final moisture content.
S_i	=	Initial degree of saturation.
S_f	=	Final degree of saturation.
$\frac{\Delta H_T}{H}\%$	=	Total percentage increase in height after initial consolidation.
$\frac{\Delta H_i}{H}\%$	=	Percentage increase in height above the initial height.
$SP_{max.}$	=	Maximum swelling pressure.
γ_{di}	=	Initial dry density.

SUMMARY

Expansive clays cause considerable damage to structures erected on them. Moisture variations cause these soils to expand and shrink, producing differential heaving, detrimental to buildings. A similar problem is also encountered in roads and airfield construction, where moisture variations in the compacted subgrade cause volume changes and subsequent waving of the paved surface. The expansive clays are particularly detrimental to hydraulic structures placed on them. The degree of expansion, the uplift pressures and loss of stability are great because of the presence of water which eventually saturates the soil.

The possibility of damage to structures due to swelling of clays is complicated by the problem of identifying those soils likely to possess undesirable characteristics. Some research has been carried out by various investigators to develop a reliable means of predicting the potential expansion characteristics of clays. Microscopic examinations, X-ray diffraction determinations and differential thermal analysis are valuable to determine the presence of objectionable clay minerals which may ultimately cause expansion. A number of empirical approaches, based on the use of index tests, have been developed as an aid to anticipate the

expansion characteristics. A simple free swell test provides a quick qualitative means of assessing expansiveness. Indices such as liquid limit, plastic limit, shrinkage limit and percentage clay fraction are good indicators of expansion characteristics of clays when considered together. It is suggested that the shrinkage index (defined as liquid limit minus shrinkage limit) and the percentage clay fraction may be a reliable index to evaluate qualitatively the swelling potential of a soil. Other indices used as an aid in this respect are linear shrinkage, specific surface area, and heat of wetting. The relative importance of each of these indices with regard to the expansion properties has been discussed.

Some research has been carried out by various investigators to study the swelling and the swelling pressures developed in a soil under various conditions of confinement. It has been found that moisture, density and soil structure are among the most important factors influencing volume change. In the present study, laboratory tests were carried out to study the swelling characteristics of expansive soils. Tests were carried out by two different methods - (a) Indirect method and (b) Direct method. In the indirect method, a standard consolidometer with suitable modifications was used to study the rate and amount of swell under various conditions of initial dry density, initial moisture content, and applied load. The data shows that the amount of swell is inversely proportional to the restraining pressure. From the tests, an attempt was made to deduce the maximum balancing swelling pressure, i.e. the pressure at which the sample undergoes no volume change under the particular conditions of confinement. It has been observed that if the soil is at a low initial dry density and low initial moisture content, and it is subjected to heavy initial load, very high swelling pressures develop with increase in moisture content.

In the direct method, the tests were carried out in an apparatus specially designed to find the absolute maximum swelling pressures developed in a soil under complete lateral and vertical confinement. This apparatus could also be conveniently used to find the swelling pressures developed in a soil, after allowing it to expand vertically to a predetermined value. It has been found that the swelling pressures are greatly reduced if expansive soils are permitted to swell a little. The rate of build up of pressure and the maximum pressure developed in a soil under conditions of complete lateral and vertical confinement and at various values of initial dry density and initial moisture content were observed.

The results obtained by the two methods for the determination of swelling pressure developed under conditions of complete confinement were compared and they showed good agreement. An effort was made to study the relationships between swelling pressure, initial dry density, initial moisture content, and volume change. The data shows the significant effect of dry density and moisture content on swelling pressures. A straight-line relationship between swelling pressure and initial moisture content was obtained. From the limited data, it seems that there may be no consistent relationship between the swell of soils under some standard conditions and swelling pressures measured under conditions of no expansion. An attempt was also made to study the influence of the height of the sample on the maximum pressure developed under particular conditions and to see whether undisturbed samples behave similar to disturbed samples.

Some recommendations have been outlined for future work. It has been suggested that soil-suction and moisture content relationship be obtained at various values of initial dry density. The suction pressures deduced from the above relationship may be compared with the maximum swelling pressures measured by the direct method, to establish any possible correlation.

CHAPTER 1

INTRODUCTION

1.1. Statement of the Problem.

A particular characteristic of clays is the change in volume generated by the change in moisture content as shown in Fig. 1.1. The degree of volume change depends on various factors. One of these factors is the mineral composition. Presence of montmorillonite is considered to give a higher degree of volume change than the presence of either illite or kaolinite. If volume change is prevented by controlling the boundary conditions, swelling pressures are developed. These are the effects of the release of internal stress, associated with hydration and osmotic phenomenon. They may be represented by the loading which has to be applied to a soil to prevent swelling when in contact with water.

The characteristic of an expansive soil to undergo change in volume with change in moisture content causes great damage to structures. In regions which have well-defined, alternately wet and dry seasons susceptible soils swell and shrink in regular cycles. Beneath the centre of the building, where the soil is protected from rain and sun,

1. 2. 3.

FIG.1.1. SHRINKAGE OF EXPANSIVE CLAYS BY DRYING.

(1) BLACK COTTON SOIL (2) GREEN CLAY (3) SEPIOLITE CLAY.

the moisture content changes are small. Beneath the outside walls the movements are the greatest. This results in severe damage to the building. In arid regions where the soils are normally dry, the problem is somewhat different. Added moisture from leaking pipes and irrigation, or the reduction of evaporation caused by the presence of a building or a pavement can bring about appreciable swelling. When the source of moisture is eliminated, the movement will reverse, causing the same damage to structures as the seasonal volume changes. In humid regions where the soils are ordinarily moist, severe desiccation may cause susceptible soils to shrink and bring about severe settlement of structures. Associated with the volume changes are the swelling pressures which are developed. These are important because differential wetting means that the soil will differentially exert an upward pressure on the structure. Unless the structure is heavy enough to resist the swelling pressure developed, it will distort.

1.2. Scope of the Investigation.

The basic aim of the present research was to measure and evaluate the swelling pressures developed in a soil under controlled conditions of vertical and lateral confinement. Two methods were used to study the above problem.

1. Indirect Method : The tests were carried out in the standard consolidometer, with specially designed cells. A constant pressure was applied to the specimen during the test. The rate of swell of the soil and the maximum swell under various conditions of initial dry density, initial moisture content and load were studied. An attempt was made to deduce the balancing swelling pressure, i.e., the pressure at which the sample undergoes no volume change from the results obtained. Tests were also carried out to see whether undisturbed samples behave similar to disturbed samples under identical conditions.

2. Direct Method : These tests were carried out in an apparatus specially designed to find the absolute maximum swelling pressures developed in soils under complete confinement. The rate of build up of pressure and the maximum pressure developed in a soil at various initial dry densities and initial moisture contents were observed. An attempt was also made to study the influence of the height of the sample on the maximum pressure developed. The apparatus could also be conveniently used to find the pressure developed in a soil, after allowing it to expand vertically to a predetermined value.

The results obtained by the direct and indirect methods were compared. The various factors influencing the development of swelling pressure were examined and an attempt was made to predict the maximum swelling pressure developed in a soil under known conditions of placement and confinement.

1.3. Swelling Clays.

1.3a. Type of Clay Studied.

In the study the type of clays mainly considered are the heavy black clays. They occur over wide areas in the tropics, particularly in Africa and India, and are recognised by such local terms as "black cotton soil", "regur", "mbuga" and "adobe". They occur principally in regions with well defined wet and dry seasons, poorly drained topography, and in valley bottoms. The proportion of clay in the soils can vary a good deal from place to place, although it is usually very consistent with depth, and can be quite high. The black colour is attributed to the presence of a particular type of iron oxide and/or titanium and not due to excessive organic matter (maximum about 0.5%). They are chemically reactive, generally being acidic in nature, and they possess the important property of ion-exchange.

1.3b. Formation of Clays.

Clays are the result of chemical weathering of the parent rock. Igneous rock is one of the more common parent rocks. If a basic igneous rock, containing considerable magnesium, weathers under conditions which, because of poor drainage or low rainfall, permit the magnesium to remain in the weathering zone after it is released by the breakdown of the parent mineral, montmorillonite is formed. If, however, because of high rainfall and good drainage, the magnesium is removed as soon as it is released from the parent mineral, kaolinite will be the product of chemical weathering.

An acid igneous rock containing considerable quantities of potassium as well as magnesium, under weathering conditions permitting potassium and magnesium to remain in the weathering environment after the breakdown of the parent mineral, will yield, illite and montmorillonite as end products. If the content of magnesium is low, illite will be the only product, and if the content of potassium is low, montmorillonite will be the only product. Rapid removal of potassium and magnesium leads to the formation of kaolinite. It is thought that calcium favours the formation of montmorillonite, with a tendency to retard the formation of kaolinite.

Calcareous silty clays, which occur in Iran, Iraq, Jordon and on the coast of North Africa also show marked swelling and shrinkage following alternate wetting and drying. Hummocks known as "gilgai" are formed by swelling in the wet season and deep cracks formed during dry season become filled with debris soil.

1.4. Previous Work.

1.4a. Identification Tests to Classify Expansive Soils.

A number of index tests have been used to evaluate qualitatively the swelling potential of a soil. The simplest is the free-swell test (1)•, which gives a rough approximation of the total volume change. Holtz and Gibbs (1) have suggested that such indicators as liquid limit, plastic limit and percentage clay fraction give a better idea of the swelling potential of a soil when considered together.

• Numbers in parenthesis refer to the references listed in Bibliography.

Bolton Seed, Woodward and Lundgren (2) have suggested an equation in terms of activity and percentage clay fraction to predict quantitatively and qualitatively the swelling potential of a soil. The authors have also claimed that the plasticity index, when used as a single factor to predict the swelling potential, can give results to an accuracy of ± 35%.

Ranganatham and Satyanarayana (3) used the shrinkage index (defined as liquid limit minus shrinkage limit) as a single factor to predict the swelling potential. This index can give results to an accuracy of ± 34%. It is felt by the writer that the usefulness of the classification system proposed by the above mentioned authors could be increased by considering the influence of clay fraction on the swelling potential of an expansive soil. The use of shrinkage index and percentage clay fraction is suggested for this purpose.

Other indices such as shrinkage limit (4), linear shrinkage (4), heat of wetting (6), specific surface area (7) have been suggested by other investigators.

Tests such as microscopic examination, x-ray diffraction determination and differential thermal analysis are valuable in determining the presence of objectionable clay minerals.

1.4b. Study of Swelling Characteristics of Expansive Soils.

Holtz and Gibbs (1) studied the effects of moisture content and density on expansion. Swelling of a soil could be controlled by varying the initial moisture content and density. It was found that shear strength of soils decreases as the moisture contents are increased and the densities are decreased. The disturbed specimens were found to behave similar to the undisturbed specimens.

Parcher and Liu (8) studied the relationship between the horizontal and vertical swelling. The results showed that both the horizontal and vertical swelling vary inversely with the initial moisture content. They also observed that greater swelling in both the horizontal and vertical direction results when the compaction is increased, regardless of the initial moisture content. It was noted that in all cases the swelling in the radial (horizontal) direction is considerably greater than that in the vertical direction. The increase in volume of the specimens was always found to be smaller than the volume of water imbibed during swelling. Under similar conditions, compacted specimens swell more than do undisturbed specimens.

Bolton Seed et.al. (9) investigated the possibility of stabilizing clays by decreasing their swelling characteristics by means of appropriate blending of clay minerals to produce a soil with characteristics better than both of its constituents.

Jennings and Knight (10) have presented a method, based on the degree of desiccation, the activity of the soil, over-burden and applied loading, to predict total heave of structures from the results of the "Double Oedometer Test". de Wet (11) made an attempt to predict the time-heave relationship for structures founded on expansive clays.

Komornik and Zeitlen (12) devised a special apparatus to measure the lateral pressures developed during the saturation of expansive clay soil. Alpan (13) studied the pressures developed in a soil throughout a range of moisture variations below full saturation.

1.5. Factors affecting Swelling.

Some progress has been made in understanding the mechanism of swelling in soils. The degree of swelling depends on a variety of factors and these may be logically divided into two groups. In group (1) are those that depend on the nature of the soil particles. It is, in fact, these factors which determine in the first instance, whether the soil would have the capacity to swell under any conditions. The factors in group (2) are determined by the placement and the environmental conditions of the soil. They determine the extent to which the swelling potential of the grains comprising the soil may be realized.

CHAPTER 2

REVIEW OF PREVIOUS LITERATURE

2.1. Identification Tests to Classify Expansive Soils.

2.1a. Atterberg Limits and Clay Fraction.

Kantey and Brink (14) used Atterberg limit tests and Casagrande's plasticity chart (15) (i.e. a plot of the plasticity index against the liquid limit) as a guide for classifying expansive soils. Most of the expansive soils fell above the Casagrande 'A' - line in the plasticity chart, to the right of the 30% liquid limit line and above the 12% plasticity index line. The following criteria was suggested as a very rough guide for the recognition of expansive soils:

(i) linear shrinkage over 8%.

(ii) liquid limit over 30%.

(iii) plasticity index over 12%.

de Bruyn, Collins and Williams (7) attempted to predict the expansive potential of a soil from the relationship between activity and percentage clay fraction (see Fig. 2.2). Activity has been defined by Skempton (17) as the ratio of the plasticity index to percentage of the clay fraction (minus 2 micron size). The diagram shows some definite

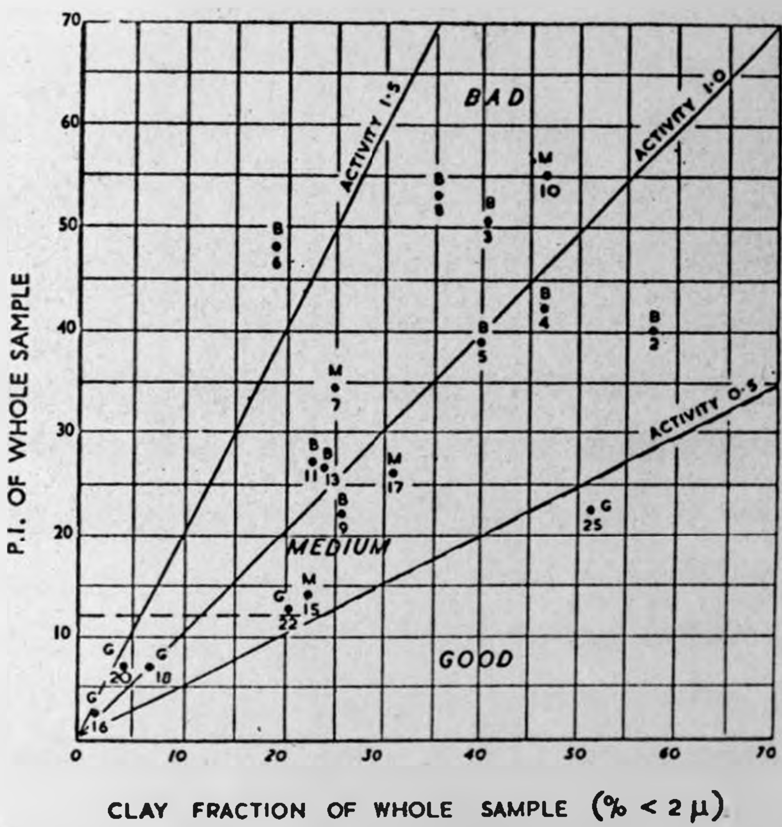

Fig. 2.2. –Activity diagram derived by plotting plasticity index against clay content. (B = bad, M = medium, G = good).

trend towards grouping of the bad, medium and good soils for structures, but more definite limits than those indicated are required to get a better idea of the expansiveness of a soil.

One of the most extensive studies to identify expansive soils was undertaken by the Bureau of Reclamation, United States Department of the Interior (USBR) and the results have been discussed by Holtz and Gibbs (1). Emphasis was laid mainly on those identifying tests which are simple and which may be performed in a normal soils laboratory.

The simplest of these is the free-swell test. The test is performed by slowly pouring 10 c.c. of dry soil passing the No. 40 sieve into a 100 c.c. graduated cylinder filled with water and noting the swelled volume of the soil after it comes to rest at the bottom. The free-swell value in percent is obtained by the expression,

$$\frac{(\text{final volume minus initial volume})}{\text{initial volume}} \times 100$$

Soils having free-swell values as low as 100 percent may exhibit considerable volume change when wetted under light loadings and should be viewed with caution, while soils having free-swell values below 50 percent very seldom exhibit appreciable volume change, even under very light loadings. A good grade of high-swelling commercial bentonite will have a free-swell value of 1,200 to 2,000 percent. Close correlation could not be obtained between free swell values and actual total volume change of undisturbed soil specimens, but the graph (Fig. 2.3) does show the general trends. The free-swell is a useful supplementary test.

Two undisturbed samples were taken by Holtz and Gibbs to find the total volume change for the specimen. After determining their initial volume, one of the samples was allowed to shrink to the shrinkage limit. The final volume of the sample was determined by the mercury displacement method. The volume change of the sample (V_1) from the natural moisture content to the shrinkage limit was determined (a three-dimensional phenomenon). The other sample was placed in a consolidometer ring under a vertical load of 1-psi and the maximum vertical expansion of sample noted (a one-dimensional phenomenon). The volume change of the specimen (V_2) from the natural moisture content to full saturation was determined. The total volume change was determined as V_1+V_2.

It may be difficult to correlate the above two phenomena to give accurately the total volume change.

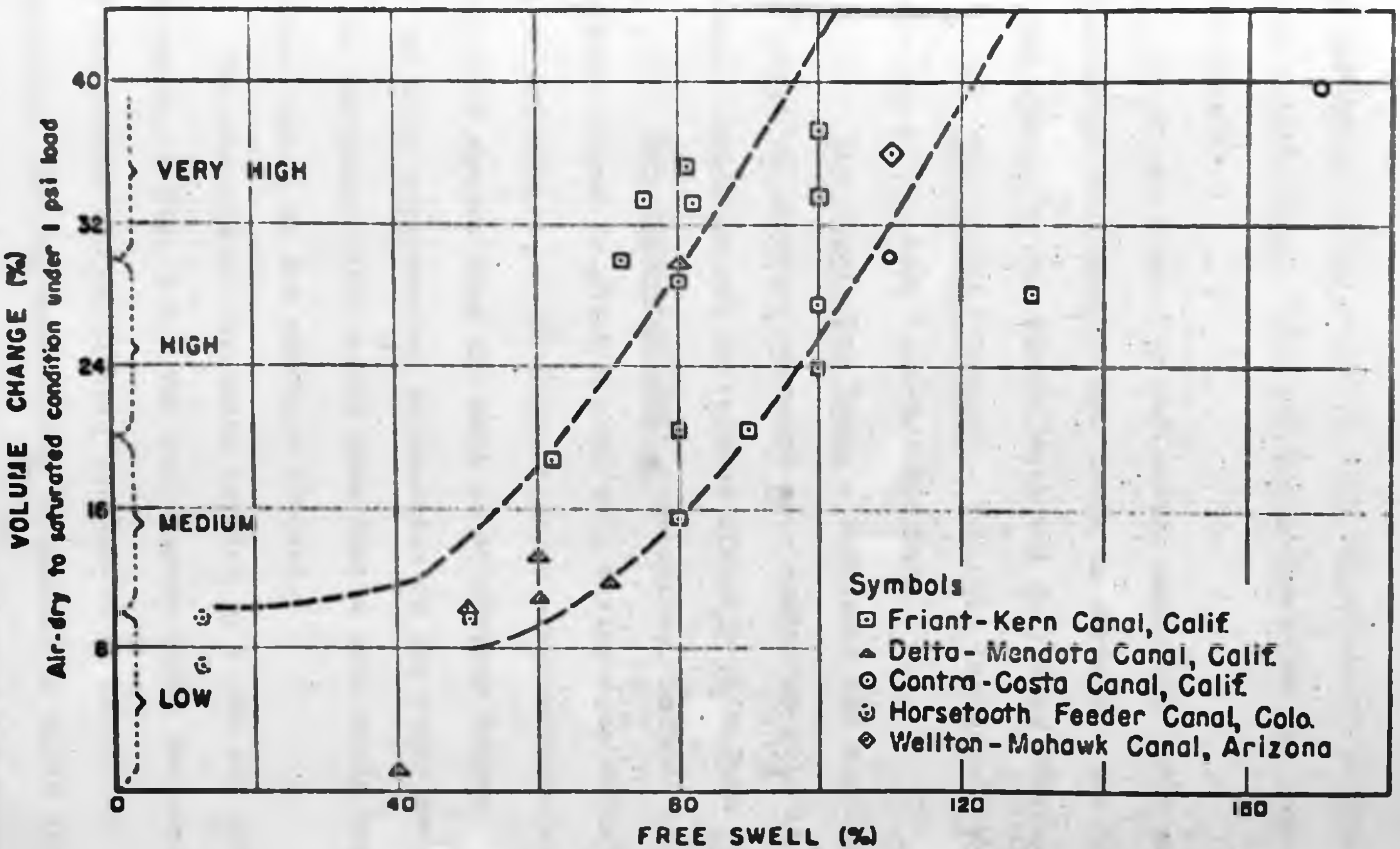

Figure - FREE SWELL VERSUS VOLUME CHANGE

The shrinkage curve could be used conveniently to find the total volume change both for the undisturbed and remolded soil samples.

Holtz and Gibbs (1) did actual expansion tests on 38 undisturbed soil samples and tried to correlate the total volume change of the sample with the following indices:-

(a) The Colloid Content - the soil ingredient which contributes the most towards expansion.

(b) The Plasticity Index - indicates the magnitude of the range of moisture contents over which the soil is in a plastic condition and is related directly to volume change.

(c) The Shrinkage Limit - describes, indirectly, the minimum volume to which a soil will shrink upon drying and is an expression of the percentage of water necessary to fill void spaces when the soil is at minimum volume. It is valuable supplemental information to the other two data. A low shrinkage limit would show that a soil could begin volume change at low moisture content.

The results of the tests carried by Holtz and Gibbs are shown in Fig. 2.4. The thin dashed lines enclose most of the points and still keep the limits as narrow as possible. The recommendations formulated by Holtz and Gibbs for predicting the expansion characteristics are reproduced

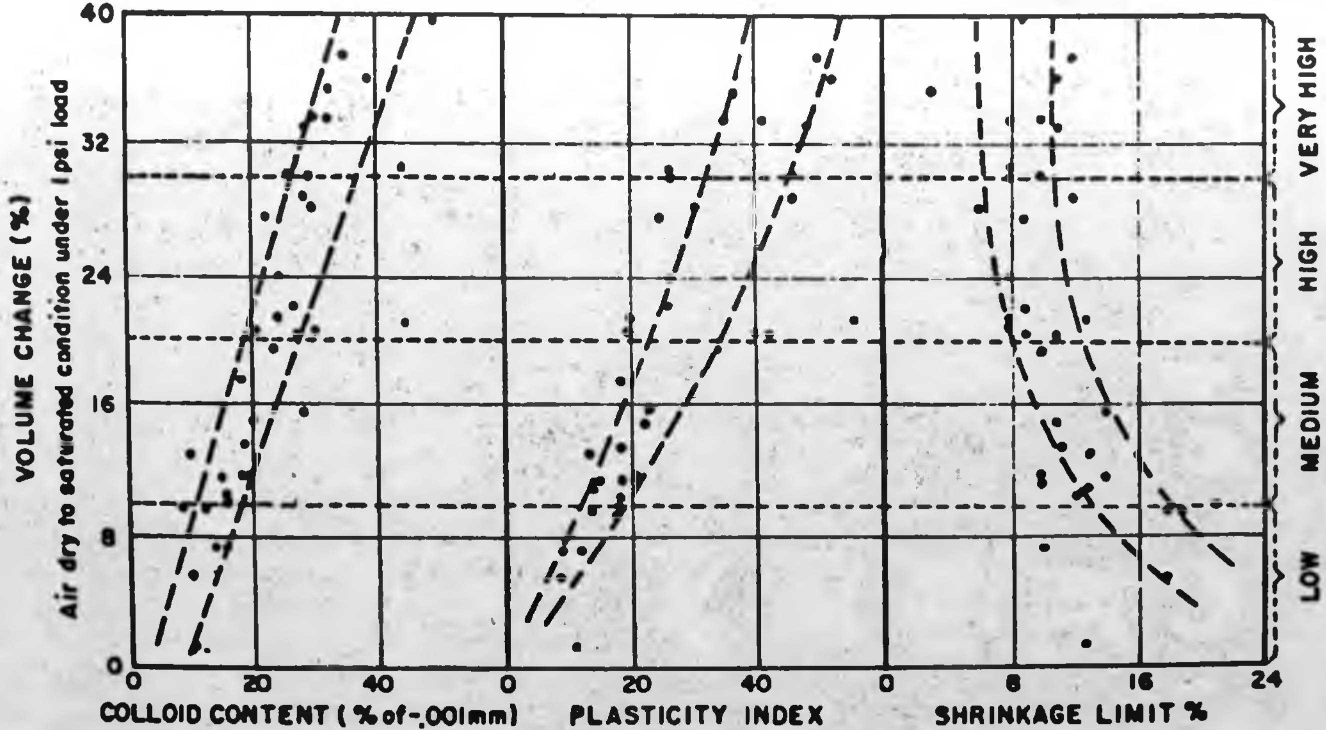

FIG. 24. —RELATION OF VOLUME CHANGE TO COLLOID CONTENT, PLASTICITY INDEX, AND SHRINKAGE LIMIT

in Table 2.1.

TABLE 2.1.

Data for Making Estimates of Probable Volume Changes for Expansive Soils (after, Holtz and Gibbs)

Data From Index Tests			Estimation of probable Expansion (% Total Volume change dry to saturated condition)	Indication of the Degree of Expansion
Colloid Content (% minus 0.001mm)	Plasticity Index	Shrinkage Limit (%)		
>27	>32	<10	>30	Very High
18-27	23-45	6-12	20-30	High
12-27	12-34	8-18	10-20	Medium
<17	<20	>13	<10	Low

Better and more consistent results would have been possible if the authors had carried out tests on compacted specimens, in which density and moisture conditions could be controlled, than on undisturbed samples, with a variety of density conditions.

It must be emphasised that all the indicators such as colloidal content, plasticity index and shrinkage limit must be considered together and not individually in assessing the swelling characteristics of any soil. A soil, for example, with high colloidal clay content, but of low plasticity index and high shrinkage limit values should not be judged as having a compromised value, but be judged as a soil with highly concentrated clay content that has little swelling and shrinkage potential. However, a soil containing active swelling clay may not be of serious consequence if colloidal content is not of high concentration. Used in this manner, the correlations provide a helpful approach to predict the approximate expansion characteristics of a soil.

Bolton Seed et al (2) made an attempt to test the general validity of the results obtained by Holtz and Gibbs (1) with the aid of more data. Detailed research was carried out to determine the applicability of predicting the swelling characteristics of clays by the use of clay content, plasticity index and shrinkage limit. However, a slightly different approach was adopted by the authors from that of Holtz and Gibbs. The swelling potential of a soil was measured under standard conditions of placement and test. The swelling potentials of different soils were compared by determining the amount of vertical expansion that each soil would exhibit if it were soaked under a 1-psi surcharge after being compacted to maximum density at optimum water content in the standard AASHO compaction test. The procedure adopted was similar to the standard CBR soaked test. The value of swell at maximum density and optimum moisture content was chosen to represent the swelling potential.

The initial series of tests conducted by Bolton Seed et.al. were on samples artificially prepared in the laboratory to minimise variations in test data due to variability of soil sample and to facilitate a representative study of a large number of soils. Twenty three artificial soils prepared in the laboratory were tested to determine liquid limit, plastic limit, shrinkage limit, grain-size distribution and their swelling potential. Artificial soils were prepared by mixing commercially available clay minerals (Wyoming bentonites, illite and kaolinite) individually and in the combinations, and each of these clays was also mixed with varying proportions of fine sand. The typical results of one of the tests are shown in Fig. 2. 5.

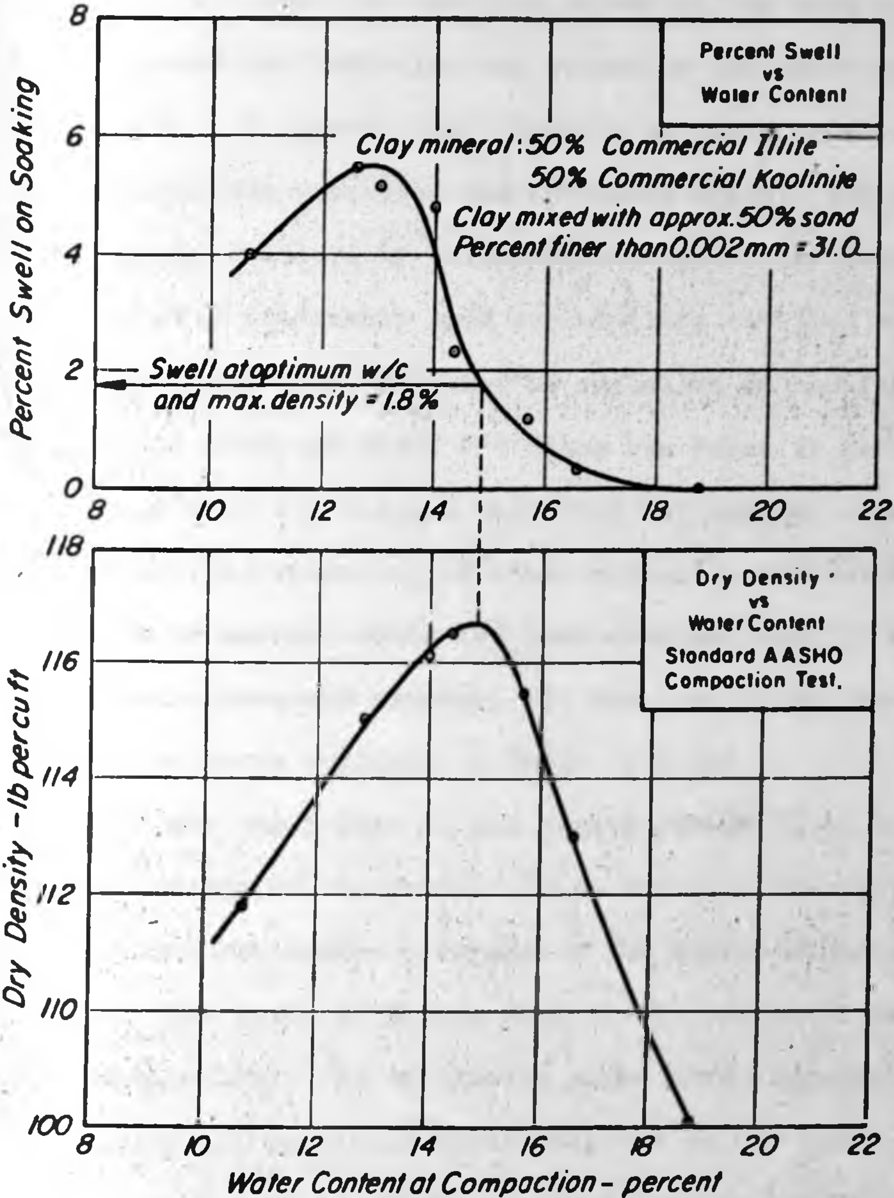

FIG.25.—SWELLING TEST DATA FOR COMPACTED SAMPLES

Fig. 2.6 shows the swelling potential for each of the soils against the corresponding values of the above proposed indicators. It appears that there is no close correlation between swelling potential and shrinkage limit. The poor correlations obtained by Bolton Seed et.al. (Fig. 2.6) when compared with reasonably good correlations obtained by Holtz and Gibbs (Fig. 2.4) may be explained in the following terms. The study of Holtz and Gibbs was based on natural clay soils from the Central Valley of California. The soils were classified according to their expansion properties: (1) in their natural state (2) when remolded and (3) after drying and subsequent wetting. It has been found that the proposed criteria outlined in Table 2.1.and Fig. 2.4. are applicable to all plastic clays (i.e. SC, CL or CH groups of the Unified Classification System) which are found in the western regions of the United States of America. The study of Bolton Seed et.al. was based on artificial soils. The artificial soils used contained practically all particles finer than the No. 40 sieve, and thus the soil used to establish the Atterberg limits represented all the material.

It seems unlikely that shrinkage limit by itself can be considered as a reliable index of volume changes due to

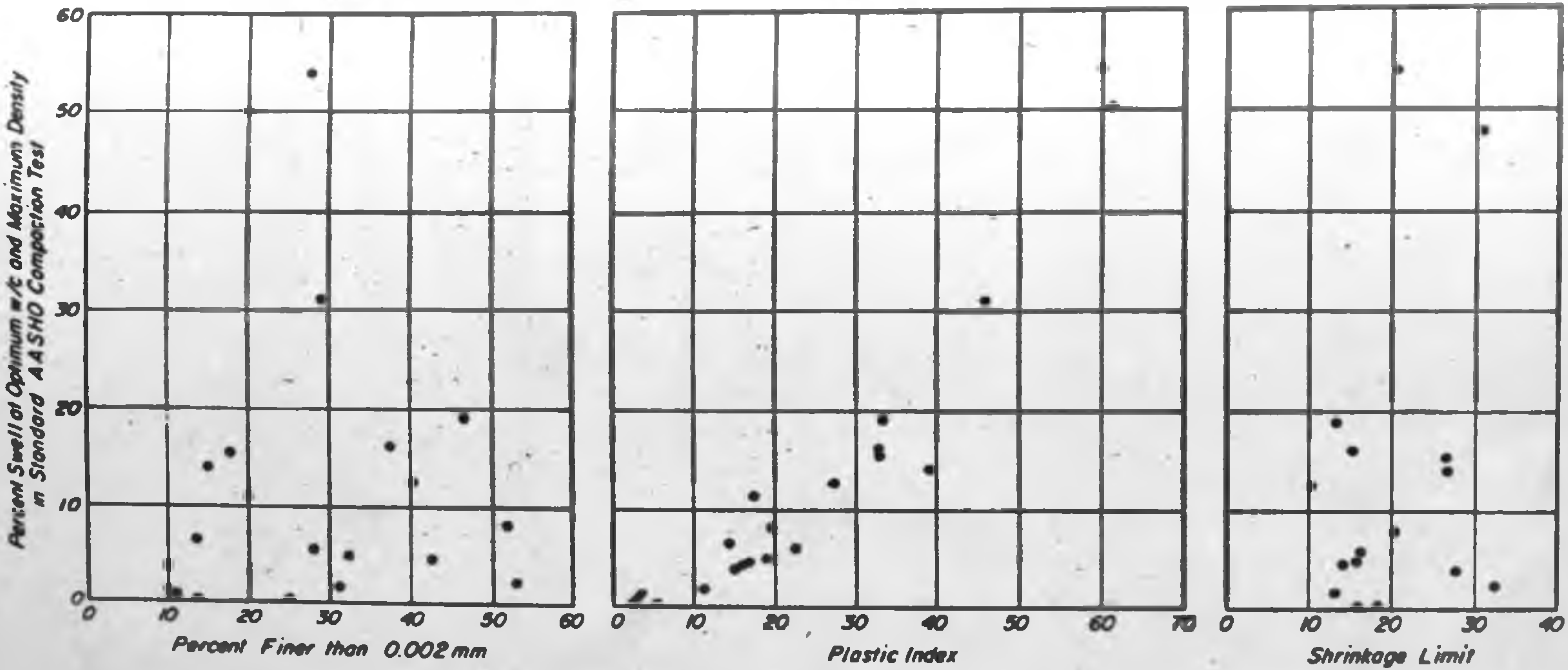

FIG. 26.—RELATION OF SWELL POTENTIAL TO PERCENTAGE OF CLAY SIZES, PLASTICITY INDEX, AND SHRINKAGE LIMIT FOR EXPERIMENTAL SOILS

the following considerations:-

(1) The usual range of values of this soil parameter for soils varying from low to very high swelling potential is approximately 8 to 16; yet the accuracy with which the shrinkage limit is measured by the mercury displacement method is approximately $\pm$ 3. The method of coating the oven-dried specimen with wax, and taking the volume as the numerical difference between the weight in air and submerged in water may be more reliable than the mercury displacement method as in this method some air will often be entrapped in the top of the glass disc and it is compressed. Also an optical projection method developed at the Road Research Laboratory has been found to be more accurate and convenient.

(2) When the shrinkage limit is used as an index of swelling potential, it is usually considered that higher values (greater than approximately 14%) are indicative of non-swelling soils. Altmeyer (4) has suggested the following, somewhat oversimplified limits:-

(a)	Shrinkage Limit (%)	<10	10-12	>12
	Degree of Volume Change	Critical	Marginal	Non-critical
(b)	Linear Shrinkage (%)	>8	5-8	<5
	Degree of Volume Change	Critical	Marginal	Non-critical

Yet, it is found that soils containing the high proportions of montmorillonite, indicative of high expansion characteristics, can have values much higher than 14% (See Table 5.2). The shrinkage limit for a wide range of English soils is nearly constant at about 12% and thus not directly dependent on the value of clay content or liquid limit. In contrast, in Texas, soils may be divided into two types - those having a shrinkage limit of about 11% and those whose shrinkage limit is about 30%. It is known that some active clays have shrinkage limits as low as 9% and some non-active clay soils have shrinkage limit as high as 28% to 36%. Mica may have a shrinkage limit of as high as 103%.

The writer, however, feels that the shrinkage limit is valuable as an additional index for estimating the total volume change when both drying and wetting must be anticipated. The degree of swelling of compacted clays that are placed at optimum moisture contents is related to the combination of plasticity index and clay content, and not related to the manner in which the soil might shrink on drying to the shrinkage limit. But the total volume change that the soil can undergo from dry to saturated conditions expressed as the swelling potential of the soil is related to the shrinkage limit and the plasticity index. In natural soils, it is not practical to evaluate the swelling potential on the basis of only increased moisture from an intermediate condition, such as optimum moisture content, which lies between the shrinkage limit and the liquid limit. In undisturbed state the soils are often much drier than optimum moisture or the plastic limit moisture. Embankments are constructed near optimum moisture conditions, but significant drying usually occurs before pavements or other structures are built on them.

The results obtained by Bolton Seed et.al. (2) showed that there may be some sort of relationship between swelling potential and plasticity index (Fig. 2.6.). The correlation between the indices was examined in detail. The following equation was deduced from the experimental results,

$$S = KC^{x} = KC^{3.44} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where,

S is the swelling potential, i.e. percentage of vertical swell under 1-psi surcharge for sample compacted to maximum dry density at the optimum water content in the standard AASHO compaction test;

C is the percentage of clay sizes finer than 0.002 mm;

x is the exponent depending on the type of clay and is found to be 3.44;

K is the characteristic index of the swelling potential of any given clay type and it could be evaluated from the following relationship,

$$K = kA^{y} = (3.6 \times 10)\ (A^{2.44}) \ \ldots\ldots\ldots\ldots\ldots (2)$$

where,

k defines the value of K when A = 1; and

y is the exponent defining the slope of the line represented by equation (2).

A is the activity defined by the authors as

$$\text{Activity, A,} = \frac{\text{Change in Plasticity Index}}{\text{Corresponding change in clay content}} = \frac{\Delta PI}{\Delta C} \ \ldots (3)$$

The above definition of activity has been modified by the authors from the original definition proposed by Skempton (17) and defined as,

$$\text{Activity} = \frac{\text{Plasticity Index}}{\text{Percentage of clay sizes}} = \frac{PI}{C} \ \ldots\ldots\ldots\ldots (4)$$

FROM EQUATIONS (1) and (2),

$$S = (3.6 \times 10^{-5})\ (A^{2.44})\ (C^{3.44}) \ \ldots\ldots\ldots\ldots (5)$$

Because the clays used in the tests covered a wide range of clay minerals, it seems reasonable to expect that the relationship would apply to all clay types with a satisfactory degree of accuracy. In trying to correlate these results with the results obtained by Holtz and Gibbs (1), the authors arrived at the following deductions based on equation (5).

Degree of Expansion	Swelling Potential (%)
Low	0 to 1.5
Medium	1.5 to 5
High	5 to 25
Very High	>25

On superimposing the actual test data obtained for the soils tested on the curve representing equation (5), the authors found that there is excellent agreement between the test data and the proposed boundaries.

The authors also tried to test the validity of this method by analysing the swelling test data obtained by the United States Bureau of Reclamation (USBR) (1), inspite of the fact that USBR determinations were carried out under different conditions of placement. For purposes of analysis, the authors adopted essentially the same subdivisions as proposed by USBR which are listed below.

Degree of Expansion	Total expansion, air-dry to saturated conditions.(%)
Low	0-10
Medium	10-20
High	20-35
Very High	>35

The authors also deduced the following approximate correlation between the swelling potential values (used to fix the boundaries) and the total expansion from air-dry to saturated condition used as an index of expansion by the USBR:

Swelling Potential (%)	Total Expansion (%)
1.5	10
5	20
25	35

According to the authors, the above correlation appears to offer greater possibilities for correctly classifying the degree of expansion of undisturbed soils than any of the individual indicators previously proposed by various investigators for this purpose.

It seems to the writer that in trying to establish the above correlation, the authors assumed that the maximum swelling potential of highly expansive artificial soil tested by them is close to the maximum swelling potential of highly expansive undisturbed soil tested by Holtz and Gibbs. This may not be true.

The use of plasticity index as a single factor for predicting the swelling potential was examined by Bolton Seed et.al. and the following approximate formula for natural soils was proposed:

$$S = (K)\ (60)\ (PI^{2.44}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

The authors claim that this relationship is accurate to within ±35% for soils with clay contents approximately in the range of 8% to 65%. Finally the authors conclude that in view of the difficulty in accurately determining the percentage of clay sizes present in a soil, and the approximate nature of the relationship between the percentage of clay sizes and the actual amount of clay in the soil, it appears that the use of the plasticity index as an indication of swelling potential would be sufficiently accurate for all practical purposes. This conclusion definitely requires further examination by analysis and experimentation. Some values of the swelling potential of Black Cotton soil were calculated by Ranganatham and

Satyanarayana (3) with the aid of the empirical methods suggested by Bolton Seed et.al. and the results compared with the experimental values. The error of 30% to 65% was noticed.

It is also well-worth noting that when a particular method is used in the preparation and testing of a limited series of soils there appears to be good correlation between the results. When this knowledge is to be applied to other soils of entirely different origins and characteri--stics, it becomes essential that the method used should be selected to yield the nearest approach to the true natural conditions existing at the time. Unless this is attained, the results obtained may not be satisfactory.

The work of Bolton Seed et.al. (2) showed that the linear relationship between plasticity index and percentage clay sizes does not pass through the origin, but makes a positive intercept on the percentage clay axis. This is contrary to the data obtained by Skempton (17) for natural clays. According to Skempton,the reason that these lines converge towards the origin could be that for the type of clay tested, zero clay content (less than 2 micron) gave rise to zero plasticity index. It is more likely that the linear relationship should make a positive intercept with the plasticity index as it is probable that soil particles between No. 40 sieve size and 2 microns may have some tendency to develop plasticity. If the line makes a negative intercept with the plasticity index, it means that the strength of the soil has increased with the addition of water. This seems very unlikely. The strength of some soils increases with the increase in moisture content. This is due to the thixotropic characteristics of the soils. This phenomenon, however, occurs well above the plastic limit.

The effect of non-clay mineral part of the less than 2 micron fraction of a soil on activity was quantitatively evaluated by Esrig (26) from the results of a series of tests performed on mixtures of commercially available illitic and montmorillonitic soils mixed with silt manufactured from a relatively pure quartz sand. In preparing samples for testing, only the fraction of the illitic and montmorillonitic material passing the No. 200 sieve was mixed with the silt. It was found that the intercept of the lines of the graph plasticity index versus clay fraction is strongly affected by the presence of non-clay particles. If all the non-clay particles were excluded from the analysis, it would be expected that the line would intercept the percentage of clay axis at, or near the origin.

Ranganatham and Satyanarayana (3) have recently suggested the use of "Shrinkage Index" to assess the potential expansiveness of soils. The shrinkage index defines the range of moisture content between the liquid limit and shrinkage limit. This seems to be more appropriate for the prediction of volume change characteristics. For all the artificially prepared soils tested by Bolton Seed et. al. (2), the authors deduced the following relationships,

$$SP = (4.57 \times 10^{-5}) \times (SA)^{2.67} \times (C^{3.44}) \quad \ldots\ldots\ldots (7)$$

where,

SP is the swelling potential, defined earlier on page 32.

C is the percentage clay fraction, -.002 mm. particle size.

SA = Swell Activity

$$= \frac{\text{Change in Shrinkage index}}{\text{Corresponding change in clay fraction}} = \frac{\Delta(SI)}{\Delta C}$$

The above relationship (Eq.7) has been expressed in terms of shrinkage index as follows,

$$SP = (263 \times 10^{-5})\ (SI)^{2.67} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where,

SI is the shrinkage index.

From Equation (7), the expansion characteristics of a soil can be quantitatively evaluated in terms of clay fraction and the swell activity. The authors claim that Equation (8) could be used to predict the swelling potential to an accuracy of $\pm 22\%$ for artificial soils, and $\pm 34\%$ for natural soils and have suggested the classification system shown in Table 2.2.

Table 2.2

Classification of Expansive Soils

Classification	Swelling Potential (%) (after Bolton Seed et. al.)	Shrinkage Index (%)
Low	0-1.5	0-5
Medium	1.5-5	5-15
High	5-25	15-35
Very High	>25	>35

It is felt by the writer that the usefulness of the above classification system could be increased by considering the influence of clay fraction on the swelling potential of an expansive soil. A graph of shrinkage index against the clay fraction, from the results obtained by Holtz and Gibbs (1) has been drawn, and the boundaries indicating the range of low to very high swelling potential soils have been defined in Fig. 2.9. On the basis of these boundaries, the suggested classification is outlined in Table 2.3.

Table 2.3

Suggested Classification of Expansive Soils.

Classification	Shrinkage Index (%)	Percentage clay fraction* (minus .001 m.m. size)
Low	16	23
Medium-low to Medium	16-44	12-38
Medium-High to High	32-57	25-49
High-Very High to Very High	43	35

*Percentage clay fraction in this case represents -.001m.m. particle size, according to Holtz and Gibbs.

Super-imposing the data obtained by Bolton Seed et al (2)

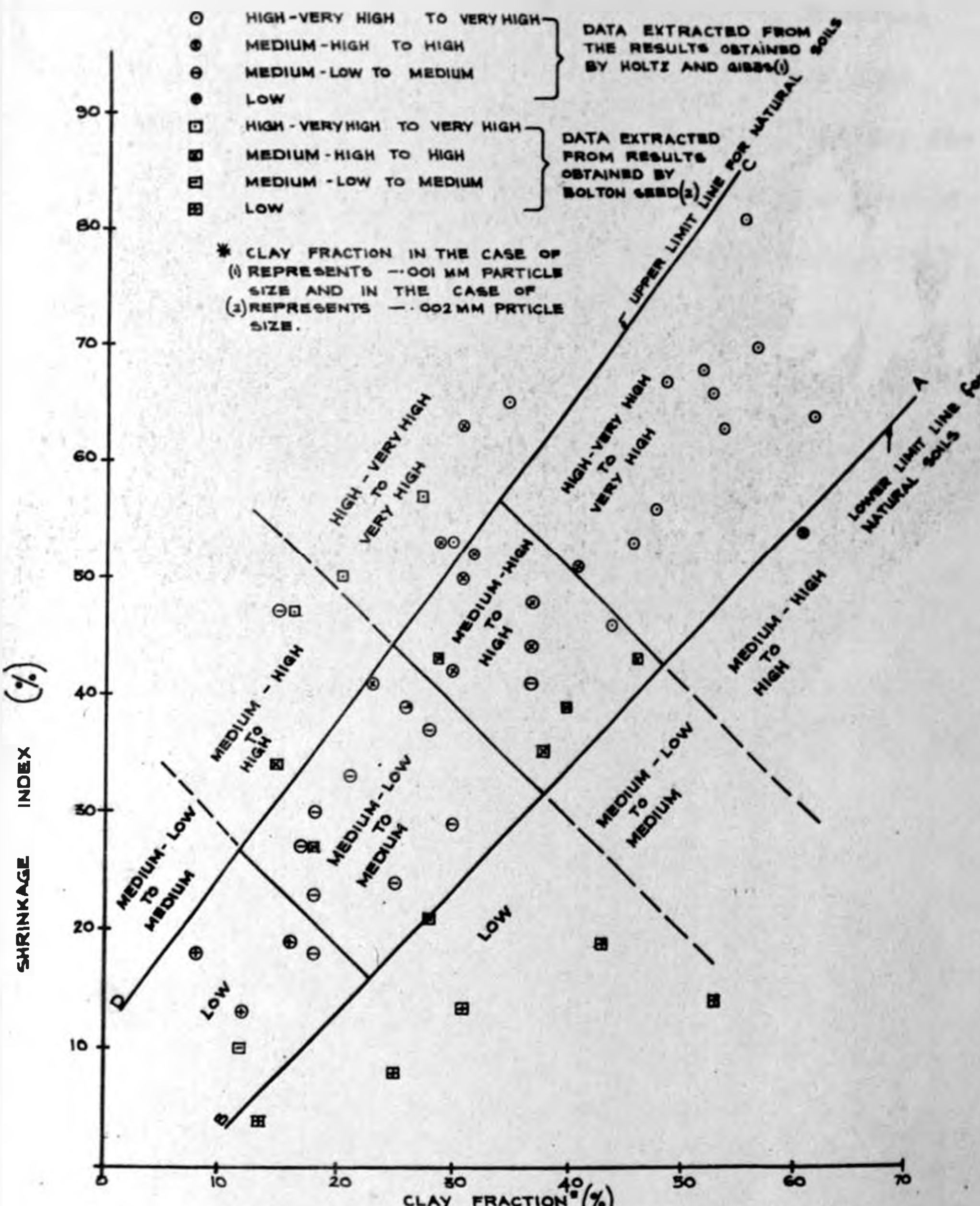

FIG. 2·9. CLAY FRACTION* (%) v. SHRINKAGE INDEX (%)

for artificial soils, in which case the clay fraction represents -.002 m.m. particle size, it is seen that those soils lying inside the envelope fairly satisfy the subdivisions on the graph. But a number of these soils fall outside the envelope. For the classification of these soils the following procedure is suggested:-

(1) If the soil falls above the upper limit line DC and it falls inside the extended range, say, medium-high to high, then the soil may be classified as high-very high to very high and so on.

(2) If the soil falls below the lower limit line AB and it falls inside the extended range, say, medium-high to high, then the soil may be classified as medium-low to medium.

It has been established by various investigators that the determination of the Atterberg limits and the percentage clay fraction are subject to the influence of various factors. Differences in the preparation of the samples and the degree of working cause considerable differences in the values of Atterberg limits in case of most of the soils. Kersten and Krieger (24) found that the liquid limit increases with gentle manipulation whereas the plastic limit appears not to be affected significantly. Thornburn (20) found that the liquid limits of sodium-and lithium-saturated montmorillonite samples are very high and the determination is difficult because of the high degree of thixotrophy of such materials. Moreover, the gelation or development of thixotrophy takes an appreciable time. So the liquid limit may change with increasing time following the preparation of sample. A similar difference was also experienced by the writer and is shown in Table 5.2.

The determination of clay fraction by the hydrometer test in accordance with the British Standard 1377, 1961, (22) involves hydrogen peroxide treatment prior to the addition of the dispersing agent. As opposed to this, the determination of clay fraction in accordance with SDTM D-422 (50) requires no treatment of the soil. The difference arising from these two procedures may contribute to the differences in the values of activity.

Non-colloidal minerals may be of importance in the behaviour of swelling soils and among these calcium carbonate is the most important. Soils in desiccated areas are often rich in calsium carbonate. It has been experienced by Zolkov (27) in case of a soil having calcium carbonate content of 50% that most deviations from generally anticipated behaviour based on the usual identification tests derive from the presence of calcium carbonate. It is, therefore, recommended that the calcium carbonate content of the soil should be determined. Unfortunately, there is, at present little data available to show the precise effect of organo-mineral soils when compared to inorganic soils of the same mineral composition.

Despite the errors caused by the various factors, the indices such as the Atterberg limits and the percentage clay fraction provide a valuable guide for assessment of the swelling potential of soils.

2.1b. Specific Surface.

The ability of a soil to absorb water depends upon, among other things, the specific surface which varies with particle size, shape and gradation. The specific surface is defined as the ratio of surface area in square meters to the weight of matter in grams. A relatively fine, well-graded material will normally have much greater absorptive power than a coarse-grained uniform material. Table 2.4 shows the dimension of typical clay platelets. It is noticed that the very high specific surface of montmorillonite is indicative of its high swelling characteristics. The amount of free swelling of various clay minerals is shown in Table 2.5.

Table 2.4

Dimensions of typical clay platelets (28)

Characteristic + ratios of Dimensions		Approximate range of actual Dimensions in Angstroms		Specific surface (sq.m/gm)
		Length & Breadth	Thickness	
Montmorillonite	100 x 100 x 1	1000 to 5000	10 to 50	800
Illite	20 x 20 x 1	1000 to 5000	50 to 500	80
Kaolinite	10 x 10 x 1	1000 to 5000	100 to 1000	10

Figs. 2.7 and 2.8 (30) show the amount and rate of absorption of water for a series of clay minerals in which the samples were composed of air-dried uncompacted material of minus 200-mesh particle size. The data show the very

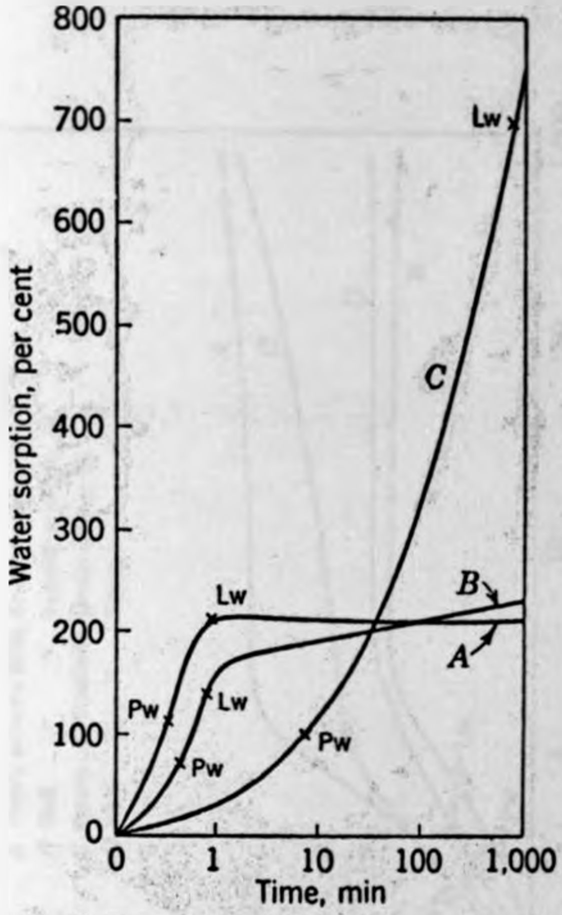

Fig. 2.7. **Water-sorption curves for attapulgite (*A*), calcium montmorillonite (*B*), and sodium montmorillonite (*C*), after White and Pichler (1959).**

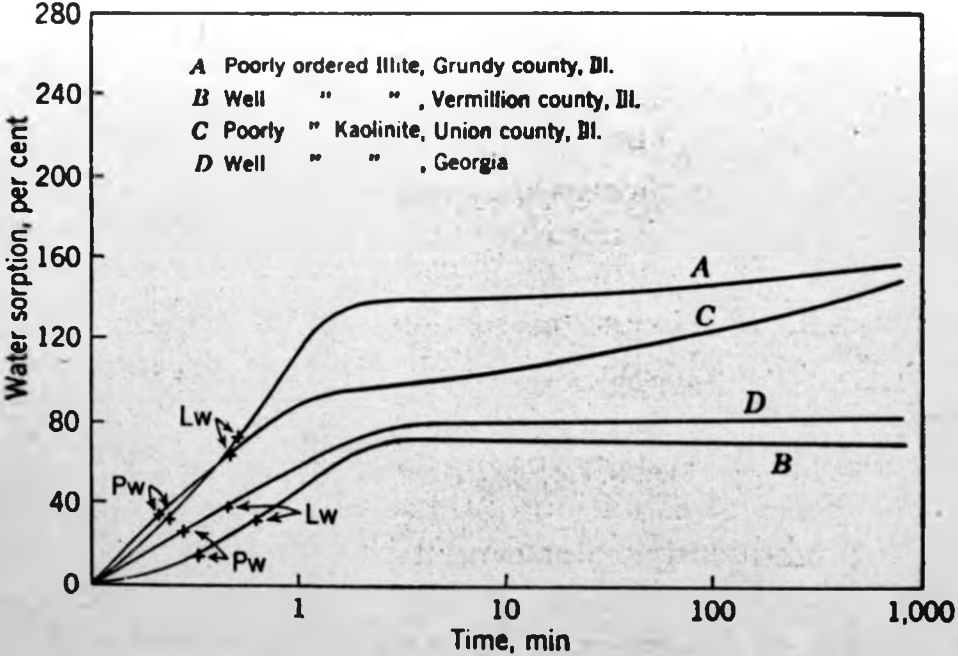

Fig. 2.8. Water-sorption curves for kaolinites and illites, after White and Pichler (1959).

high water adsorption of sodium-montmorillonite and the amount of water adsorbed by other samples increases in the order of attapulgite, and calcium-montmorillonite. Initial slow adsorption of sodium montmorillonite is probably due to the low permeability of the material. It is seen that for the clay minerals water adsorption is very rapid in the first few minutes with slow, if any, adsorption with greater lengths of time. The rate is relatively less for the clay minerals with the lower amounts of water adsorption.

Table 2.5

Free-swelling data for clay minerals (in percent)

(After Mielenz and King (31))

Sodium-Montmorillonite	1,400 - 2,000
Calcium-Montmorillonite	45 - 145
Halloysite	70
Illite	15 - 120
Kaolinite	5 - 60

de Bruyn, Collins, and Williams (7) made an attempt to predict the swelling potential of a soil from the determination of its specific surface. The authors arrived at the following tentative conclusions. Soils with total specific surfaces of less than 70 m^2/g and equilibrium moisture contents (at 85% humidity) of less than 3% may be classified as good (non-expansive) and those with total specific surface areas of more than 300 m^2/g and equilibrium moisture contents (at 85% humidity) of more than 10% as bad, the intermediate group being classified as medium.

2.1c. Heat of Wetting.

As a tentative index test, Rolf C. Vold(6) made determinations of heat of wetting for several soils and tried to correlate the values obtained with the total volume change. No close correlations could be obtained between the total volume change of the soil and the heat of wetting. The results obtained are not surprising due to the fact that a major part of the heat evolved is caused by the first film of water molecules coating the soil particles, whereas the volume change of a soil occurs only after the shrinkage limit. The heating of the soil to about 110°C brings about a structural change of the particles. As is expected, the heat of wetting decreases as the moisture content of a soil increases. It is seen that at a fairly low moisture content the value of the heat of wetting is relatively high.

Heat of wetting of various minerals are as shown in Table 2.6 and values of other minerals probably lie between them. The table shows that the heat of wetting for calcium montmorillonite is greater than that of sodium montmorillonite, although the swelling potential of sodium montmorillonite is greater than that of calcium montmorillonite. Therefore, it seems that the use of heat of wetting to predict the swelling potential, even qualitatively, is not promising.

Table 2.6

Heat of Wetting (28)

(Cal/Gram)

Kaolinite				Montmorillonite			
Ca^{++}	H^{+}	Na^{+}	K^{+}	Ca^{++}	H^{+}	Na^{+}	K^{+}
1.45	1.40	1.30	1.22	22.1	20.1	12.1	9.9

2.2. Study of Swelling Characteristics of Expansive Soils.

Holtz and Gibbs (1) carried out some load-expansion tests in which sufficient load was applied through out the test to prevent vertical movement and the swelling pressures developed in a soil determined. It is, however, not clear how consolidation and creep effects were compensated. The effects of various moisture and density conditions on expansion were studied (Figs 2.10 and 2.11). Such a plot indicates that combination of placement densities which are lower than those obtained by standard compaction and placement moisture near or higher than the standard optimum are required to ensure low amounts of expansion. It is evident that the reduction of density alone will not always provide the desired reduction in expansion control, especially in the higher expansion ranges. It is, however, not usually possible to place these soils at high moisture contents because of construction difficulties. The best solution under the circumstances is a combination of moderately high moisture and low density control. Figs. 2.10 and 2.11 show that considerable advantage is gained by reducing density without necessarily increasing the moisture content. It was also found that the remolded clays behaved in a similar manner as the

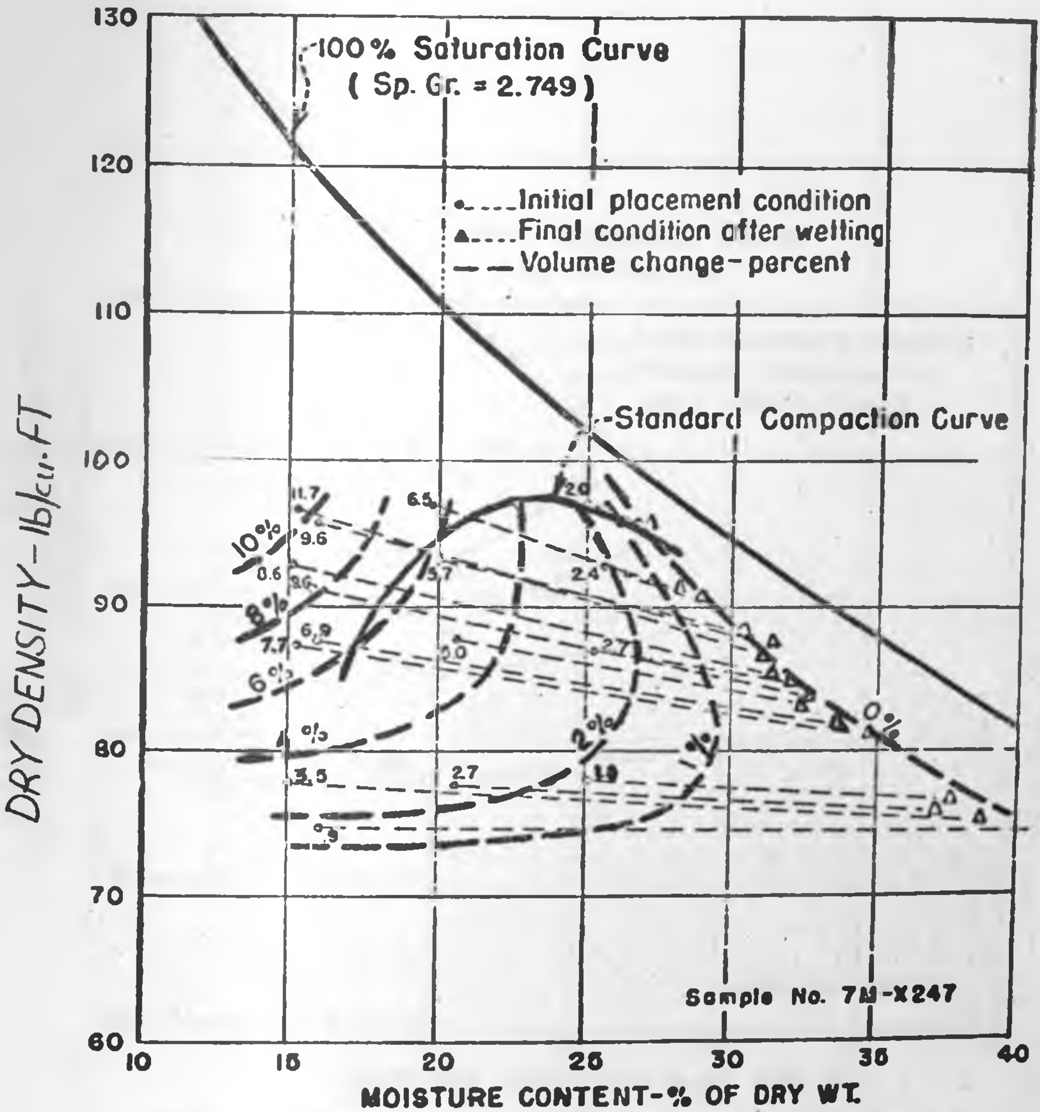

Figure 2.10 PERCENTAGE OF EXPANSION FOR VARIOUS PLACEMENT CONDITIONS WHEN UNDER 1-PSI LOAD

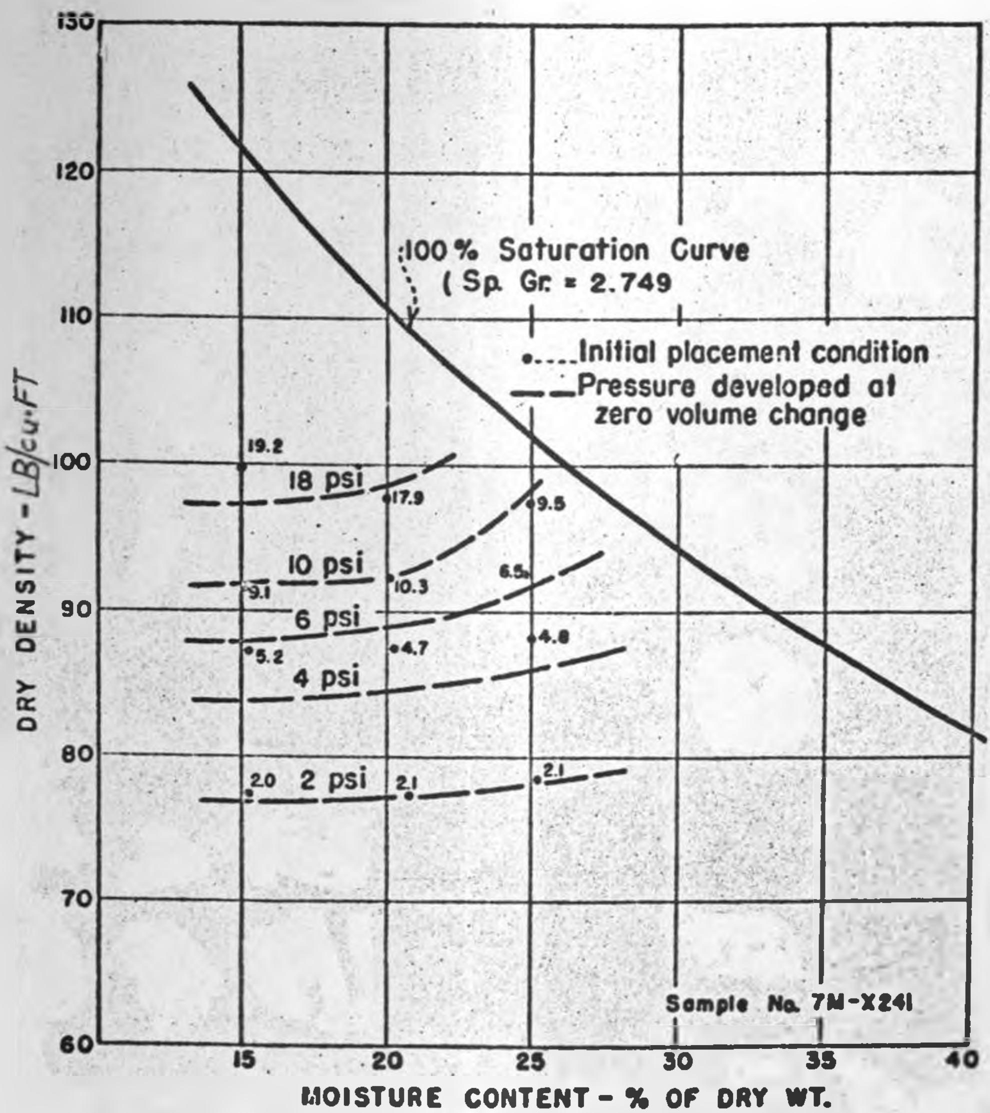

Figure 2 .11.TOTAL UPLIFT PRESSURE CAUSED BY WETTING—FOR VARIOUS PLACEMENT CONDITIONS undisturbed clays. A specimen which was loaded to a very light load e.g. 1-psi load, expanded considerably more than an identical specimen which was first loaded sufficiently to prevent expansion and then allowed to expand by reducing the load in equal decrements.

In addition to the difficulties which can be encountered from the uplift of expansive soils when saturated, the loss of stability or shear strength is an important factor to be considered. In general, the shear strength become less as the moisture contents are increased and the densities are decreased when the soils expand through saturation. This loss of strength may be great.

Seed et. al. (2) presented data to show that when bentonite is mixed with illite, the high activity of the bentonite particles tends to be masked out, or significantly reduced. But when it is mixed with kaolinite, it has a very large influence on the over-all activity of kaolinite/bentonite mixtures. The above result has been attributed by the authors to the cementation of clay size particles (decreasing their effective surface area), and to the interstratification of montmorillonoid and illite particles to form interstratified or compound particles with illite particles. Some tests carried out by the authors "for clay mineralogical stabilization" substantiate the above hypothesis.

The above result is of great practical significance. High degree of expansion of many soils is often attributed to the fact that they contain a high proportion of montmorillonite or bentonite. As the activity and plasticity index reflect significantly soil characteristics such as swelling (Equation (5), page 33), the authors attempted the possibility of stabilizing clays by decreasing their swelling characteristics by means of appropriate blending of clay minerals to produce a soil with characteristics better than both of its constituents or, in other words, to produce a reduction in activity or plasticity index for the case concerned.

The use of the above results in practice is limited due to difficulties in locating the suitable materials and in mixing. In practice, it would be rather impossible to break down and mix natural clay soils to anything approaching the extent of artificial soils, and interstratification or compound particles could occur between large aggregates of clay particles. It is very difficult to get any appreciable penetration of chemicals into the clay soil except where the clay has dried out and cracked, and even then, the chemicals will not penetrate the large blocks of clay.

Although the results obtained were for artificial soils, it seems likely that the same principle should apply to the natural soils.

It has been found that there is a difference in the behaviour of the dry mixtures, subsequently wetted, and that of the mixtures obtained from wet material such as for work conducted in the field. The adsorbed cations have opposite effects for montmorillonite as for illite and kaolinite e.g. important modifications can occur in the properties of the mixture, because of the facility of calcium cations in replacing the adsorbed sodium on montmorillonite particles.

Parcher and Liu(8) carried out laboratory swelling tests on compacted and undisturbed clay specimens of natural over-consolidated Permian red clay deposits to determine the relationship between the horizontal and vertical swelling using a modified triaxial test appratus that enables vertical and horizontal swelling to be measured independently. Of the many factors affecting swelling, only two - initial moisture content and structure as related to compaction procedures were varied.

The results obtained showed that both the horizontal and vertical swelling vary inversely as the initial moisture content. Greater swelling in both the horizontal and vertical direction results when the compaction effort is increased, regardless of the initial moisture content. It was noted that in all cases the swelling in the radial (horizontal) direction is considerably greater than that in the vertical direction.

Swelling ratios of unity may be expected if the particles making up a soil specimen are situated in a perfectly random manner. Flat particles under pressure tend to become oriented with their broad surfaces perpendicular to the direction in which the pressure is applied. Compacted specimens, if not random in structure, should tend towards a structure in which the particles are horizontally oriented. For the heavily over-consolidated soils, it seems certain that the particles would tend toward horizontal orientation of a fairly high order. Such an arrangement should result in potentially greater swelling vertically than horizontally.

It is suggested that the results obtained by Parcher and Liu (8) may be confirmed, by adopting the following modification in the testing procedure:

(1) A few samples may be tested in which the vertical axis during compaction or in situ becomes the horizontal axis during testing.

(2) Some tests may be carried out with smaller height - diameter ratios so that the penetration of water through the sample is satisfactory.

(3) Similar tests may be carried out with another type of soil, particularly, an expansive soil, containing different mineralogical composition as pointed out by Wooltorton (36).

It has been established by some investigators that swelling may be grouped into two catagories:

(i) The lattice swelling which depends on the clay minerals present in the soil.

(ii) Swelling caused by the removal of surface tension forces in the voids.

Only lattice swelling occurs perpendicular to the direction of plates. The lattice swelling is not predominant in a less swelling clay mineral (leda clay) and in such soils undisturbed samples with random particle orientation showed higher swelling than remolded samples where parallel orientation will exist to a certain extent. The opposite effect was found for the Seven Sisters clay, which contains more swelling clay particles. Wooltorton and Clarke (36) found that for an undisturbed alluvial Sodium Severn clay the ratio of horizontal to vertical swelling was approximately two, but for an undisturbed Calcium Kummeridge clay, the ratio was unity.

Parcher and Liu (8) also observed that the increase in volume of the specimens was always smaller than the volume of water imbibed during swelling, causing an increase in degree of saturation. The degree of saturation after swelling has ceased is practically independent of the initial moisture content or initial degree of saturation. Compacted specimens swell more than the undisturbed specimens, under identical conditions of initial moisture content and initial dry density.

Alpan (13) designed an apparatus which enables the measurement of swelling pressures developed in a soil throughout the range of moisture variation below full saturation. When a soil sample is allowed to take up water under tension, it takes up moisture until a suction equilibrium is reached - the soil at this stage is not necessarily at full saturation. In fact, the higher the tension in the water, the lesser the degree of tension attained by the soil. Hence, the soil may be brought to the required degree of saturation and its expansive behaviour studied. The effort to keep the volume of the soil sample constant was not very successful due to the deflection of the proving ring which was used to measure pressures developed in the soil. The stiffer proving ring, preventing expansion to a marked degree, caused the greater swelling pressures to be developed in a shorter time. The writer feels that this difficulty may be overcome by using pressure transducers to measure the swelling pressure. When the degree of saturation is less than 110%, the author assumes that the water phase is uniform. Under these conditions, it is quite probable that moisture content gradient is set up in the sample.

Komornik and Zeitlen (12) devised a special apparatus to measure the lateral pressures developed during the saturation of expansive clay soils. The apparatus consisted of a special consolidation ring with a thin wall section in its central portion, allowing the use of electrical strain wires to determine the applied internal pressure. The authors maintain that the device gave consistent and reproducable results both during calibration and testing of soil in swelling studies. It was found that the lateral pressures were much larger than the vertical pressure during swelling. Dawson (39) also indicated the possibility of lateral pressures being developed due to the swelling clays, but confirmed that all observations indicated that at least the major portion of the stress was an upward lift.

Mielenz and King (30) found that a soil composed of montmorillonite showed a total volume increase from a dry to a saturated condition ranging from 2 to 21.6% with variations in dry density from 74.1 to 96.5 lb/cu. ft. The pressures developed in maintaining the constant volume range from 5.1 to 146.6 psi in the same series. These authors point out that if expansive clays are subjected to initial loads of high magnitude tremendous swelling pressures are developed. They found values as high as 540 psi developed in sodium montmorillonite that was originally compressed at 5,000 psi, and subsequently wetted following a period of unloading sufficient to permit relaxation of the specimen. Initial loading of the clays at 3,000 psi instead of at 5,000 psi, produced lesser swelling pressures. Consolidometer tests of undisturbed clays and shales containing illite or montmorillonite - type minerals in the clay fraction indicated potential hydration pressures as high as 15 tons/sq.ft. Dawsan (30) reported that expansive pressures exerted by confined bentonite clays have been observed to reach 15 tons/sq.ft., with the usual range being from 1 to 6 tons/ sq.ft.

The variation in the value of the swelling pressures as obtained from various sources (40) is shown below. The methods by which the pressures were determined, however, are not known.

Location	Clay Minerals	Desiccation	Swelling Pressures (tons/sq.ft.)
San Antonio, Texas	Calcium Bentonitic	To 17 ft.	4 - 5
Cisco, Texas	Generally Bentonitic	Yes	0 - 7
London, England		To about 5 ft. During droughts	1.2
South Africa	Generally Illitic	To at least 10 ft.	3 - 5
Mandalay, Burma	Illite with some Montmorillonite	12 ft.	1.75
Nkalagu, Nigeria	Montomorillonite and Kaolinite	To at least 18 ft.	11.5

Jennings and Knight (10) proposed a method, based on the degree of desiccation, the activity of the soil, over-burden and applied loading, to predict the total heave of structures from information given by "Double Oedometer Test". The findings apply mainly to a particular condition, where continual movement of covered surfaces is obtained. de Bruijn (41), William (42) and Donaldson(43) discussed the validity of the application of the results of the double oedometer test. Williams (44) gave experimental evidence in support of the above test as defined and applied by the authors. He found that the same ulitmate condition is achieved regardless of whether inundation occurs first and then loading, or vice versa. But the results obtained are contrary to those obtained by Holtz and Gibbs (1) who emphasised that the final swelling pressures or conditions attained depend on the procedure adopted during the test.

So far, most of the research effort on the heaving of expansive clay has been concentrated on finding methods of predicting the final heave of structures. Apparently, the only serious attempt to predict the time-heave relation for structures founded on expansive clays has been that of de Wet (11). de Wet found that there is always a constant relationship between the lateral and vertical strains. This is contrary to the results obtained by Parcher and Liu (8).

CHAPTER 3

PROBLEMS ASSOCIATED WITH EXPANSIVE CLAYS

Damages to structures resulting from the swelling of clays have been well documented over the years. In regions which have well defined alternately wet and dry seasons, susceptible soils swell and shrink in regular cycles. The volume changes associated with these and the pressure thus developed are liable to cause considerable distress to structures. In India the black cotton soil cracks very badly on drying and in worst cases the cracks are almost 6 in. wide and travel about 9 ft. to 10 ft. deep.

The most difficult problem associated with the above phenomenon is to decide the source and the variation of moisture. Besides accidental sources such as broken drains, there are three possible sources of moisture:-

(a) Upward differential flow of moisture from the water-table.

(b) Penetration of rain-water down vertical or near vertical fissures in the soil profile, along the perimeter of the structure. The lateral diffusional flow takes place into the soil beneath the structure.

(c) Lateral penetration of rain-water under the sturcture in the layer of sandy surface soil, followed by downward diffusional flow towards the water-table.

It should be noted that in the case of diffusional flow, the supply of water is not sufficient to fill the fissures in the soil and flow takes place by diffusion through the pores of the soil. In rain-water penetration, an abundance of water is present and flow takes place through fissures in the soil. Penetrational flow consequently takes place at a more rapid rate than diffusional flow (47). Swelling takes place through the supply of moisture from all three of these sources in combination. However, it may be that the effects of one or two of the sources of supply will have an overriding effect, under different climatic and soil conditions. It is important to appreciate the results of extraneous influences, such as broken drains and gardening operations. Common causes of these effects are broken drains, leaking water pipes, local concentration of stormwater, heavy watering of gardens, and and French drains. All these cause increase in moisture content. The drying out due to trees and shrubs, and reducing watering of gardens causes a decrease in moisture content. This effect may, at times, be severe.

Soil type, climate and topography are among the most important factors determining the seasonal moisture regime. Once the effects of changes in the moisture distribution under existing or proposed structures have been appreciated, suitable engineering solutions to counter these effects can be applied.

The structures most commonly damaged by expansive soils are:

(a) Dwelling houses of conventional construction (48)

(b) Roads and runways with impervious surfacing (46)

(c) Irrigation canals and spillway structures (1)

Structures may be designed either to resist differential movements caused by heave, or to accommodate these movements by means of flexible construction (37). For such structures the amount of differential heave to be expected constitutes an important part of the design information. The rate at which the expected movement will take place is, however, of secondary importance. However, in case of some structures e.g. roads and low cost housing it may prove more economical to use conventional construtions and to repair the structures at intervals while the heave movement is in progress. In order to decide on the most economical solution, it may be necessary to have predictions not only of the total movement to be expected, but also of the rate at which the movement will take place and the swelling pressures which are likely to develop. However, it has been found that damage due to swelling is so persistent that repair costs may usually run high.

The type and cause of damage produced by swelling pressures due to expansive clays and the preventive methods to overcome the damage have been discussed by various authors, notably, Means (35); Williams (44); Russam and Dagg (38); Dawson (39); Mohan and Rao (19); Mc. Dowell (29); Blight and de Wet, (47).

Past and current research in Israel (Zeitlen and Komornik, (33); Kassiff and Zeitlen, (34)) where the climatic conditions consist of a long dry summer and a relatively short winter, has shown that damage to structures founded on clay has been caused mainly by swelling and the effect of shrinkage is relatively minor. Laboratory tests carried out by Palit and Joshi (32) showed that most of the force which is generated inside the soil due to shrinkage is used up in compacting the soil. Only a small amount is available for exerting pressure on adjacent structure. It was found in Israel that relatively high stresses resulted in pipes through inequalities in the lateral and vertical swelling behaviour of the clay and that these stresses could be greater than those caused by internal pressure. The swelling forces were greatly influenced by the effect of lateral restraint of the pipe. Reports from various countries of rupture of cast iron, concrete and asbestos pipes, buried in swelling clays are available (Baracos and Bazozuk, (25)).

According to Holland (23) asbestos-cement pipes laid in expansive clays break mainly under the influence of soil shrinkage forces and that the magnitude of shrinkage forces on buried pipes is much greater than those resulting from swelling.

CHAPTER 4

FACTORS AFFECTING SWELLING

4.1. General.

The magnitude of swelling of a clay water system depends on a variety of factors. Amongst these are the factors that depend on the nature of the soil particles, and determine, in the first instance, whether the soil would have the capacity to swell under any conditions. These factors may be considered as those determining the swelling potential of a soil. As these factors depend only on the nature of the soil particles, they can be determined by laboratory classification tests on disturbed or remolded samples. There are other factors which have an important influence in determining the extent to which the swelling potential of the grains comprising the soil may be realized. These are not related to the nature of the soil particles. In fact, these factors are determined by the placement conditions and the environmental conditions for the soil.

4.2. Clay Mineral Composition.

The mineralogical composition of a soil is important as certain clay minerals e.g. montmorillonites have greater swelling potential than others. It is also important to determine the cation present, as sodium-montmorillonite has far greater swelling potential than calcium-montmorillonite.

4.3. Non-clay Mineral Composition.

Non-clay minerals having particles finer than 2 microns (e.g. presence of cristobalite in Wyoming Bentonites) may contribute towards swelling. The non-clay minerals generally tend to be concentrated in particles coarser than 2 microns.

4.4 Organic Material.

The influence of the quantity of organic matter present in a soil is not clearly understood. It is said that organic ions are adsorbed on the basal plane surfaces of clay minerals. Gieseking (30) reported that montmorillonite clays lost their tendency to swell by water adsorption when saturated with a variety of organic cations, whereas Biczok (51) reported that the phenomenon of swelling is also exhibited by soils which contain a large quantity of organic matter.

4.5 Exchangeable Ions and Soluble Salts.

The clay minerals and some of the organic material found in clay materials have significant ion-exchange capacity.

The base-exchange capacity of a soil containing a high percentage of clay and only one clay mineral is an indication of the nature of the clay mineral present as shown below.

(after Grim (30))

Clay Mineral	Base-exchange Capacity meq./100 gms.
Montmorillonite	60-100
Illite	20-40
Halloysite	6-10
Kaolinite	3-15

The less securely the replaceable cations are held i.e. the less their surface energy of adsorption, the greater will be the induced osmotic energy available for adsorption. The correlation curve for a powdered low-energy sodium clay would thus be expected to exhibit a greater swelling and a higher plastic index for a given base-exchange capacity than would a calcium modification.

4.6 Other Minerals with Cation-Exchange Capacity.

Inorganic materials of extreme fineness have a small cation exchange capacity. Zeolite minerals may have cation-exchange capacity of the order of 100 to 500 meq. per 100 g.

4.7 Texture.

The particle-size distribution of the constituent particles, the shape and the orientation of the particles in space and with respect to each other and the forces tending to bind them tother effect swelling.

4.8 Initial Moisture Content.

The amount of swelling exhibited by a soil is proportional to the amount of water imbibed by it. An inverse relationship is to be expected between the initial moisture content of a soil and its capacity to swell.

4.9 Soil Structure.

In naturally sedimented soils the arrangement of the grains is affected by the environmental conditions prevailing during deposition. The arrangement of grains in the soil structure during compaction is affected by the energy of compactive effort; the manner in which compaction is achieved and the moisture content during compaction. Ladd(16) found that the orientation of clay particles changes with molded water-content and that compaction dry of optimum water content tends to produce a nonparallel or flocculated orientation, while compaction wet of optimum leads to a parallel or dispersed orientation of clay particles. Different methods of compaction may also yield different particle orientations, even at the same density and water content. Different particle orientations may be expected to cause difference in swelling behaviour (8).

4.10 Properties of Water.

The amount of water absorbed by soils is influenced by the nature of the exchangeable cations initially present and by the kinds of ions dissolved in the added water. In fact, one can considerably prevent an expansive clay from swelling by putting sufficient salt in the water available for imbibition.

4.11 Confining Pressure.

It has been observed that the magnitude of swelling is inversely proportional to the restraining pressure.

4.12 Curing Period.

Barber (5) found that the swelling pressure of compacted specimens decreases with increasing time of curing before testing. A substantial difference in swelling pressure was found when the curing time was increased from 5 mins. to 24 hours.

4.13 Time Permitted for Swelling.

The amount of swelling that will occur within a given period depends on the quantity of water that enters the soil. Thus, the rate of swell is proportional to the coefficient of permeability and to the hydraulic gradient.

4.14 Temperature.

It was noted by Lambe (18) that under constant load clay specimens were compressed when the temperature was increased and that they swelled when the temperature was decreased. This volume change was experimentally proved by Salas et. al. (37). However, the magnitude of this effect seems to be not important in practice.

4.15 Structure of the Clay Unit.

The clay unit or micelle may be considered to consist of a negatively charged anion, forming a nucleus, generally an alumino-silicate, surrounded by positively charged cations of the alkaline earth bases(40). In order that the whole system constitutes an electrically neutral system, the cations are not in intimate contact or association with the soil anion but exist in varying degrees of dissociation, depending upon the magnitudes of the attractive and repulsive forces acting between the various ions (Fig. 4.12).

On account of the dissociated nature of the cations the micelle has a resultant negative charge. Water molecules, being bipolar, are attracted to the particle surface and orient themselves on the surface like tiny magnets as shown in Figs. 4.13 and 4.14.

4.16 Adsorption of Water.

The adsorption of water by a clay leads to expansion or swelling. According to Mielenz and King (31), the mechanisms are involved in the swelling of soils:

(i) A relaxation of effective compressive strength related to enlargement of capillary films.

(ii) Osmotic imbibition of water by clay minerals with an expanding lattice. The high swelling of the montmorillonite-type clays is the result of the property of this mineral to adsorb water between the individual silicate layers.

Falconer and Mattson (40) explain the phenomenon of osmotic imbibition. When in aqueous solution, a soil micelle dissociates ions, which surround the particle, in a diffused atmosphere. The density of this atmosphere will depend on the nature of the ions present and increases as the surface of the particle is approached. The amount of moisture adsorbed by the micelle, whether in water or in a saturated atmosphere, is, however, greater than can be accounted for by the hydration of ions themselves.

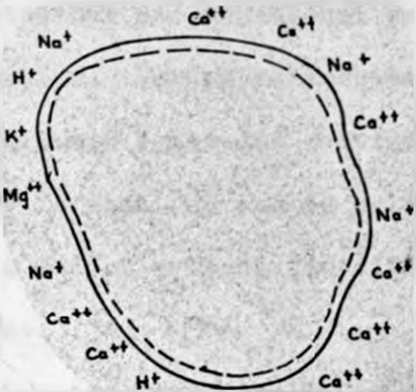

FIG. 4.12. THE HYPOTHETICAL SOIL - SALT PARTICLE IN SOLUTION.

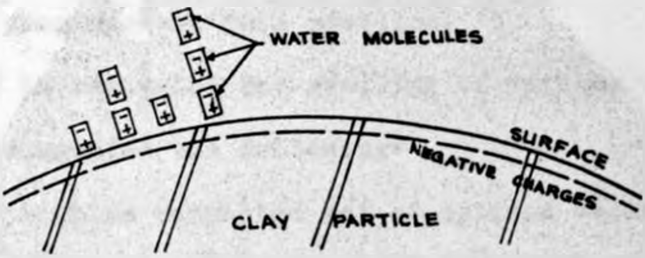

FIG. 4.13. CHARGE ON A CLAY PARTICLE

It is, therefore, conceived that when the surface energy of adsorption is satisfied, the hydrated ions form a diffused layer which acts as a membrane when an osmotic force comes into existence and additional energy becomes available for adsorption. During this state water will be adsorbed into the diffused layer and continue to enter till the osmotic pressure is equal to and is balanced by the electrostatic forces acting between the diffused ions and the colloidal nucleus.

As a sodium-saturated clay micelle is surrounded by a more diffused layer than a calcium-saturated clay micelle, it follows that the former tends to adsorb, by osmotic imbibition, far more water than the latter, resulting in considerable more swelling.

Ladd(16) investigated the swelling of various clay minerals and suggested the following:

(a) For samples compacted wet of optimum water content, swelling is mainly influenced by osmotic pressures.

(b) For samples compacted dry of optimum water content, swelling is influenced by a number of factors such as cation hydration and attraction of the particle surface for water, a flocculated particle orientation, presence of air, and osmotic pressures. The relative importance of these factors is not known.

Effect of Molded Water Content on Pore Water Tensions and Degree of Saturation has been explained by Ladd (16). Pore water tensions (water pressures less than atmospheric pressure) exist in compacted clay, particularly if compacted dry of optimum water content where the initial degree of saturation is well below 100%.

The spaces between the small clay particles form capillary tubes. When the pores of the clay are completely filled with water and there is free water on the surface as at A in Fig. 4.15, there exists no tension in the water and no compression in soil grains. Evaporation forms menisci in the pores near the surface as at B.

The tension in the pore-water is given by the following relationship;

$$U = \frac{2Ts}{R}$$

Where,

Ts = Surface tension of water,

R = Radius of the meniscus,

U = Pore-water tension.

Because the stress in the water is distributed equally throughout the water in the communicating pore spaces and tubes, the radii of all the menisci are equal. The menisci

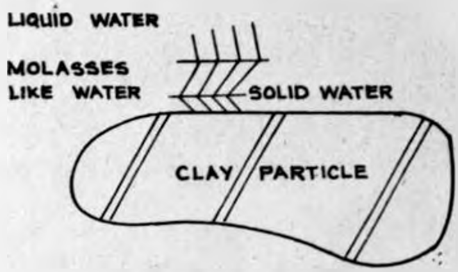

FIG. 4.14. ADSORBED WATER ON CLAY PARTICLE

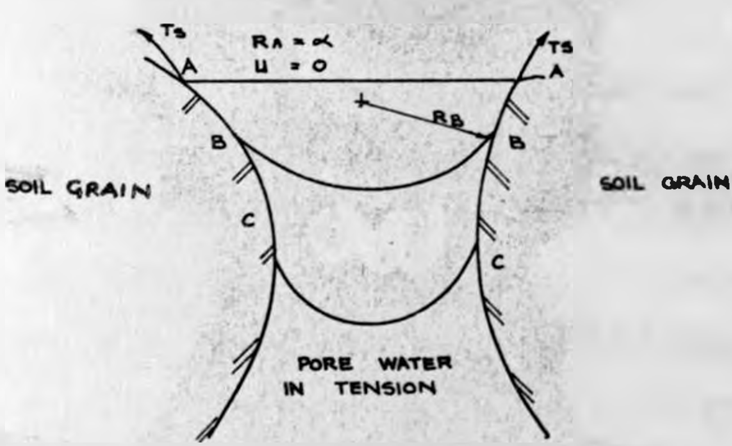

FIG. 4.15. CAPILLARY ACTION IN SOIL

react against the soil grains producing a compression between the grains somewhat as though the soil mass were held with elastic bands stressed in tension. If the soil is compressible, the compression between the grain causes a deformation or shrinkage of the volume. Further evaporation causes a receding of the menisci to the smallest portion of the largest connecting pore as at C. At this stage the greatest pore water tension and corresponding compression in the soil will be developed. The clay has attained its maximum shrinkage.

4.7 Presence of Air.

The presence of air can influence the magnitude of the pore-water tensions that are developed in compacted clay (16). Air may also influence the swelling behaviour in another way. Swelling data given by Holtz and Gibbs (1) show that the total volume of air in a compacted sample decreases during the soaking process, particularly for samples compacted dry of optimum water content, although the final degree of saturation is less than 100 percent. Thus, during the soaking process, some of the air initially in the soil voids must either escape from the soil, be dissolved by water, or be compressed by capillary forces. Most likely a combination of these conditions occur. If a nonspherical pocket of air is compressed in a soil void during the soaking process, the pressure in the air may produce tensile stresses in the soil skeleton forming the void in which the air resides. These tensile stresses could cause an increase in volume of the void, and hence an expansion of the soil.

CHAPTER 5

LABORATORY TESTS

5.1 Soil Studied in the Investigation.

The main soil used in these studies was a highly expansive clay, locally known as "Black-cotton soil". The disturbed and undisturbed samples were obtained from "Bernhard Estate", about 7 miles from the City Centre in Nairobi (Fig. 5.16).

Other soils used in the investigation were highly expansive clays from the Lake Amboseli basin. The samples were obtained from the Tanganyika Meerschaum Mines, at Sinya, about 23 miles South East of Namanga. (Fig. 5.17). The two main types of clays were as follows:

(1) Green Clay: The undisturbed soil was light grey in colour, and had a very-fine grained texture. It was very dry in its natural state. Few natural pores in the soil indicated the possibility of entrapped gases during natural consolidation. It was very hard and could not be broken into smaller pieces by hand. It was broken into about 1" pieces with a hammer. When water was added to the broken pieces, the colour changed to light green.

(2) Sepiolite: The undisturbed soil was light brown in colour, very light in weight and moderately fine-grained.

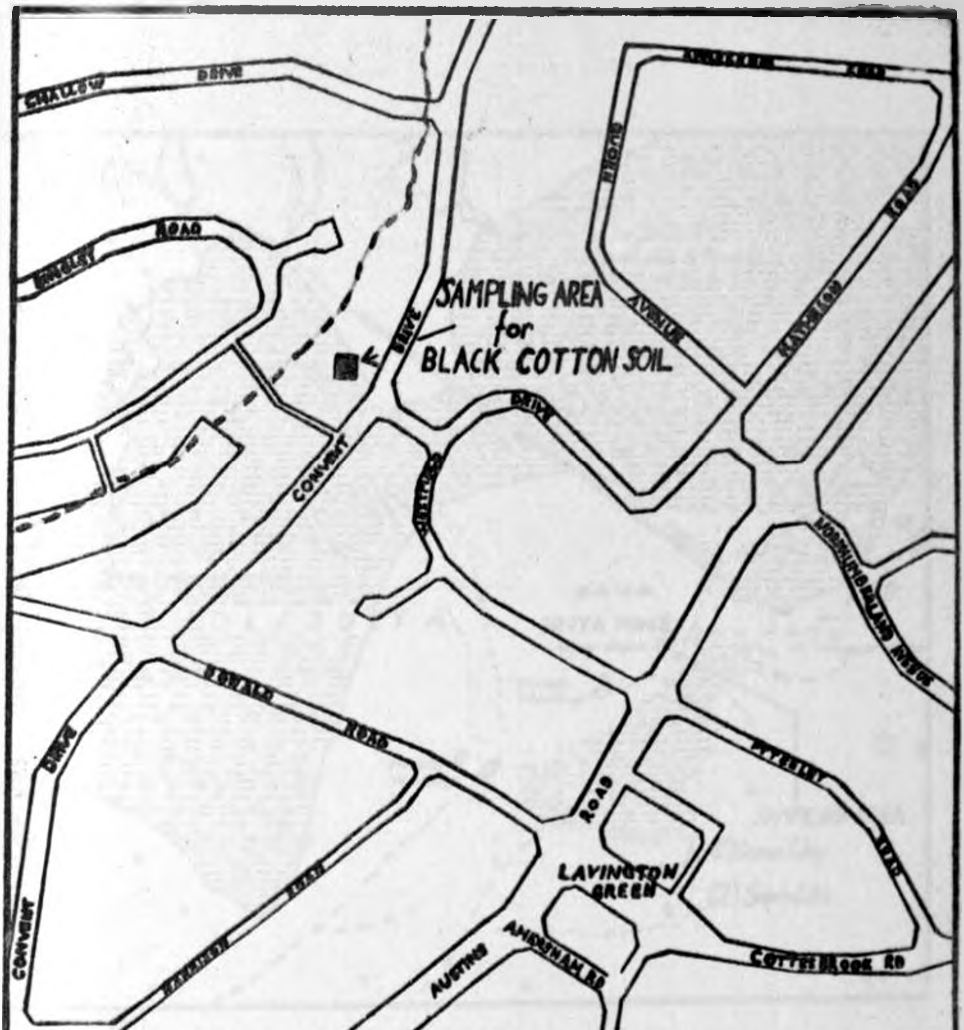

Fig.5.16. BERNHARD ESTATE (NAIROBI)
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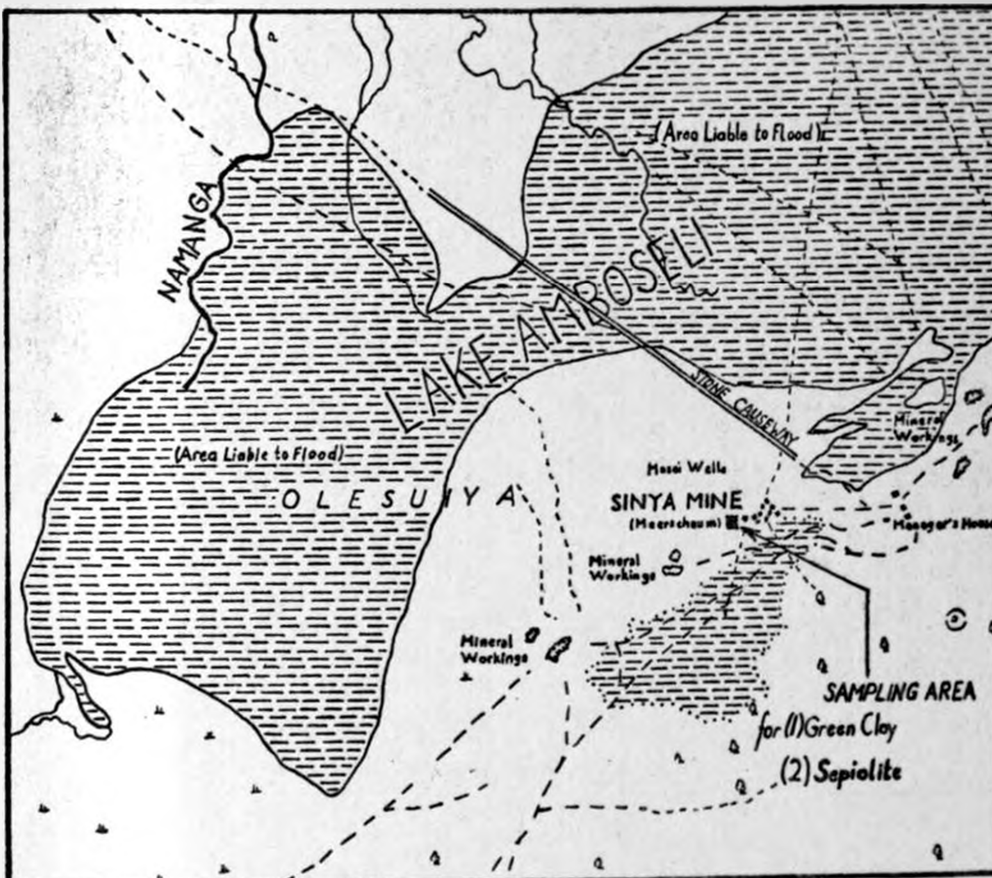

Fig. 5.17. LAKE AMBOSELI

1:50000

It was very porous and pores were evenly distributed throughout the structure indicating the possible presence of entrapped gases during formation. It was very dry and very soft in its natural state. Bigger particles could easily be crumbled with fingers. On addition of water, the colour changed to darker brown (deep tan) and the particles softened quickly.

5.2. Basic Tests.

The following tests were carried out for the soils investigated to study their basic properties.

1. Specific Gravity.
2. Liquid Limit.
3. Plastic Limit.
4. Grain Size Analysis - Hydrometer Test.
5. Compaction Test -
   (a) Standard AASHO,
   (b) Modified AASHO.
6. Free Swell.
7. Shrinkage Limit:
   (a) Mercury Displacement Method,
   (b) Determination of Shrinkage Limit from the changes in volume and moisture content.
8. Clay mineral composition.

Tests 1 to 7 (a) inclusive were carried out in accordance with the standard procedure outlined in the Laboratory Manual by Lambe (21).

Test 7 (b) was carried out in the following manner. The undisturbed sample was trimmed to the required initial volume in a consolidometer ring (or the disturbed sample could be compacted to the required density). The initial moisture content, weight and the volume of the sample were taken. The sample was then allowed to dry at room temperature. Readings of diameter, height and weight were taken at intervals. After some days the sample was dried at 110°C in the oven to constant weight. From the readings of diameter, height and weight of the sample during the shrinkage, graphs of percentage moisture content against volume were drawn and from these the shrinkage limit was determined. The foregoing procedure was employed for the determination of shrinkage limit, using remolded samples for each soil type. In case of Black Cotton soil further determinations were made on some undisturbed and some remolded samples statically compacted to various values of dry density.

The sample of Black Cotton soil was sent to the National Agriculture Laboratory, Nairobi, for the determination of the clay mineral composition, in case of Test 8. The pretreatment of the soil sample involved the removal of organic matter using hydrogen peroxide, dispersion with sodium hexameta phosphate, followed by fractionation into clay, silt and sand.

5.3. Results of the Basic Tests.

The results of the basic tests are tabulated in Tables 5.1, 5.2 and 5.3. The particle size distribution for each soil type is shown in Figures 5.18, 5.19 and 5.20. The results for the determination of shrinkage limit from the changes in volume and moisture content are presented graphically in Figures 5.21, 5.22, 5.23, 5.24 and 5.25.

Table 5.1.

Basic Properties

Test	Soil Type		
	Black Cotton	Green Clay	Sepiolite Clay
Specific Gravity	2.63	2.67	2.57
Liquid Limit (%)	93.8	253*	375*
Plastic Limit (%)	38	139*	208*
Plasticity Index (%)	55	114	167
Clay Fraction (-.002mm) (%)	75	67	61
Free Swell	387	530	1045
Proctor Test.			
(a) Standard AASHO			
(i) Optimum moisture content (%)	32.5	57.0	146
(ii) Maximum dry density(g/cc)	1.33	1.02	0.476
(b) Modified AASHO			
(i) Optimum Moisture Content (%)	26.6	47	
(ii) Maximum dry density(g/cc)	1.51	1.16	

* Considerable difficulty was experienced in carrying out the liquid limit tests for these soils due to their thixotropic characteristics.

Table 5.2.

Shrinkage Characteristics

TEST	Soil Type		
	Black Cotton	Green Clay	Sepiolite Clay
(a) Shrinkage limit (%) by Mercury displacement method	16.0	35.0	97.0
(b) Shrinkage limit (%) from shrinkage curves of an undisturbed sample	12.3		
(c) Shrinkage limit (%) from shrinkage curve of a remolded sample	13.5	33.0	79.8
(d) Shrinkage limit (%) from shrinkage curves of the statically compacted specimens compacted to initial dry densities of 1.5, 1.4, 1.2 (g/cc.)	11.3		

Table 5.3.

Clay Mineral Composition

Black Cotton Soil	
Clay <2 μ	
Montmorillonite	69.30%
Kaolinite	7.70%
Silt 2-50 μ	
Feldspars	14.70%
Quartz	6.30%
Sand 50-2000 μ	
Feldspar	1.80%
Quartz	0.20%

The Mg saturated montmorillonite expanded from 16.98A° to 18.39A° upon solvation with ethylene glycol.

$\therefore$ Expansion = 1.41A°

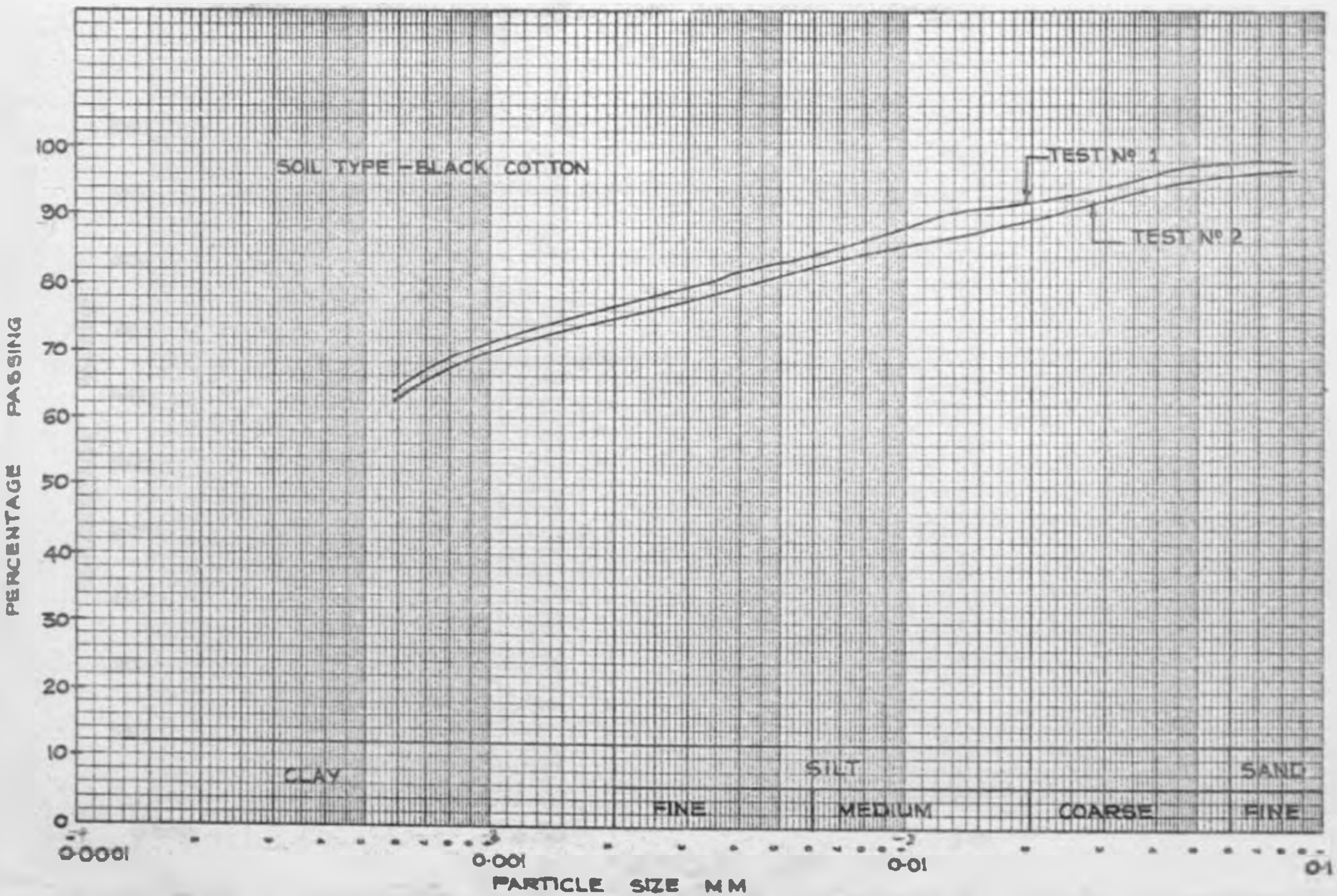

FIG. 5.18 HYDROMETER ANALYSIS - PARTICLE SIZE DISTRIBUTION

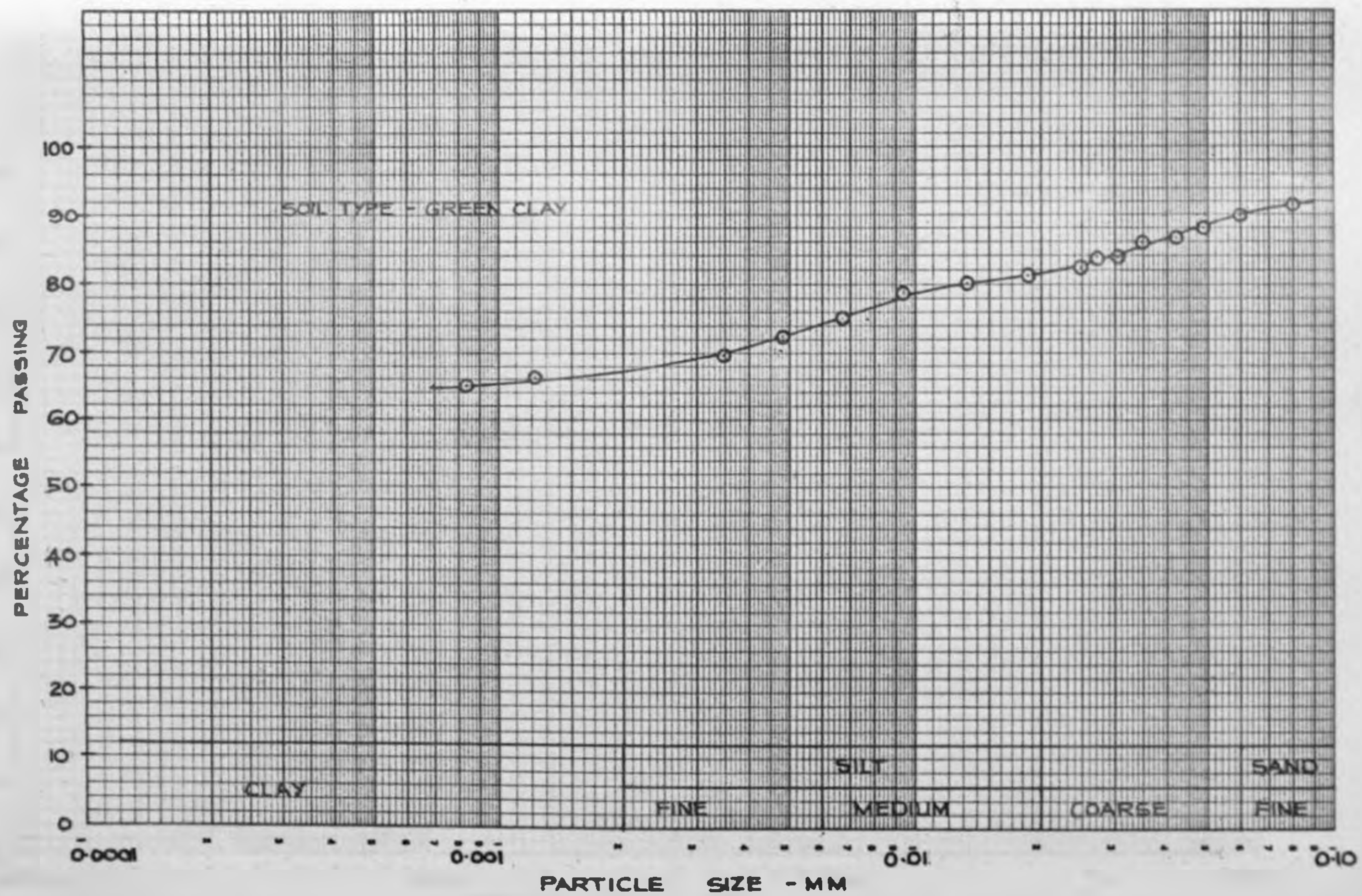

FIG. 5·19 HYDROMETER ANALYSIS - PARTICLE SIZE DISTRIBUTION

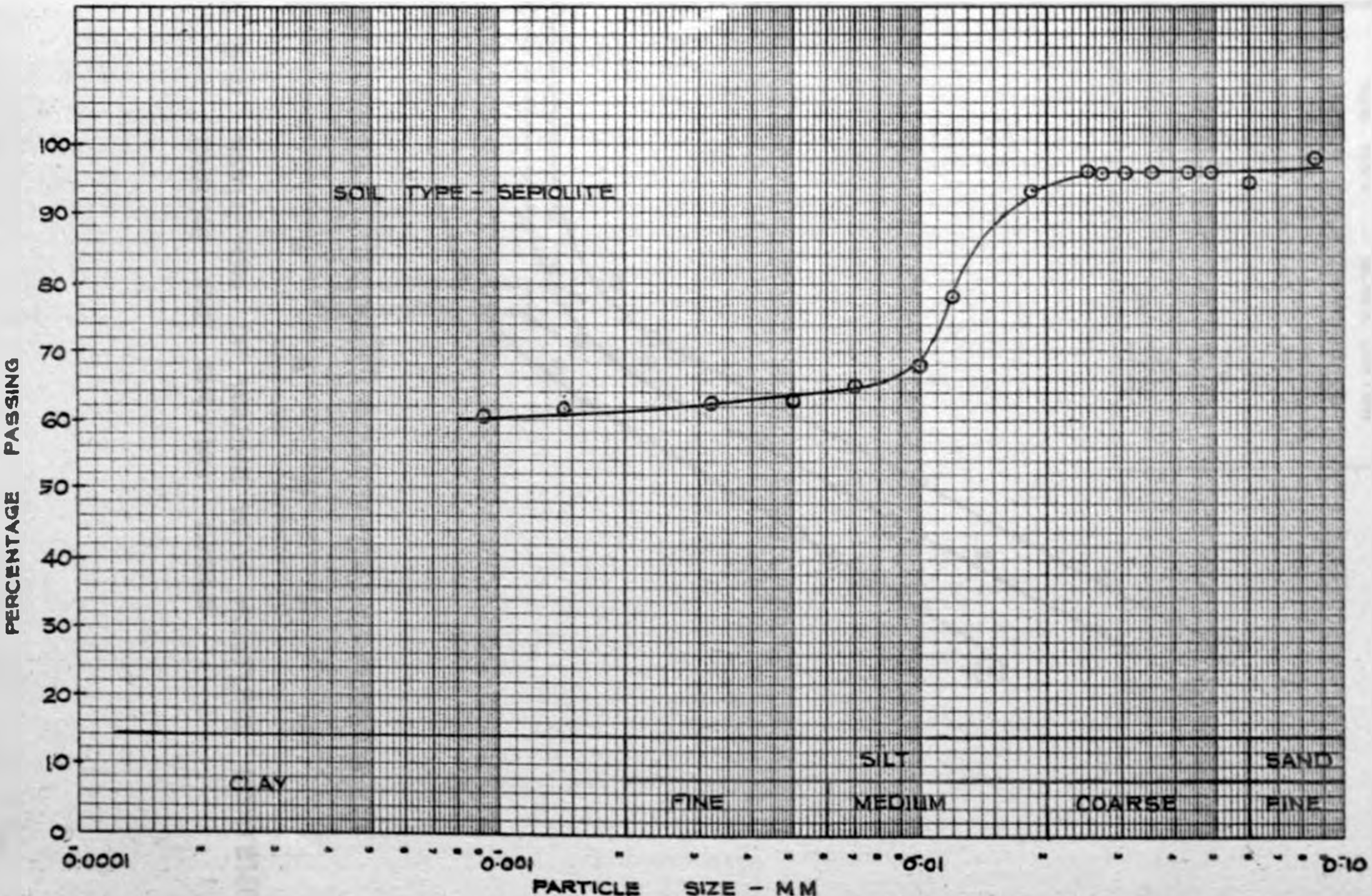

FIG. 5·20. HYDROMETER ANAYSIS - PARTICLE SIZE DISTRIBUTION

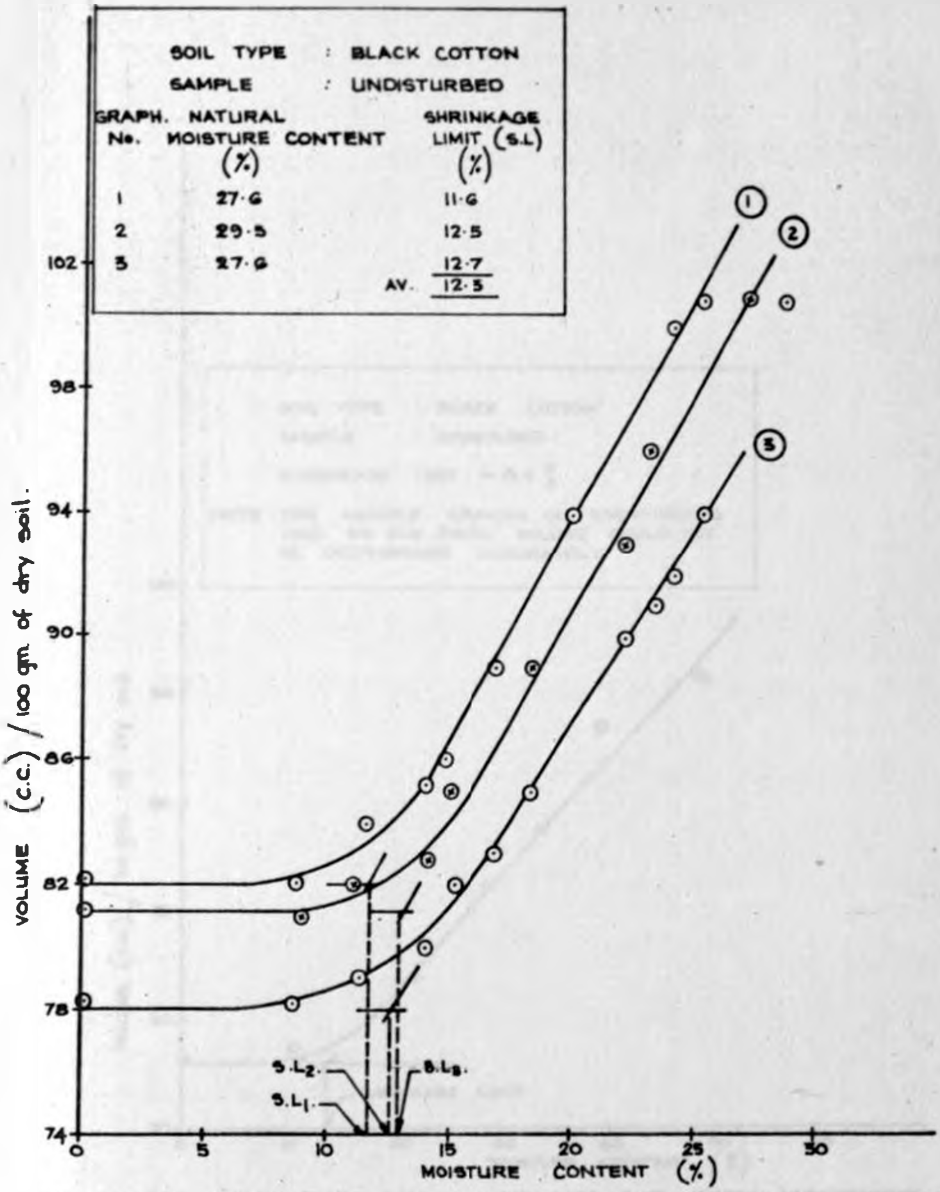

FIG. 5·21 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

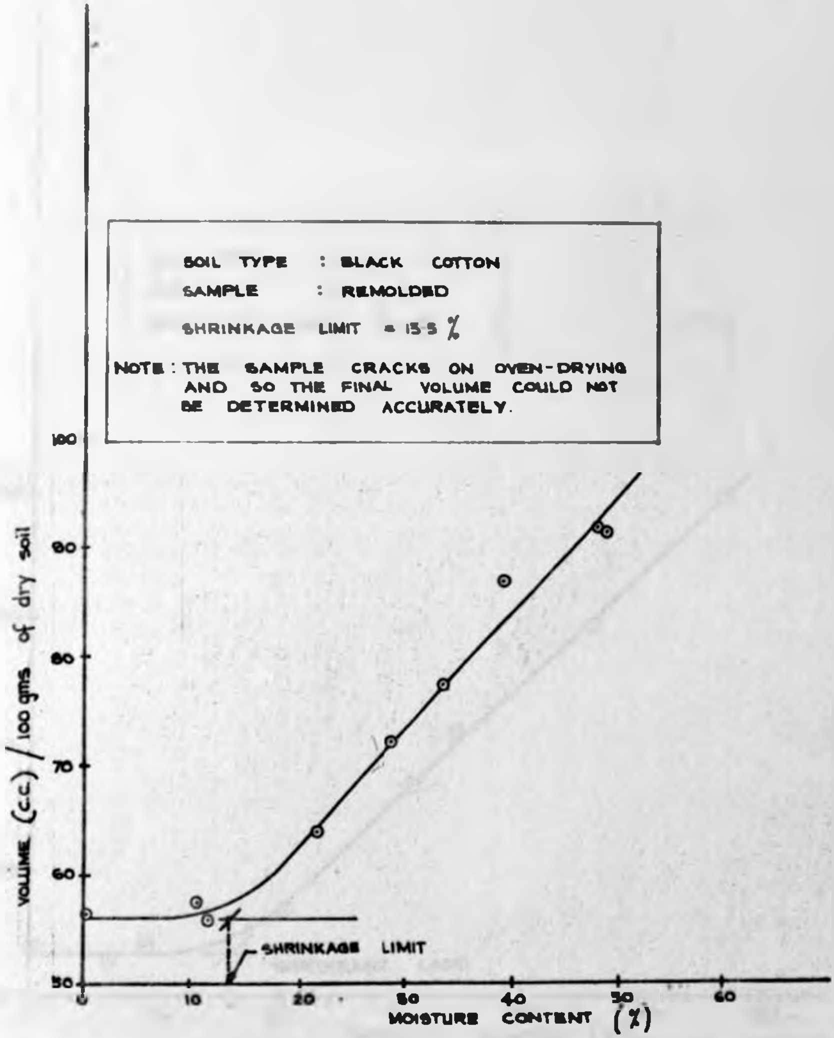

FIG. 5·22 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

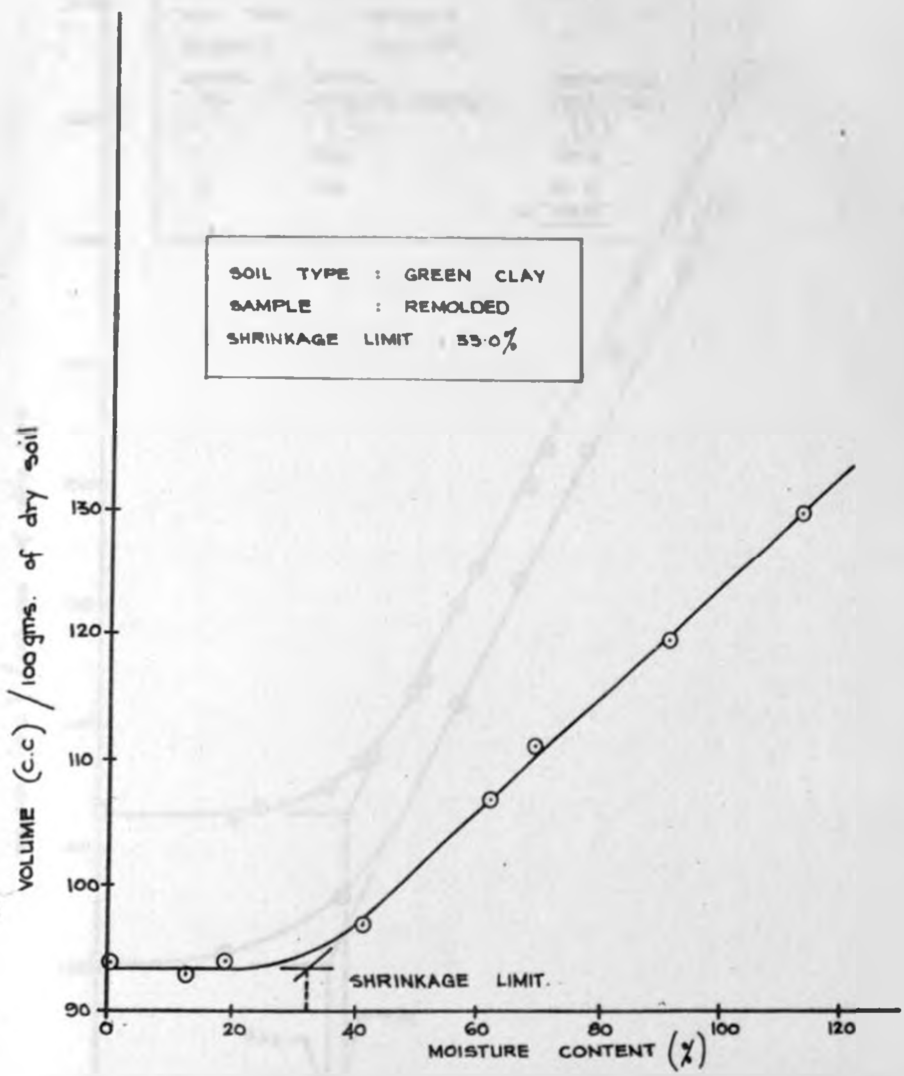

FIG. 5·23 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

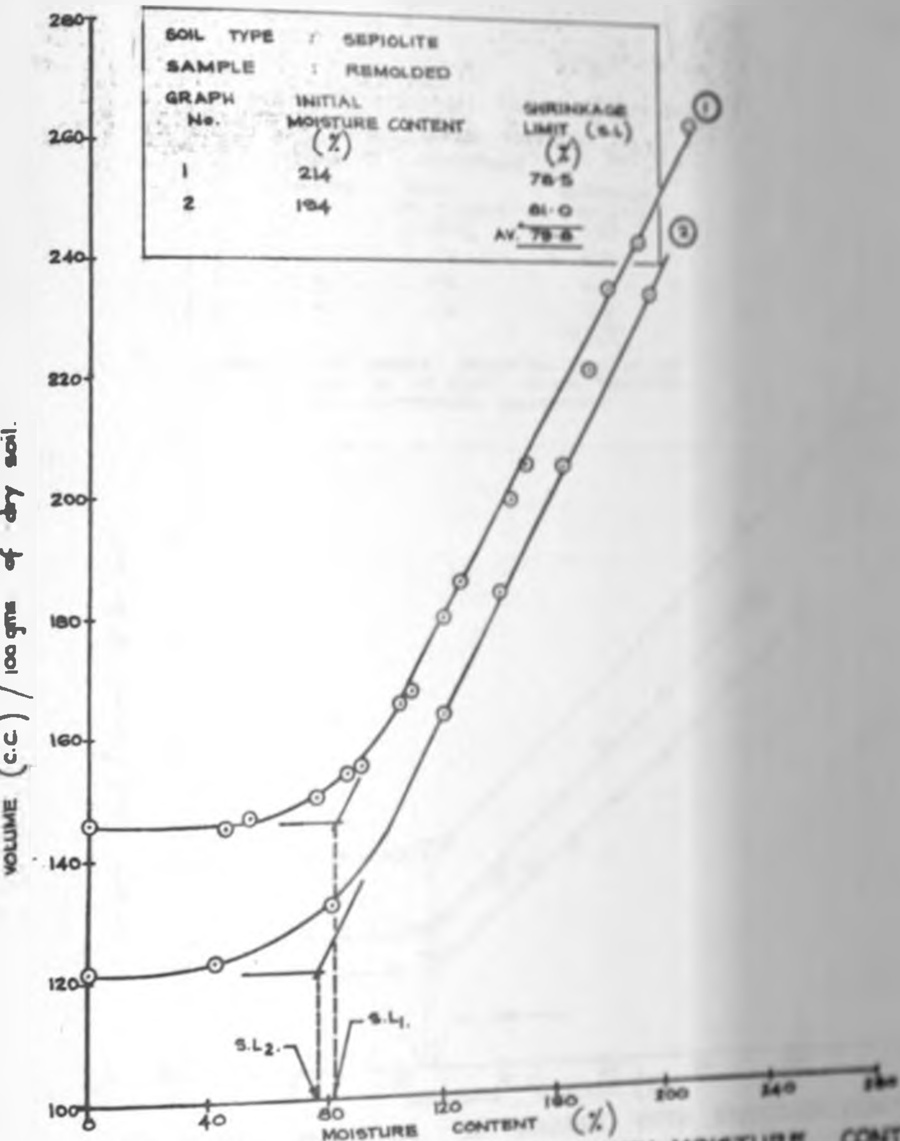

FIG. 5.24 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

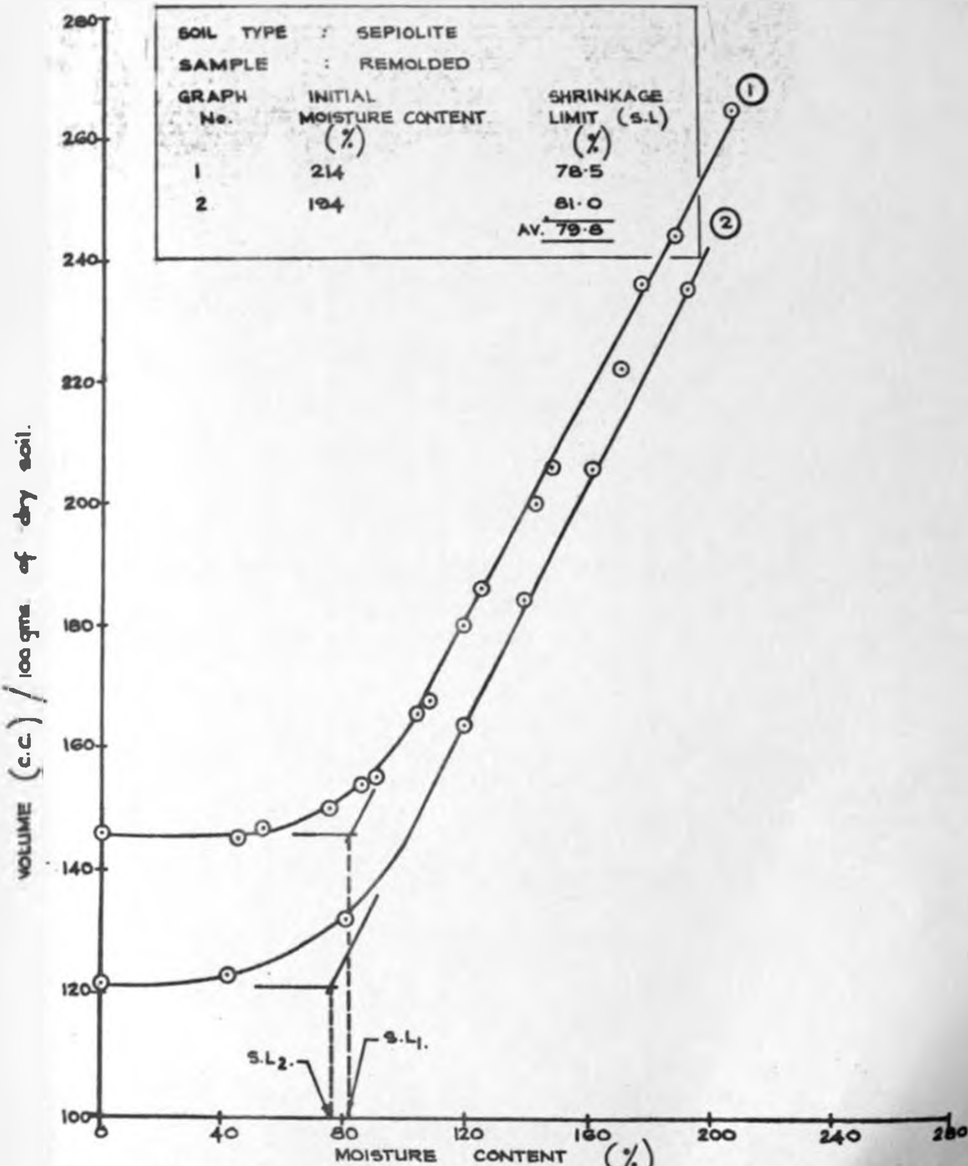

FIG. 5·24 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

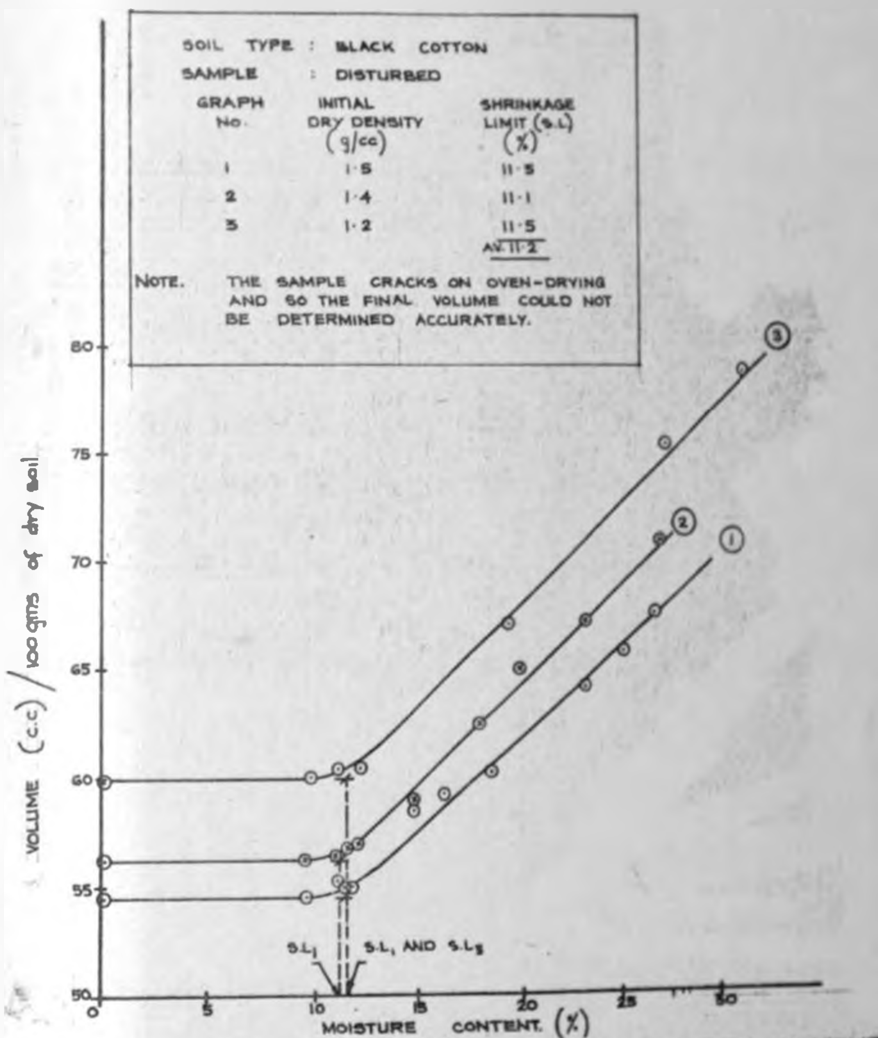

FIG. 5·25 VARIATION OF VOLUME WITH MOISTURE CONTENT DURING THE DRYING CYCLE.

5.4. Tests for the Evaluation of Swelling Pressures.

The basic aim of the research was to measure and evaluate the swelling pressures developed in expansive soils under controlled conditions of lateral and vertical confinement and at various conditions of moisture and density. The various apparatus used for the measurement of swelling pressures were designed and made in the department of Civil Engineering, University College, Nairobi. The investigation was primarily carried out by two methods:

(1) Indirect Method

(2) Direct Method.

5.4a. Indirect Method:

This method was used to evaluate the optimum swelling pressure. Under particular conditions of initial moisture content and initial dry density, this may be defined as the maximum pressure which is developed in the soil due to the increase in moisture content when the volume change is prevented. The apparatus used for the tests was the standard consolidometer, with two types of specially designed cells, for disturbed and undisturbed samples (Figs. 5.26, 5.30, 5.63). Various tests were carried out on statically compacted soil samples and on undisturbed soil samples of Black Cotton soil, at various value of initial moisture content and initial dry density. For a particular initial moisture content and initial dry density, tests were carried out on a number of samples, each sample being subjected to a predetermined load. The vertical expansion of each sample under the constant load was noted. From the results of all the tests the optimum swelling pressure was evaluated as the pressure at zero vertical expansion.

5.4b Direct Method.

The direct measurement of swelling pressure developed in a soil sample, held under controlled conditions of lateral and vertical confinement, was made possible by the Direct Method. Series of tests were carried out, on various disturbed samples of Black Cotton soil, Green clay and Sepiolite clay at various values of initial moisture content and initial dry density. The tests were carried out in a specially designed apparatus. (Fig. 5.70 and 5.75). The pressure was measured with the aid of electronic equipment, in terms of strain in a load cell. The gradual development of swelling pressure in the soil sample was recorded on an electronic recorder.

5.5. Summary of the tests carried out.

The main aim of the tests carried out was to evaluate the swelling pressure. An attempt was also made to study the influence of initial moisture content and initial dry density on the development of swelling pressure. Special tests were carried out, in case of the Direct Method, to study the influence of the height of the sample, on the maximum swelling pressure developed. The following series of tests were carried out:-

(1) Indirect Method. Test Series A-1: Tests on disturbed samples of Black Cotton soil.

(2) Indirect Method. Test Series A-2: Tests on undisturbed samples of Black Cotton soil.

(3) Direct Method. Test Series B-1: Tests on disturbed samples of Black Cotton soil.

(4) Direct Method. Test Series B-2: Tests on disturbed samples of Black Cotton soil, to study the influence of height of sample on the maximum swelling pressure.

(5) Direct Method. Test Series B-3: Tests on disturbed samples of Green clay.

(6) Direct Method. Test Series B-4: Tests on disturbed samples of Sepiolite clay.

5.6. Indirect Method. Test Series A-1: Tests on disturbed samples of Black Cotton soil:

Tests were carried out on statically compacted specimens at various initial moisture content and initial dry density conditions. At each initial moisture content and initial dry density condition, listed below, several tests were carried out under different loads.

Initial Moisture Content (%)	Initial Dry Density (g/cc.)
23.0	1.2
23.0	1.33
23.0	1.40
23.0	1.50
27.3	1.17
27.3	1.30
27.3	1.4
27.3	1.5
29.5	1.2
29.5	1.33
29.5	1.4

Initial Moisture Content (%)	Initial Dry Density (g/cc.)
32.5	1.2
32.5	1.33
32.5	1.4

5.6a Apparatus.

The apparatus consisted of a standard consolidometer, a burette, and a specially designed cell. The specially designed cell constructed for statically compacted specimens is shown in Figs. 5.28 and 5.29 and is made up of the following parts:

(a) The base A with 3/16 inch diameter hole for water intake from the burette.

(b) The ring B which contains the specimen.

(c) The base C into which the ring B can be screwed.

(d) The cover D just fits the ring B. It is used to compact the specimen in the ring which is screwed into the base C.

(e) The loading plate E fitted with a porous stone.

(f) The porous stone.

5.6b. Preparation of Sample.

The disturbed sample was first air-dried. It was then crushed and passed through British Standard Sieve No. 7. The required quantity of water was then added to bring the soil to the desired moisture content. The soil was cured for a minimum period of 48 hours in a desiccator.

5.6c. Procedure.

The ring B was oiled on the inside circumference to reduce friction between soil and the ring, and screwed into the base C. The calculated amount of soil at the desired moisture content necessary to give the desired dry density was placed in the ring. It was then statically compacted to the required dry density (Fig. 5.27). The compacting load was applied and held for at least 10 secs. This was repeated three times. When the pressure was released on final compaction, there was some rebound of the specimen - the maximum rebound was of the order of .001".

The ring containing the sample was removed from the base C. A rubber O-ring was put on it. It was screwed into the base A containing a saturated porous stone. A loading plate fitted with a porous stone was then placed on top (Figs. 5.26, 5.28 and 5.29). The cell was placed in the consolidometer. The initial dial gauge reading was

FIG.5.26. MODIFIED CELL FOR DISTURBED SOIL SAMPLE

INDIRECT METHOD TEST SERIES A-1

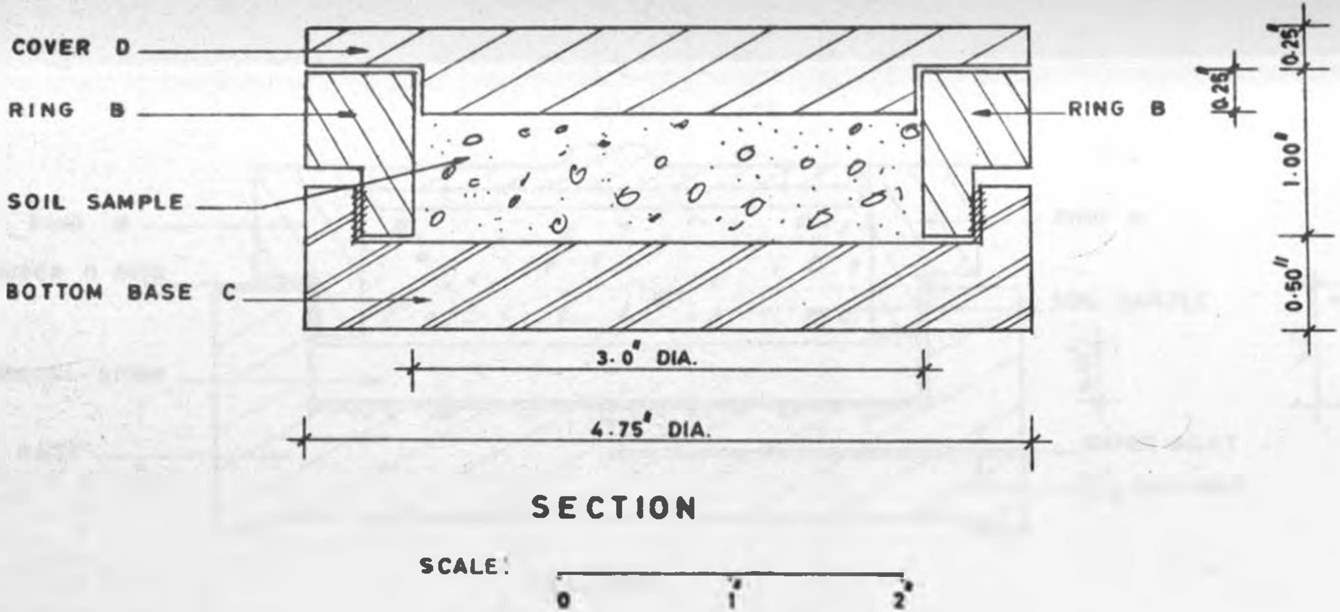

FIG.5.27. APPARATUS FOR STATIC COMPACTION OF THE SPECIMEN

INDIRECT METHOD - TEST SERIES A-1

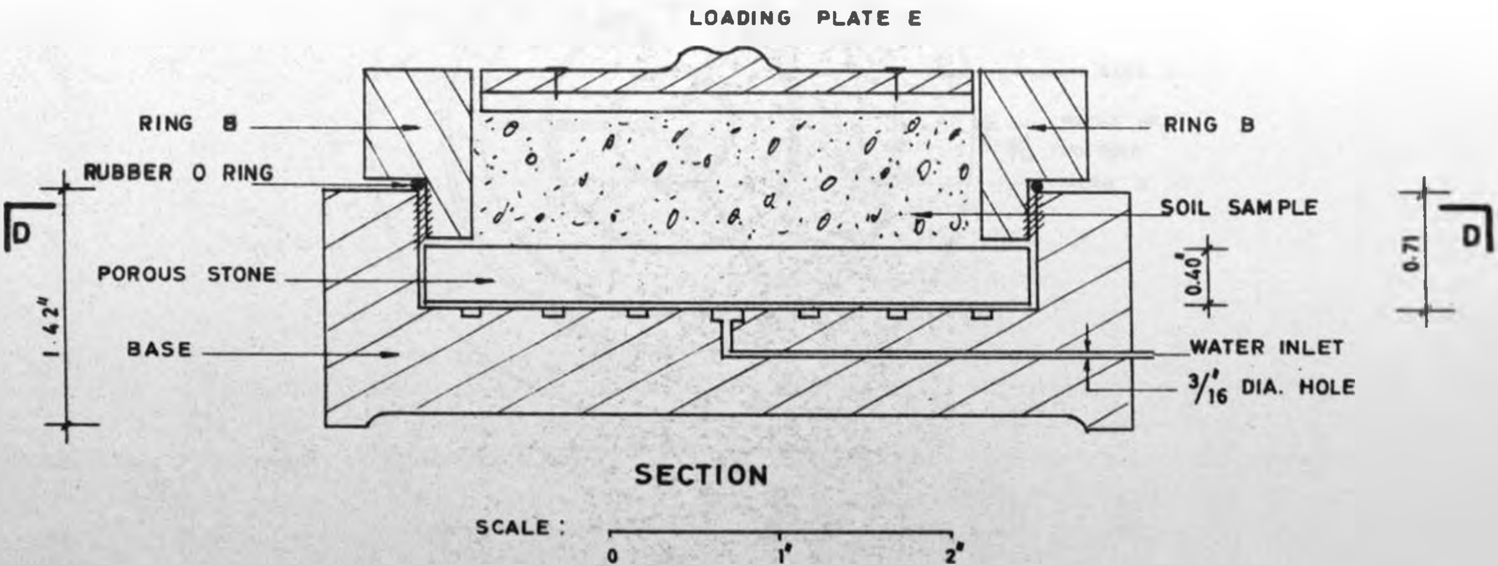

FIG. 5.28 MODIFIED CELL FOR DISTURBED SOIL SAMPLE INDIRECT METHOD TEST SERIES A-1

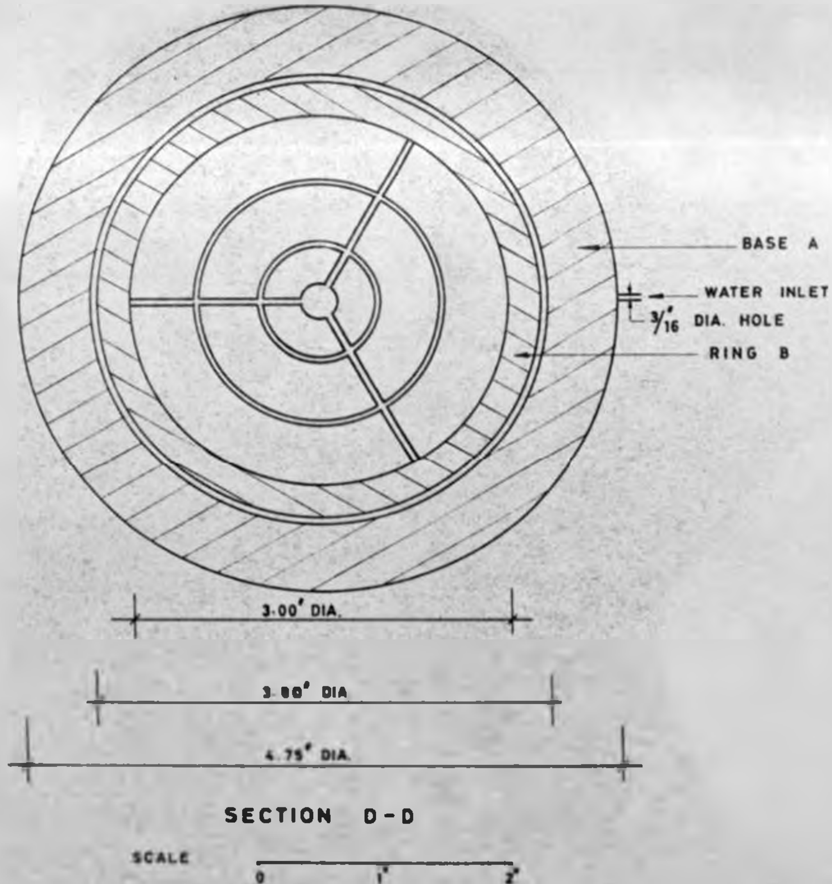

FIG. 5.29. MODIFIED CELL FOR DISTURBED SOIL SAMPLE
INDIRECT METHOD TEST SERIES A-1

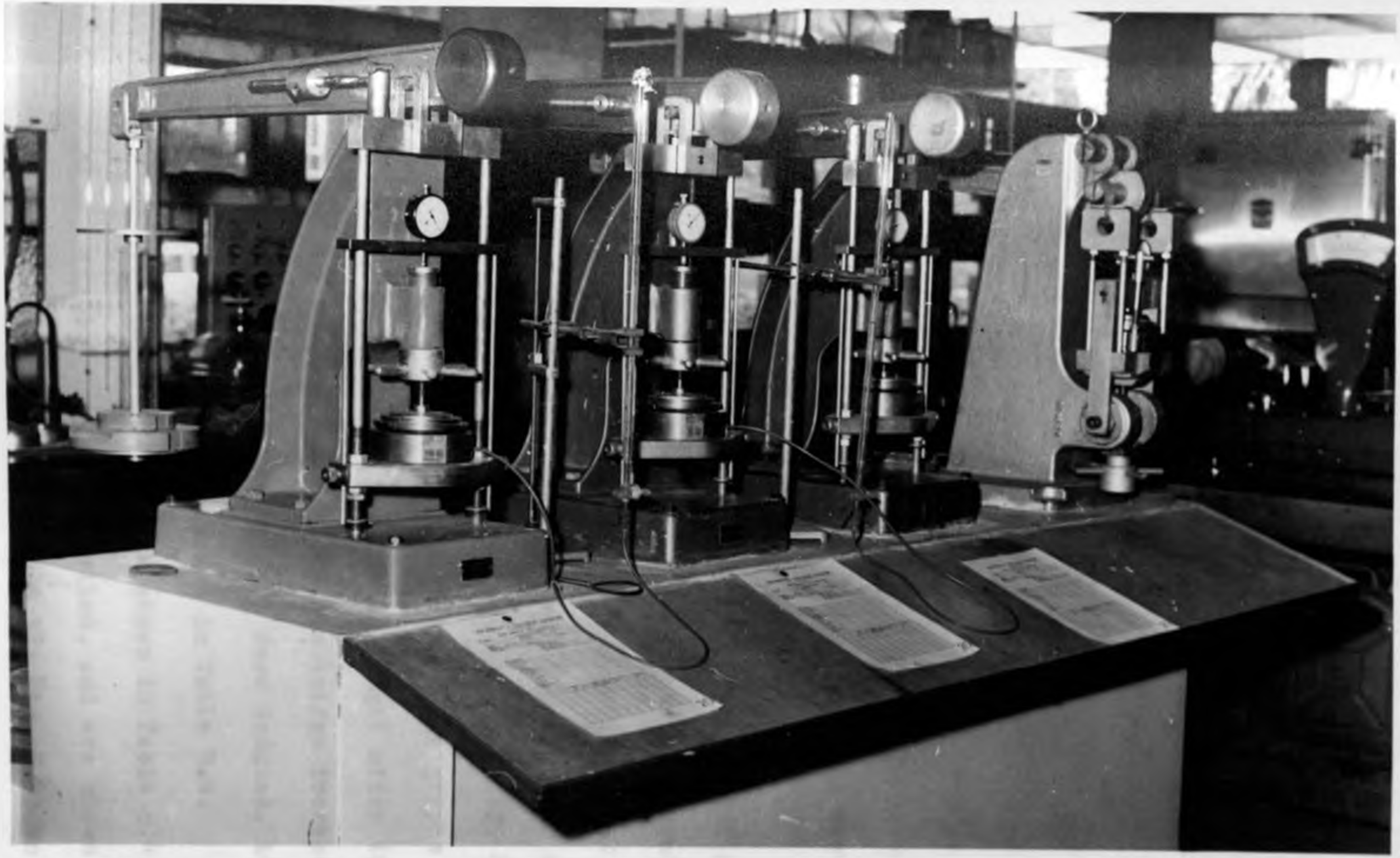

FIG.5.30. A BATTERY OF CELLS IN OPERATION.

INDIRECT METHOD TEST SERIES A-1

noted. The water tap was opened and at the same time a predetermined load was placed on the cell and this was kept constant throughout the experiment. The vertical dial gauge readings were noted at intervals. This was continued until no further swelling of the sample occured, after initial consolidation. Upon completion of the test, the sample was removed, weighed and kept in oven for at least 24 hours and reweighed to determine the final moisture content. Fig. 5.30 shows a battery of cells in operation.

5.6d Results.

For each of the tests at a particular initial moisture content, initial dry density and under constant load conditions, graphs of percentage increase in height of the specimen $\frac{\Delta H}{H}$ against time were drawn. These graphs are shown in Fig. 5.31 to Fig. 5.34; Fig. 5.39 to Fig. 5.42; Fig. 5.47 to Fig. 5.49; Fig. 5.54 to 5.56. From these graphs the maximum total percentage increase in height after initial consolidation $\frac{\Delta H_T}{H}$ and the maximum percentage increase in height above the initial height $\frac{\Delta H_i}{H}$ were deduced, in each case. These results are tabulated in Table 5.4.

With the aid of the results shown in Table 5.4, the following relationships were studied, and are shown graphically in Fig. 5.35 to Fig. 5.38; Fig. 5.43 to 5.46;

Fig. 5.50 to Fig. 5.53; Fig. 5.57 to 5.60.

(1) Balancing swelling pressure v. $\Delta H_T/H$ for various values of initial dry density.

(2) Balancing swelling pressure v. $\Delta Hi/H$ for various values of initial dry density.

(3) Balancing swelling pressure v. initial dry density at various values of $\Delta H_T/H$.

(4) Balancing swelling pressure v. initial dry density at various values of Hi/H.

The values of balancing swelling pressure, when $\Delta H_T/H$ is zero and when $\Delta Hi/H$ is zero were determined in each case from the above relationships. These values are shown in Table 5.5. The results shown in Table 5.5, enabled a study of the influence of the initial moisture content on the balancing swelling pressure at zero vertical expansion. The variation of the balancing swelling pressure at zero vertical expansion, with the initial moisture content is shown in Figs. 5.61 and 5.62.

In Table 5.5, two types of balancing swelling pressure are shown. The balancing swelling pressure, when $\Delta Hi/H$ is zero, is considered to be the Optimum swelling pressure, i.e. the swelling pressure, developed in the soil, when no vertical expansion is allowed. The balancing swelling pressure when $\Delta H_T/H$ is zero, was evaluated for each condition of initial moisture content and initial dry density to distinguish the influence of the initial consolidation of the specimen, under heavy applied loads.

Table 5.4

Results of Indirect Method Test Series A-1

Tests on samples of Black Cotton Soil.

Initial Dry Density = 1.2g/cc

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
1-1	¼	22.9	46.4	51	86	8.0	8.0
1-2	½	22.9	43.5	51	91	4.0	4.0
1-3	1	22.9	38	51	83	0.90	0.4
1-4	2	22.9		51		No expansion	
Initial Dry Density = 1.33g/cc							
2-1	½	23.2	40.7	61	93	7.3	7.3
2-2i	1	22.6	32.9	60	84	5.3	5.1
2-2ii	1	23.1	36.9	58	86	5.4	5.1
2-3i	2	22.6	32.9	60	84	2.4	1.4
2-3ii	2	21.1	31.9	56	81	2.4	1.8
2-4	3	23.8	31.3	63	82	1.2	0.45
2-5	4	22.3	31	59	81	0.4	
2-6	5	23.8	26.5	63	72	0.14	
2-7	7	22.3				No Expansion	

Initial Dry Density = 1.4g/cc

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
3-1	½	23.9	35.9	70	88	9.6	9.5
3-2	1	23.2	34.2	68	87	7.1	6.8
3-3	2	23.4	32.8	67	88	4.3	3.9
3-4i	3	23.8	31.9	70	89	2.8	2.3
3-4ii	3	22.8	29.2	71	86	3.1	2.2
3-5i	4	23.9	32	70	92	1.6	0.6
3-5ii	4	22.8	29.5	69	86	2.9	2.1
3-6i	5	22.8	30.1	69.0	82	1.7	1.1
3-6ii	5	23.0		69		1.2	0
3-7i	6	23.9	28	69	82	0.1	
3-7ii	6	22.8	29.9	68	90	1.3	0.2
3-8	7	24.0				0.5	
3-9	11	22.8				No Expansion	

Initial Dry Density = 1.5g/cc

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
4-1i	1	23.9	32.5	83	96	7.2	6.9
4-1ii	1	23.3	32	80	87	8.8	8.5
4-1iii	1	22.8	30.5	79	87	8.8	8.7
4-1iv	1	22.8				8.8	8.8
4-2i	2	23.3	30.9	83	92	7.0	6.5

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
4-2ii	2	22.8	30.8	80	91	7.9	7.5
4-3i	3	23.0	32.5	81	99	6.7	6.1
4-3ii	3	22.8	31.1	80	86	6.4	5.9
4-4i	4	23.7	29.6	83	95	4.6	3.6
4-4ii	4	22.8	30	80	94	5.8	5.1
4-4iii	4	23.0				4.7	3.9
4-5i	6	23.7	30.6	83	100	2.8	1.8
4-5ii	6	22.8	29.6	80	97.4	4.3	3.3
4-6	7	22.8	29.6	80	96	2.6	1.8
4-7	8	23.3	29.2	83	89	2.3	0.9
4-8	10	22.9	29.6	80	89	1.5	

Initial Dry Density = 1.17 g/cc

5-1	0.1	27.3	49.2	53	85	5.7	5.7
5-2i	¼	27.5	33.4	54	65	3.5	3.5
5-2ii	¼	27.5	45.5	54	84	3.5	3.5
5-3	½	26.9	44.8	58	91	2.4	1.83
5-4i	1	27	40	58	87	0.43	
5-4ii	1	26.5	43.20	57	95	0.43	
5-5i	1½	27.5		57		No Expansion	
5-5ii	1½	26.5		57		No Expansion	

Initial Dry Density = 1.30 g/cc

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_1/H$ %
6-1	0.1	26.7	41	70	86	9.0	9.0
6-2	¼	27	37.1	72	82	8.1	8.1
6-3	½	27.4	37.0	70	86	6.0	6.0
6-4i	1	27.45	36.5	71	88	4.1	3.5
6-4ii	1	26.2	37	68	90	4.1	3.6
6-5i	2	27.45	34.1	71	87	1.4	0.5
6-5ii	2	27.00	33.1	72	86	0.80	0.3
6-6	3	26.7	33.7	70	87	0.32	0
6-7i	4	27.0	34.5	72	92	0.27	
6-7ii	4	26.0	33.2	68	88	0.27	
6-8	5	27.3				No Expansion	
6-9	6	26.3				No Expansion	

Initial Dry Density = 1.4 g/cc

7-1	0.1	27.6	35.8	82	88	10.5	10.5
7-2	½	27.6	33.2	82	88	6.5	6.5
7-3i	1	26.2	34	79	94	5.00	4.1
7-3ii	1	26.7	34	80	93	4.4	4.4
7-4i	2	26.3	30.8	79	88	3.4	2.7
7-5i	4	26.6	31.1	80	92	1.6	0.90

Test No.	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
7-5ii	4	27.1	30.8	80	92	1.4	0.85
7-6i	6	25.3	30.5	78	93	0.67	
7-6ii	6	26.6	29.1	78	90	0.50	
7-7i	8	25.3	30.2	78	94	0.30	
7-7ii	8	26.5	30	80	94	0.22	
7-8i	10	27.4				0.15	
7-8ii	10					No Expansion	
7-9	12	27.5				No Expansion	

Initial Dry Density = 1.5 g/cc

Test No	Load t/sq ft	Wi %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
8-1	0.10	27.3	11.0	11
8-2	½	27.3	7.5	7.5
8-3i	1	26.8	5.8	5.8
8-3ii	1	27.3	5.8	5.7
8-4i	2	27.3	4.6	4.2
8-4ii	2	27.3	4.2	4.0
8-4iii	2	27.1	4.2	4.0
8-4iv	2	27.3	4.4	4.1
8-4v	2	27.3	4.6	4.3

Test No	Load t/sq ft	Wi %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
8-5i	4	27.3	2.8	2.3
8-5ii	4	27.2	2.7	2.1
8-5iii	4	27.3	0.93	1.0
8-6i	6	26.4	1.87	1.0
8-6ii	6	27.2	1.80	0.9
8-7	8	27.3	1.0	
8-8	11.3	27.3	0.60	

Initial Dry Density = 1.2 g/cc

Test No	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
9-1	¼	29.2	46.4	64	94	48	4.8
9-2	½	29.2		64		1.3	1.1
9-3	1	29.4		64		No Expansion	
9-4	2	29.2		64		No Expansion	

Initial Dry Density = 1.33 g/cc

10-1i	¼	29.5	37.7	79	91	6.6	6.55
10-1ii	¼	29.5	39	79	92	7.1	7.05
10-2	½	28.6	36.1	79	93	4.5	4.2
10-3i	1	29.5	38.9	79	97	3.5	3.5

Test No	Load t/sq ft	Wi %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
10-3ii	1	29.4	38.9	79	89	3.6	3.4
10-4	2	29.5	37	79	97	1.9	1.4
10-5	3	29.5	34.2	79	93	0.4	
10-6	5	29.5				No Expansion	

Initial Dry Density = 1.4 g/cc

11-1	¼	29.6	33.2	88		8.2	8.2
11-2	½	29.6		88		5.5	5.5
11-3	1	29.6	34.8	88	94	5.1	4.9
11-4	2	29.6	31.4	88	91	3.30	2.6
11-5	3	29.4	32.7	87	96	2.2	1.53
11-6	4	29.4	33	87	98	1.3	0.3
11-7	5	29.2	32.8	86	98	0.7	

Initial Dry Density = 1.2 g/cc

12-1	0.1	32.6	44.9	71	94	3.2	3.2
12-2	¼	32.5	42.5	71	90	2.23	2.1
12-3	½	32.5	42	71	92	0.40	0.20
12-4	1	32.5				No Expansion	

Initial Dry Density = 1.33 g/cc

Test No	Load T/sq.ft	Ei %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
13-1	¼	32.5	38.6	87	93	5.00	5.0
13-2	½	32.5	35	87	88	3.1	3.1
13-3	1	32.5		87		2.2	2.1
13-4	2	32.5		87		1.1	0.90

Initial Dry Density = 1.4 g/cc

Test No	Load T/sq.ft	Ei %	Wf %	Si %	Sf %	$\Delta H_T/H$ %	$\Delta H_i/H$ %
14-1	¼	32.8	32.4			7.0	7.0
14-2	½	32.0	38.0			3.7	3.5
14-3	1½	32.3	34.8			2.3	2.3
14-4i	2	32.7	32.2			1.4	1.23
14-4ii	2	32.3	34.8			1.6	1.01
14-5	3	32.7	33.4			0.26	

TABLE 5.5

SUMMARY OF RESULTS

Indirect Method Test Series A-1

Tests on disturbed samples of Black Cotton Soil

Test	Initial Moisture Content (%)	Initial dry Density (g/cc.)	Balancing swelling pressure when $\frac{\Delta H_T}{H}$ is zero (t/sq.ft.)	Balancing* swelling pressure when $\frac{\Delta Hi}{H}$ is zero (t/sq.ft.)
1.	23	1.2	1.9	1.2
2.	23	1.33	6.5	3.4
3.	23	1.4	13.5	5.4
4.	23	1.5	28	10.4
5.	27.3	1.17	1.2	.75
6.	27.3	1.30	4.9	2.9
7.	27.3	1.4	11.8	4.7
8.	27.3	1.5	19	8.4
9.	29.5	1.2	0.6	0.60
10.	29.5	1.33	3.4	2.7
11.	29.5	1.4	7	4.3
12.	32.5	1.2	0.6	0.5
13.	32.5	1.33	2.5	2.5
14.	32.5	1.4	3.25	3.0

* In Table 5.5, balancing swelling pressure when $\Delta Hi/H$ is zero, is considered to be the optimum swelling pressure, i.e. the swelling pressure developed in the soil, when no vertical expansion is allowed.

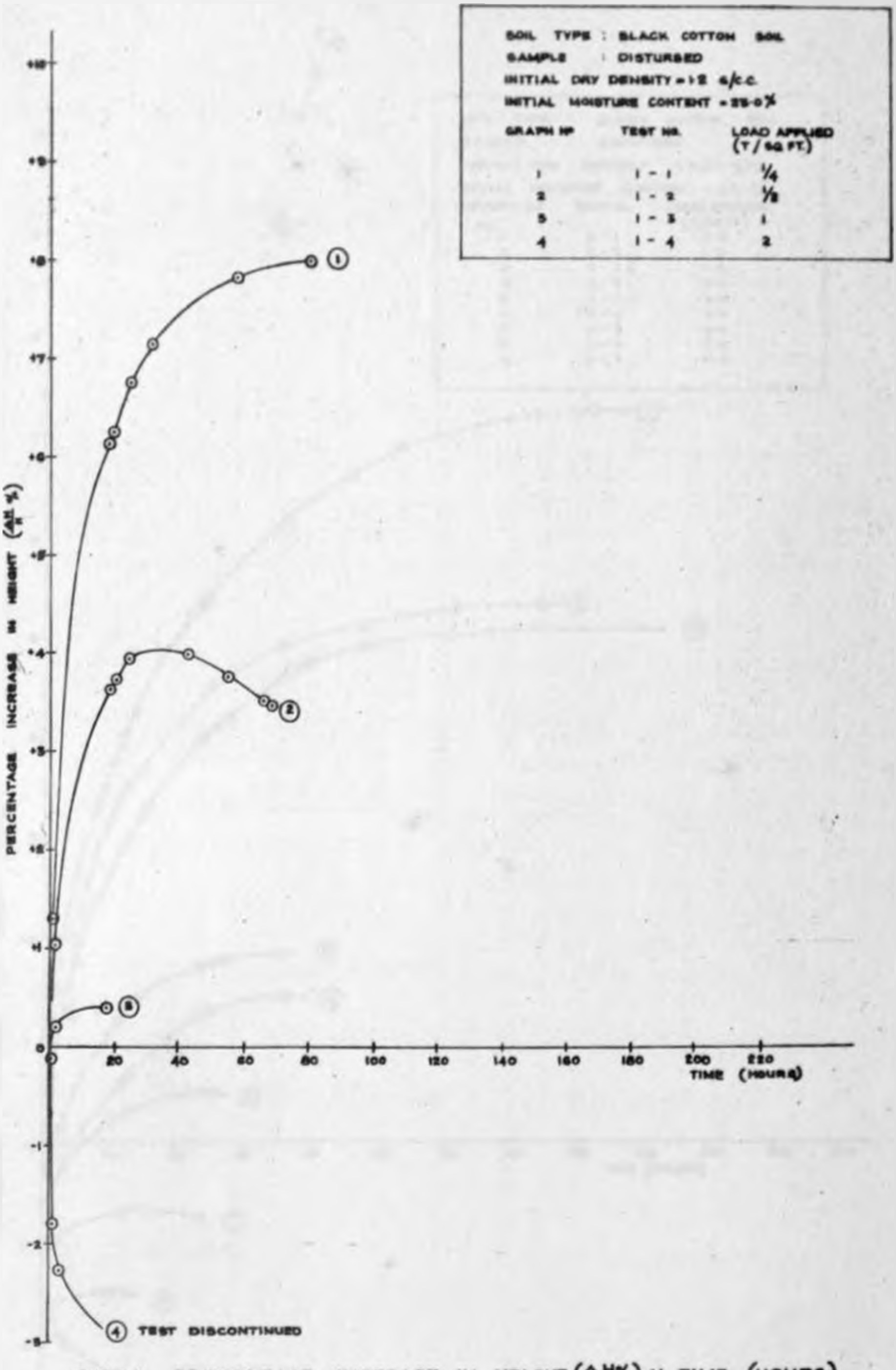

FIG.5.31 PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}$%) V TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

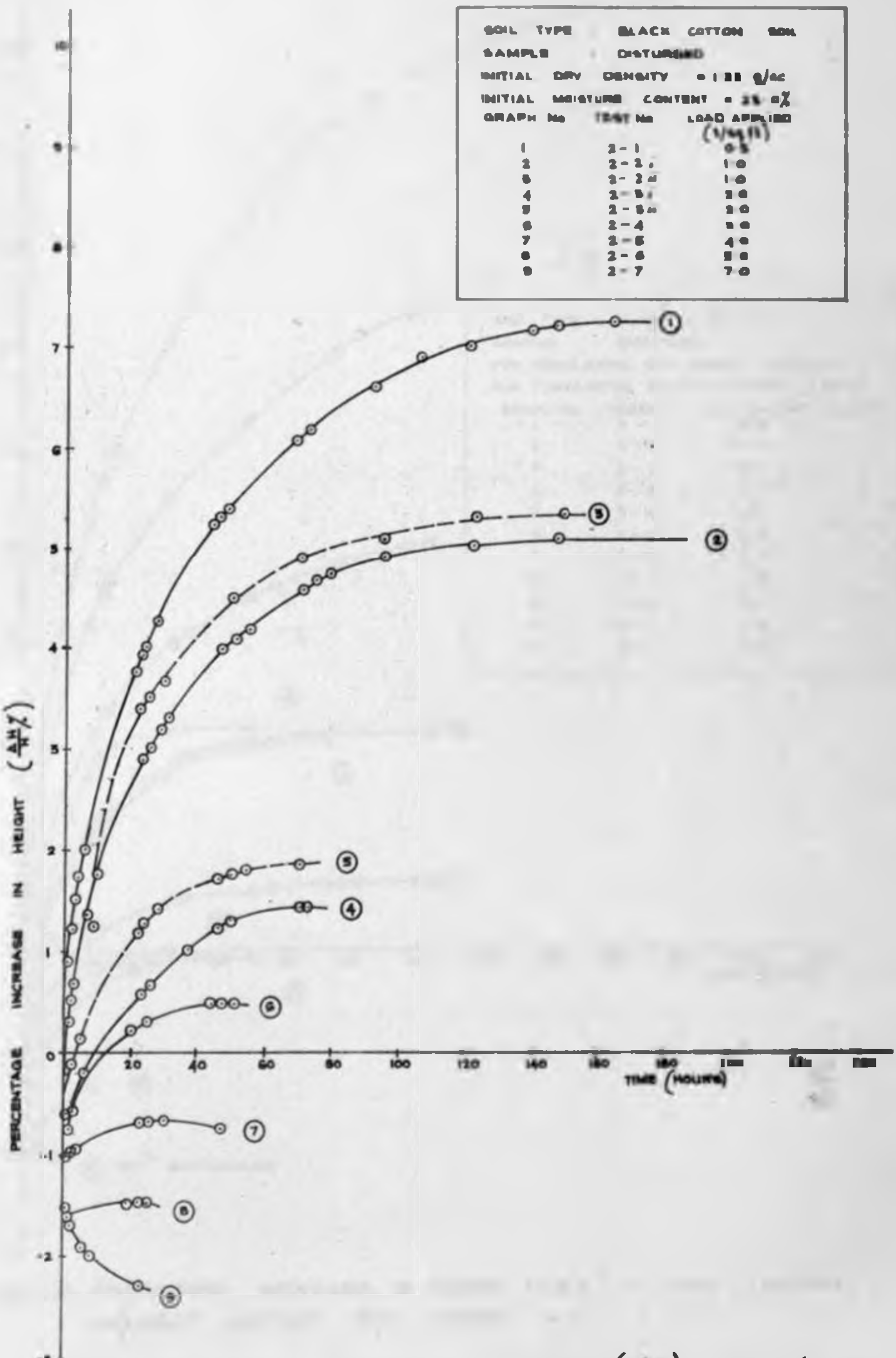

FIG. 5.32. PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H}{H}\%\right)$ v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1

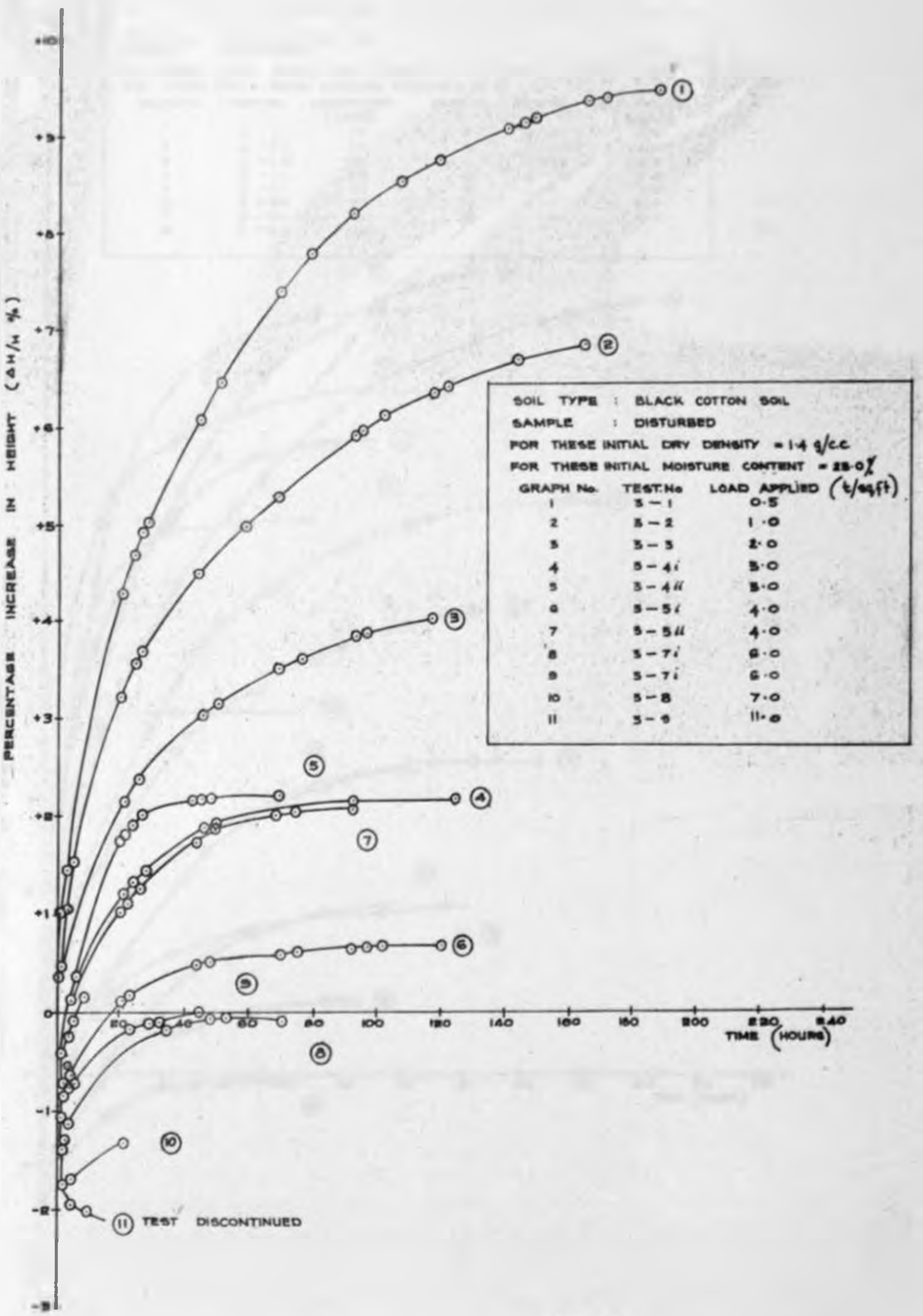

FIG 5.33 PERCENTAGE INCREASE IN HEIGHT (ΔH/H %) V TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

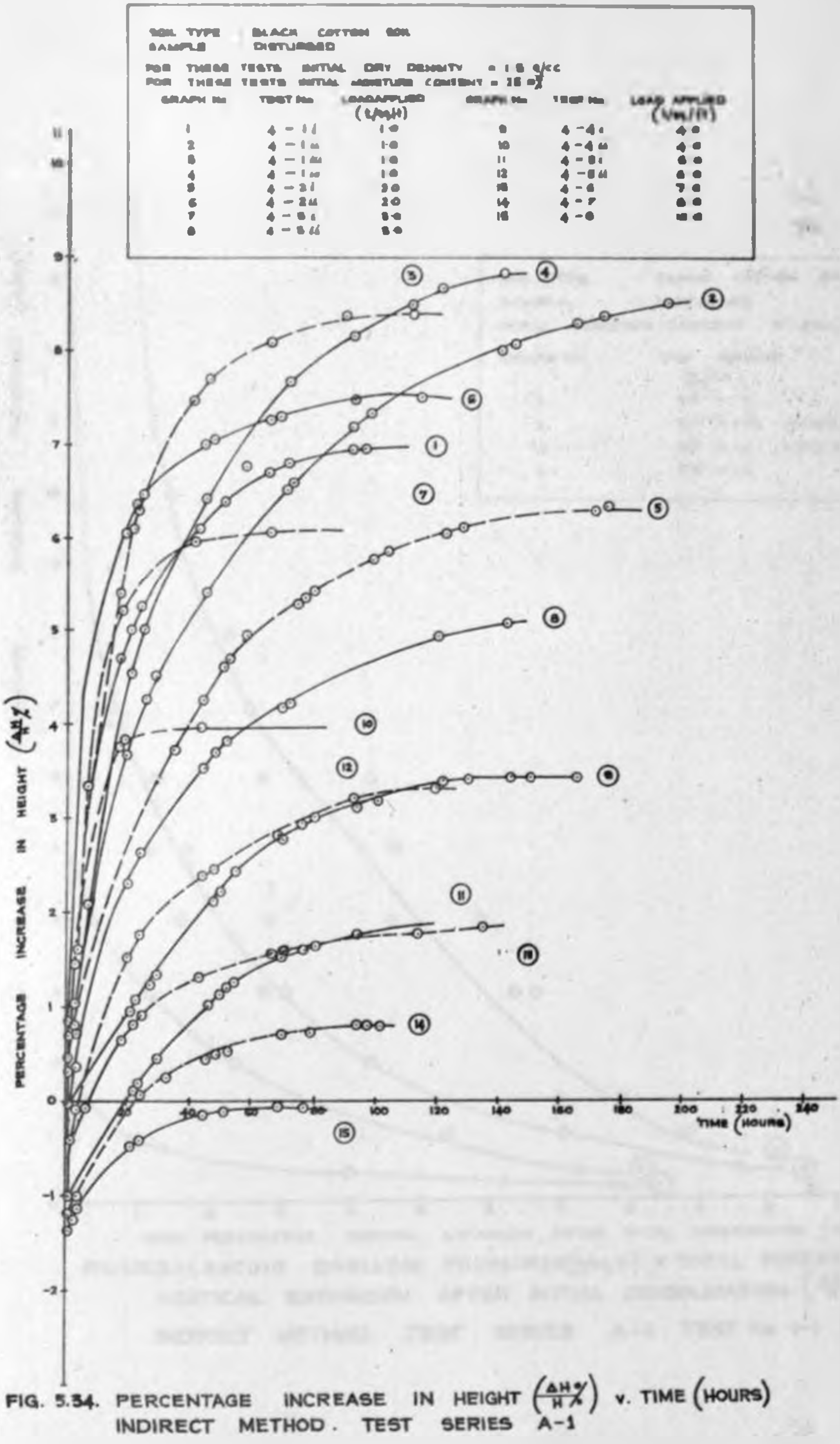

FIG. 5.34. PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H}{H}\%\right)$ v. TIME (HOURS)
INDIRECT METHOD. TEST SERIES A-1

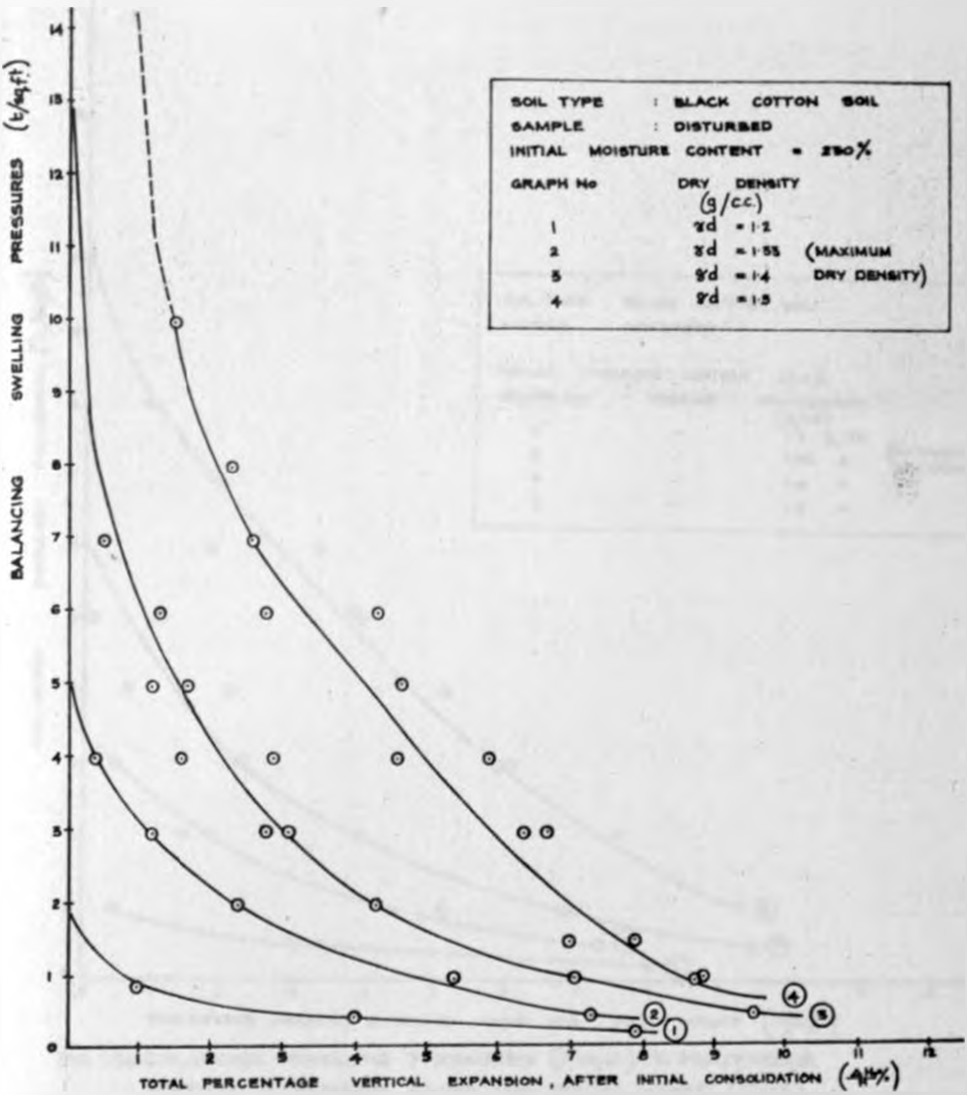

FIG 5.5 BALANCING SWELLING PRESSURES(t/sq.ft) V. TOTAL PERCENTAGE VERTICAL EXPANSION AFTER INITIAL CONSOLIDATION ($\frac{\Delta H}{H}$%) INDIRECT METHOD. TEST SERIES A-1 TEST No 1-1 TO 4-8

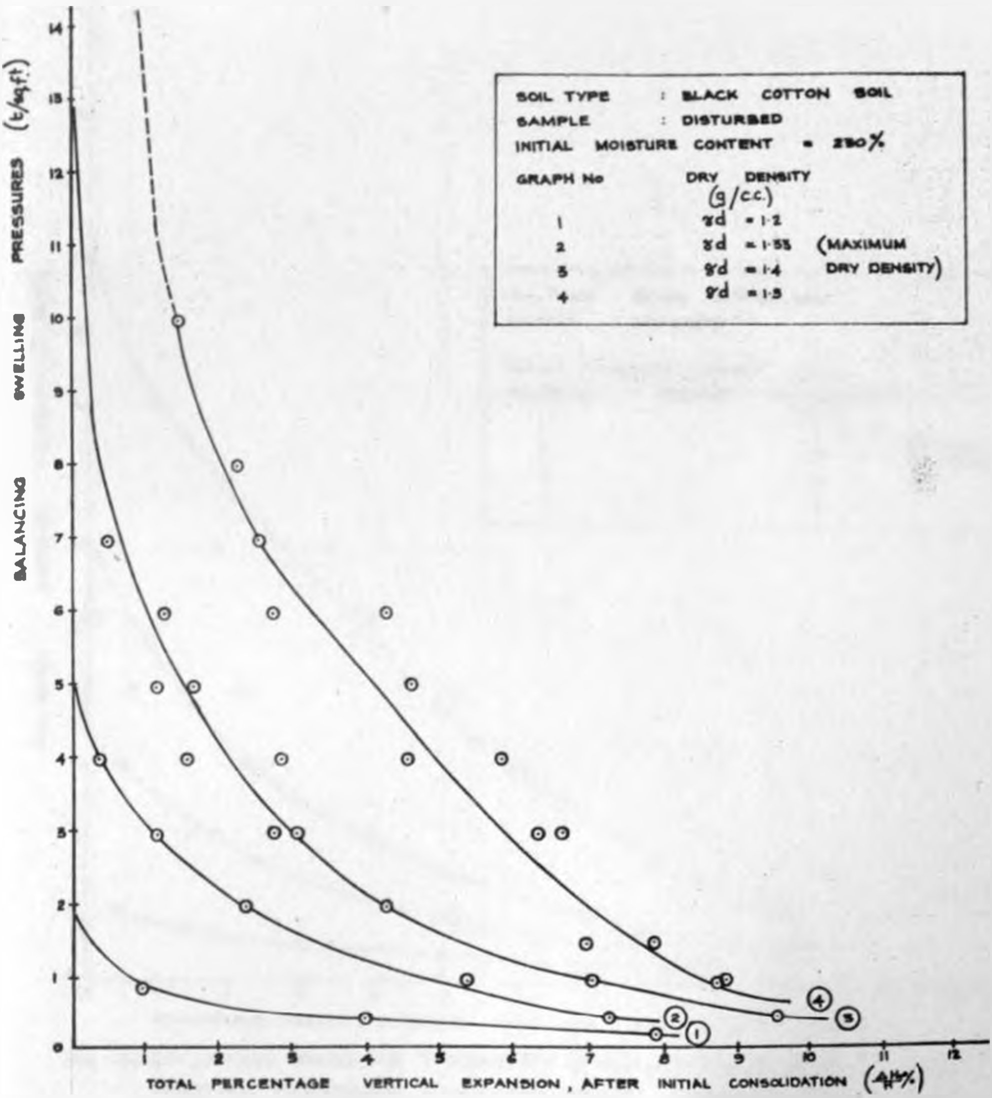

FIG 5.35 BALANCING SWELLING PRESSURES (t/sq ft) V. TOTAL PERCENTAGE VERTICAL EXPANSION AFTER INITIAL CONSOLIDATION (ΔH/H %)
INDIRECT METHOD TEST SERIES A-1 TEST No 1-1 TO 4-8

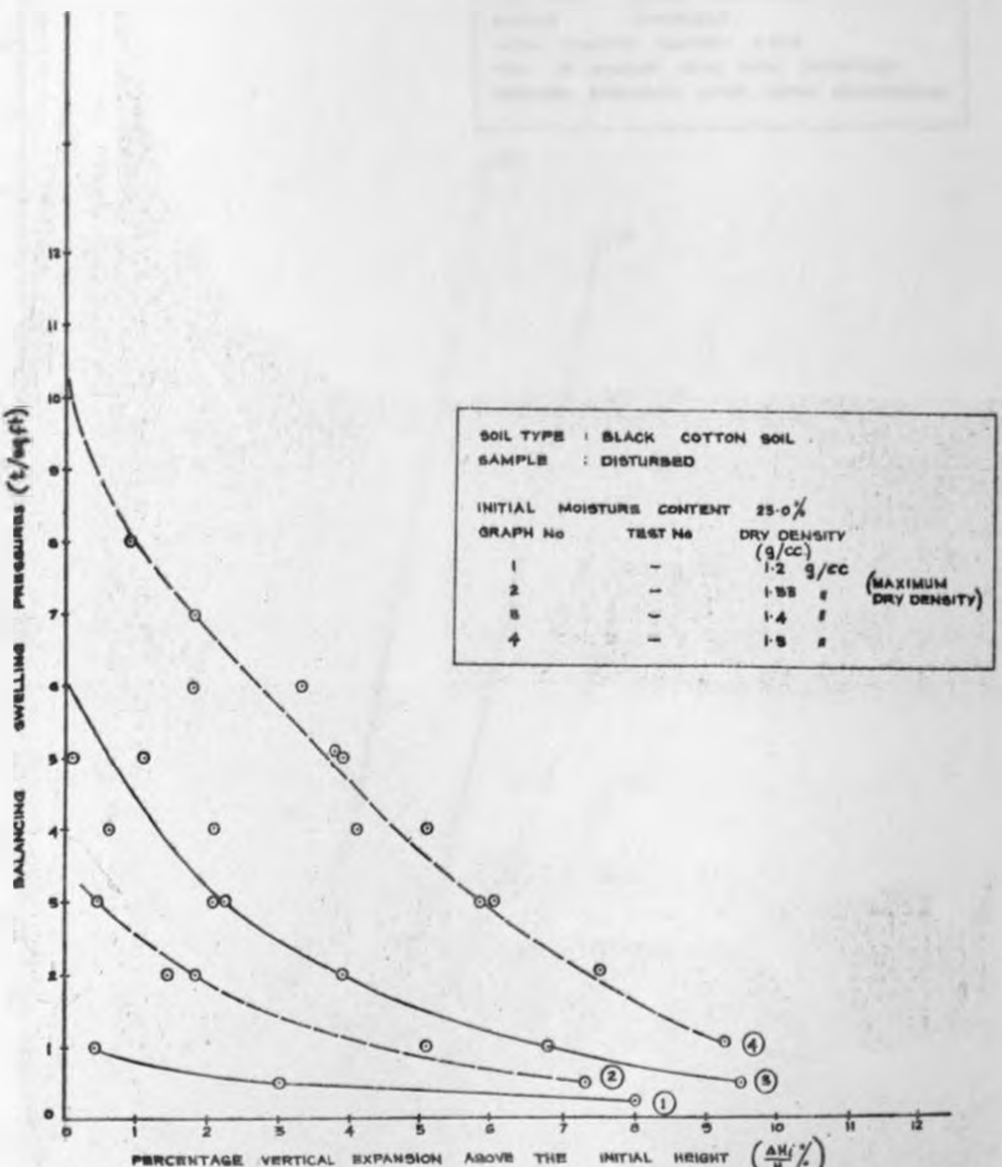

FIG.5.36. BALANCING SWELLING PRESSURES (t/sq ft) V. PERCENTAGE VERTICAL EXPANSION ABOVE THE INITIAL HEIGHT $\left(\frac{\Delta H_i}{H}\%\right)$ INDIRECT METHOD TEST SERIES A-1 TEST No 1-1 TO 4-8

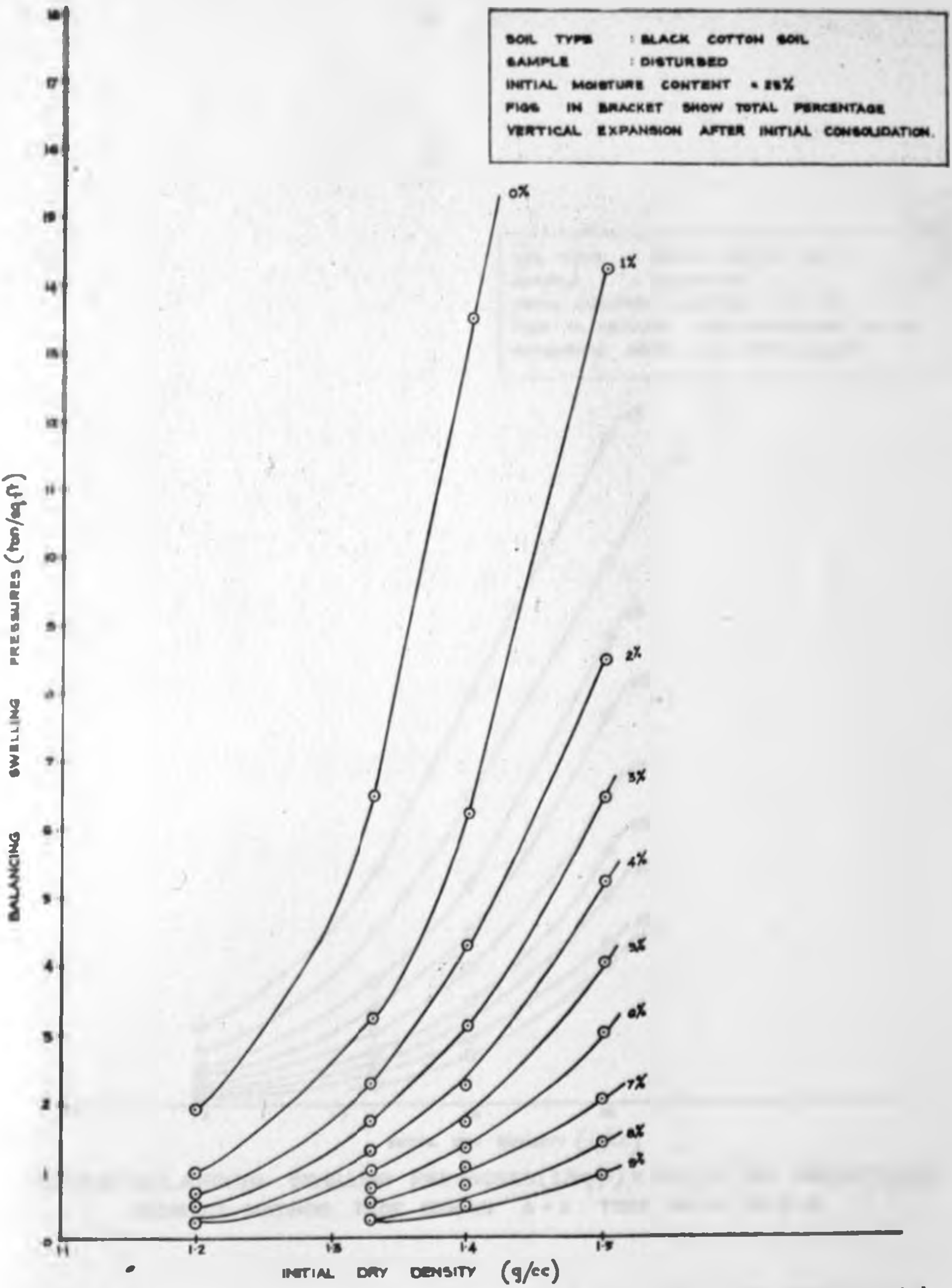

FIG. 5.37 BALANCING SWELLING PRESSURES (t/sq ft) V. INITIAL DRY DENSITY (g/cc)
INDIRECT METHOD TEST SERIES A-1 TEST No 1-1 TO 4-8

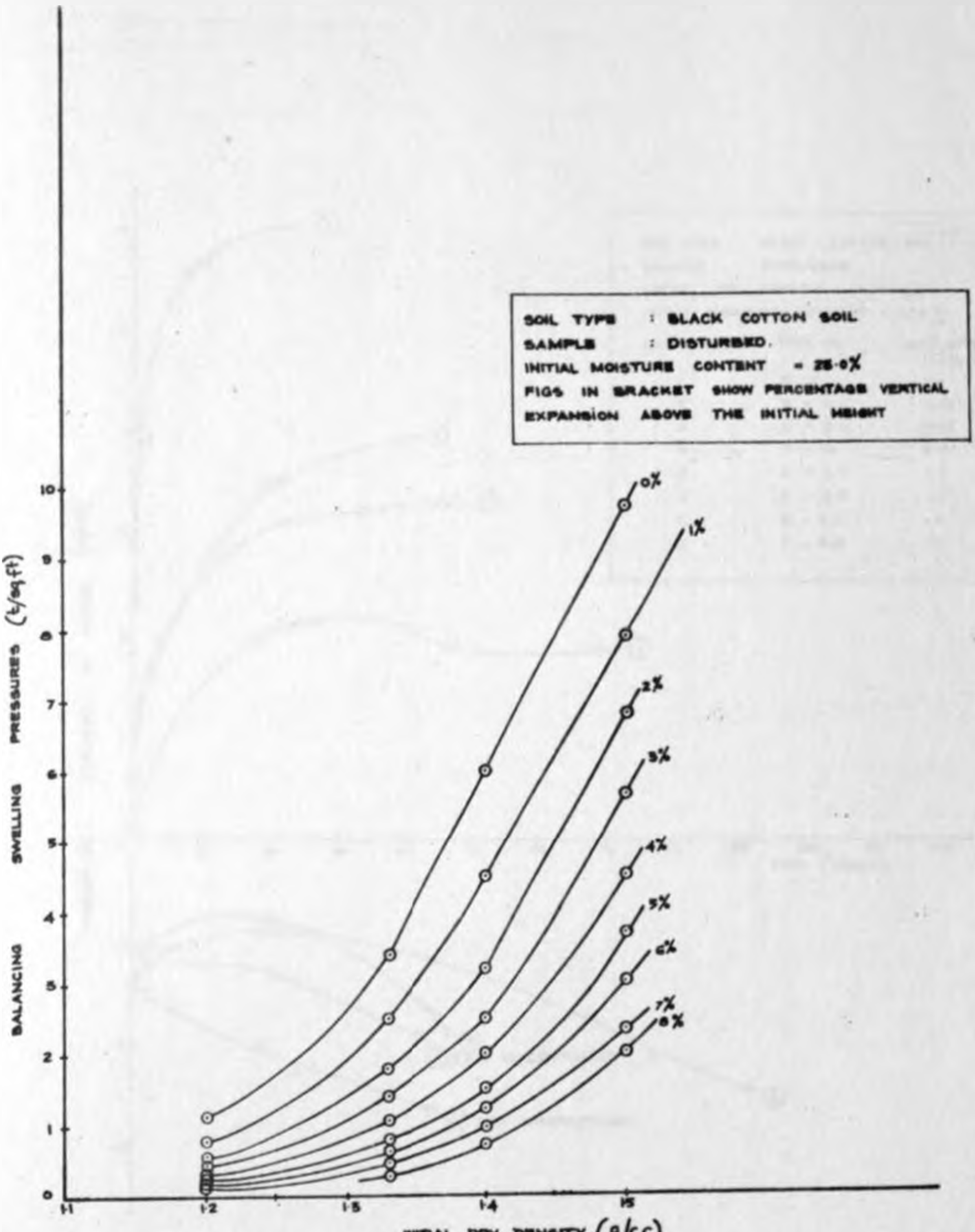

FIG. 5·38. BALANCING SWELLING PRESSURES (t/sq ft) V. INITIAL DRY DENSITY (g/cc)
INDIRECT METHOD TEST SERIES A - 1 TEST No 1-1 TO 4-8

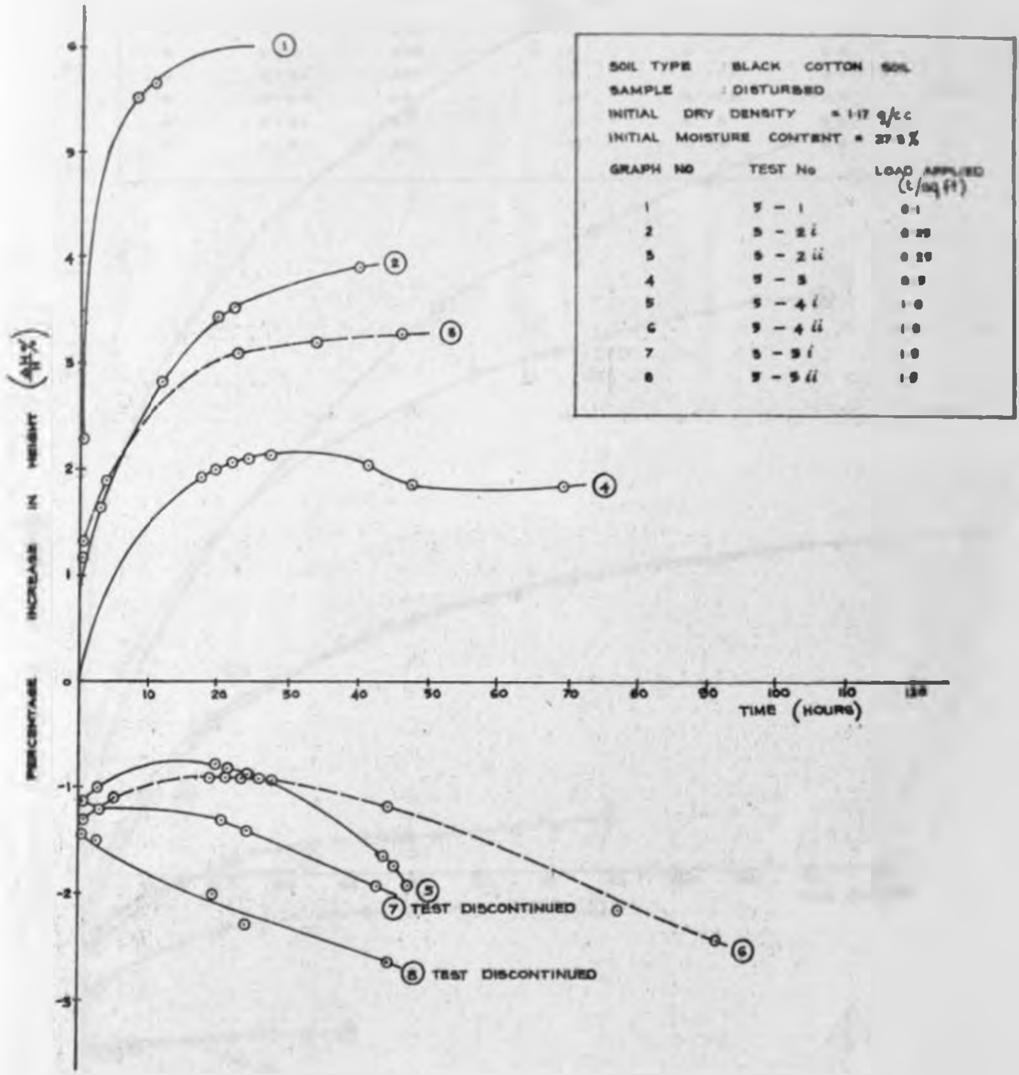

FIG. 5.39 PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}\%$) v. TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

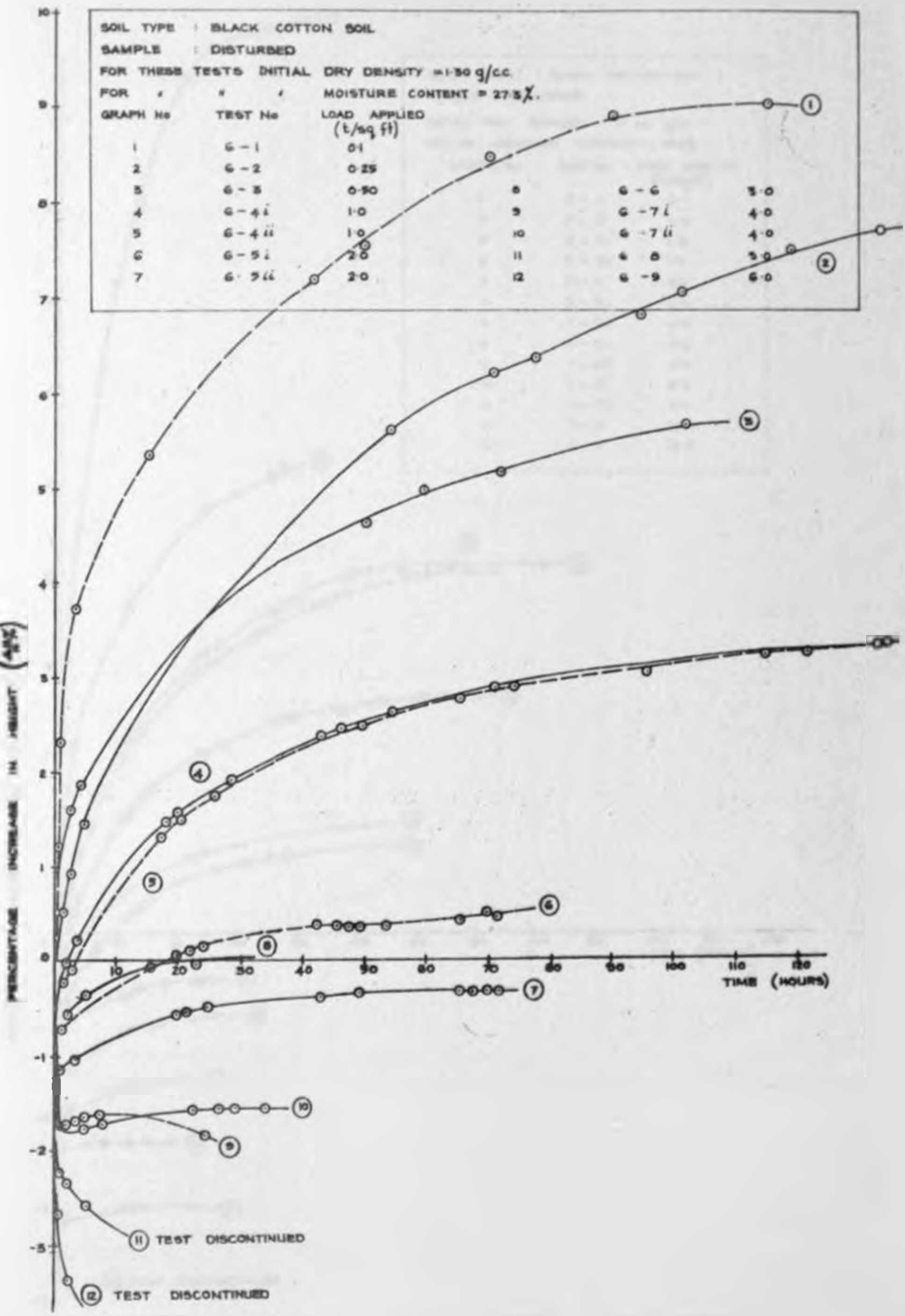

FIG 5 40. PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H}{H}\%\right)$ v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1

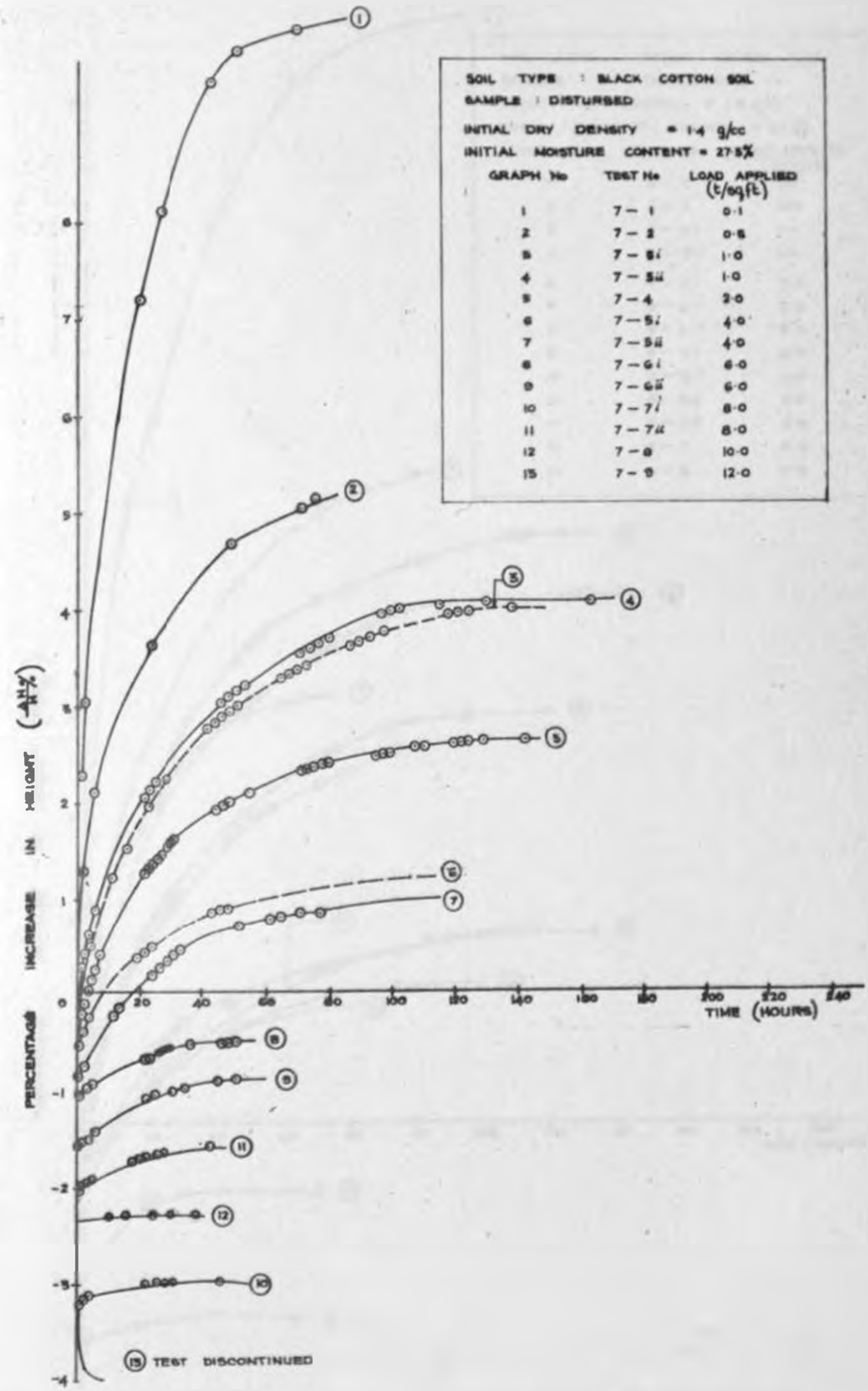

FIG. 5.41. PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}\%$) v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1

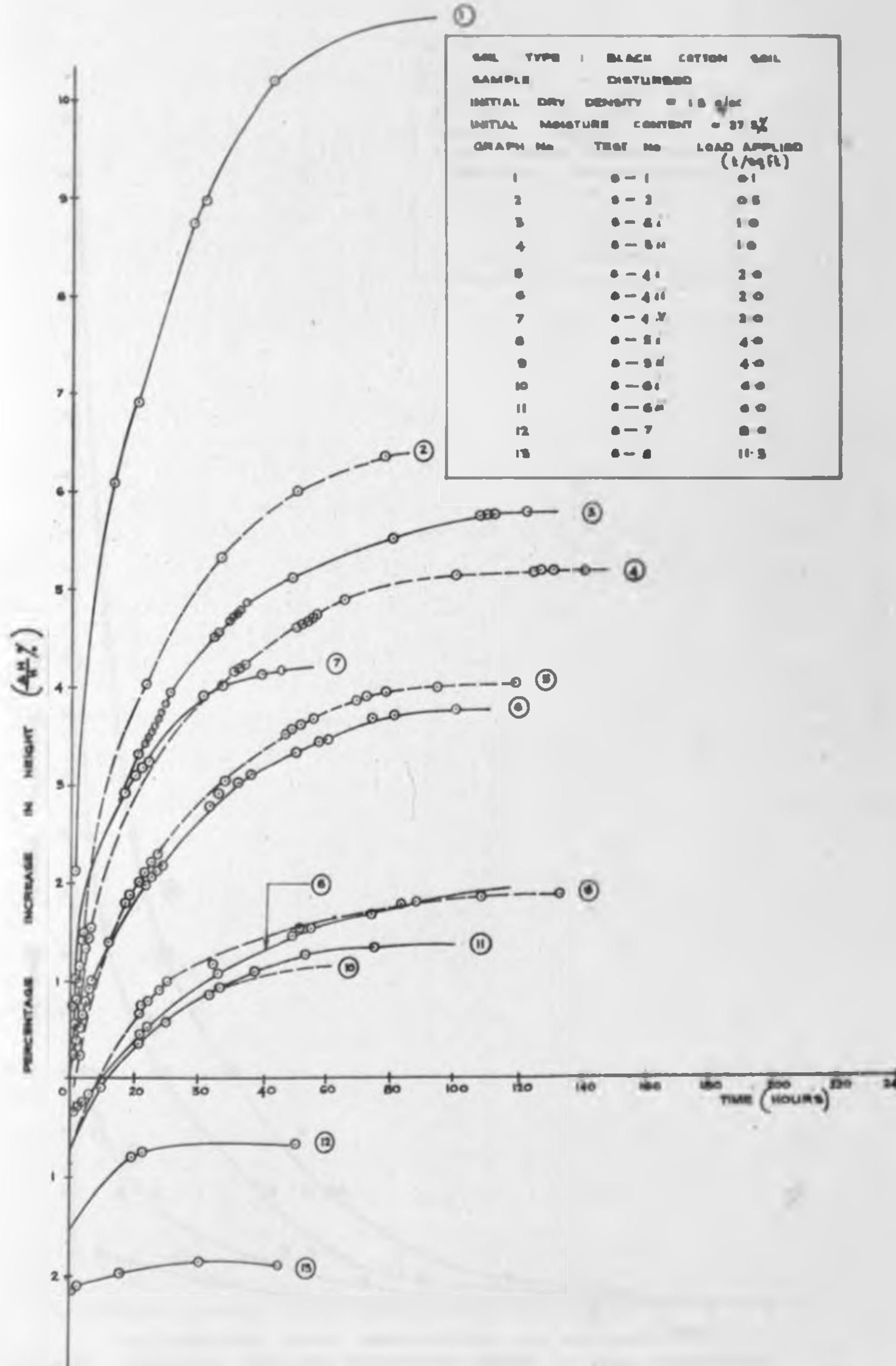

FIG. 5.42 PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H\%}{H\%}\right)$ v. TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

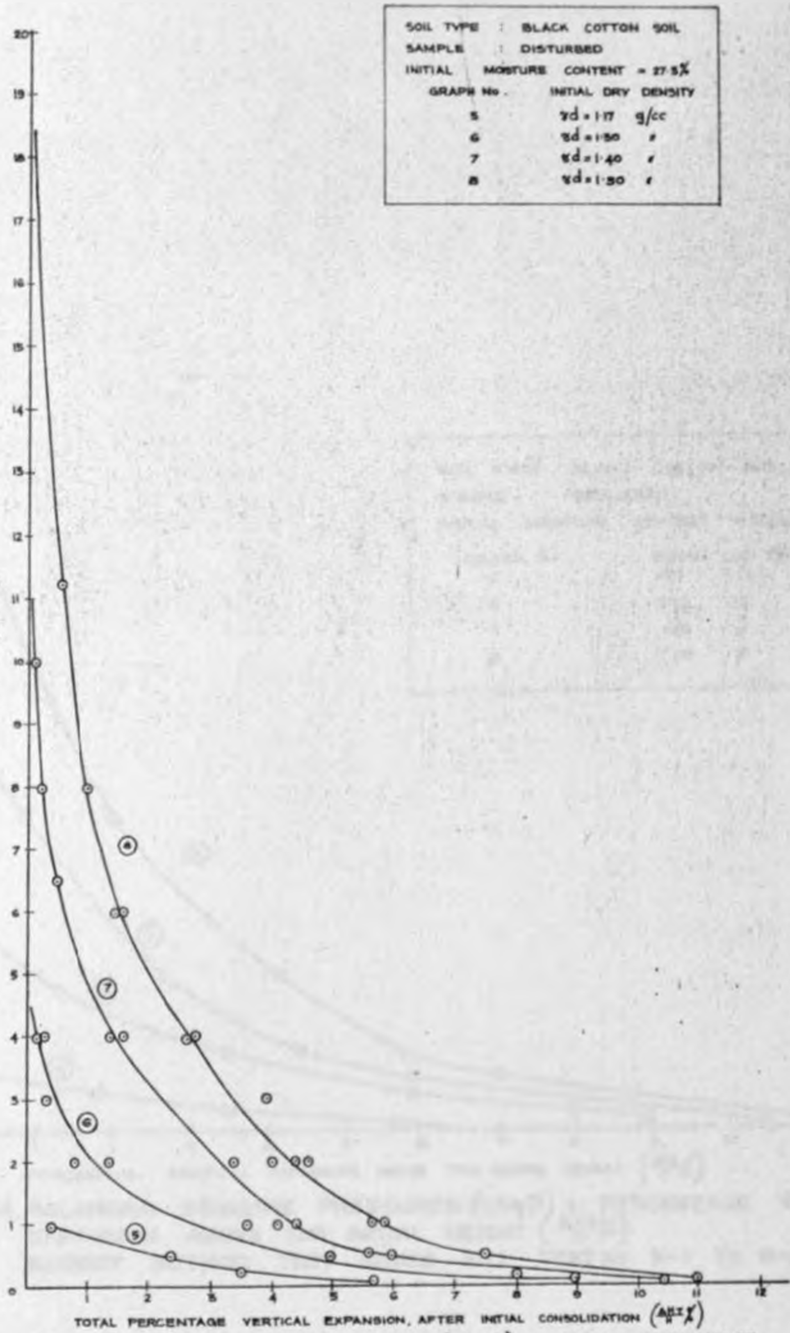

FIG 5 43 BALANCING SWELLING PRESSURES (t/sq ft) v. TOTAL PERCENTAGE VERTICAL EXPANSION AFTER INITIAL CONSOLIDATION $\left(\frac{\Delta H}{H}\%\right)$ INDIRECT METHOD. TEST SERIES A-1 TEST No 5-1 TO 8-8

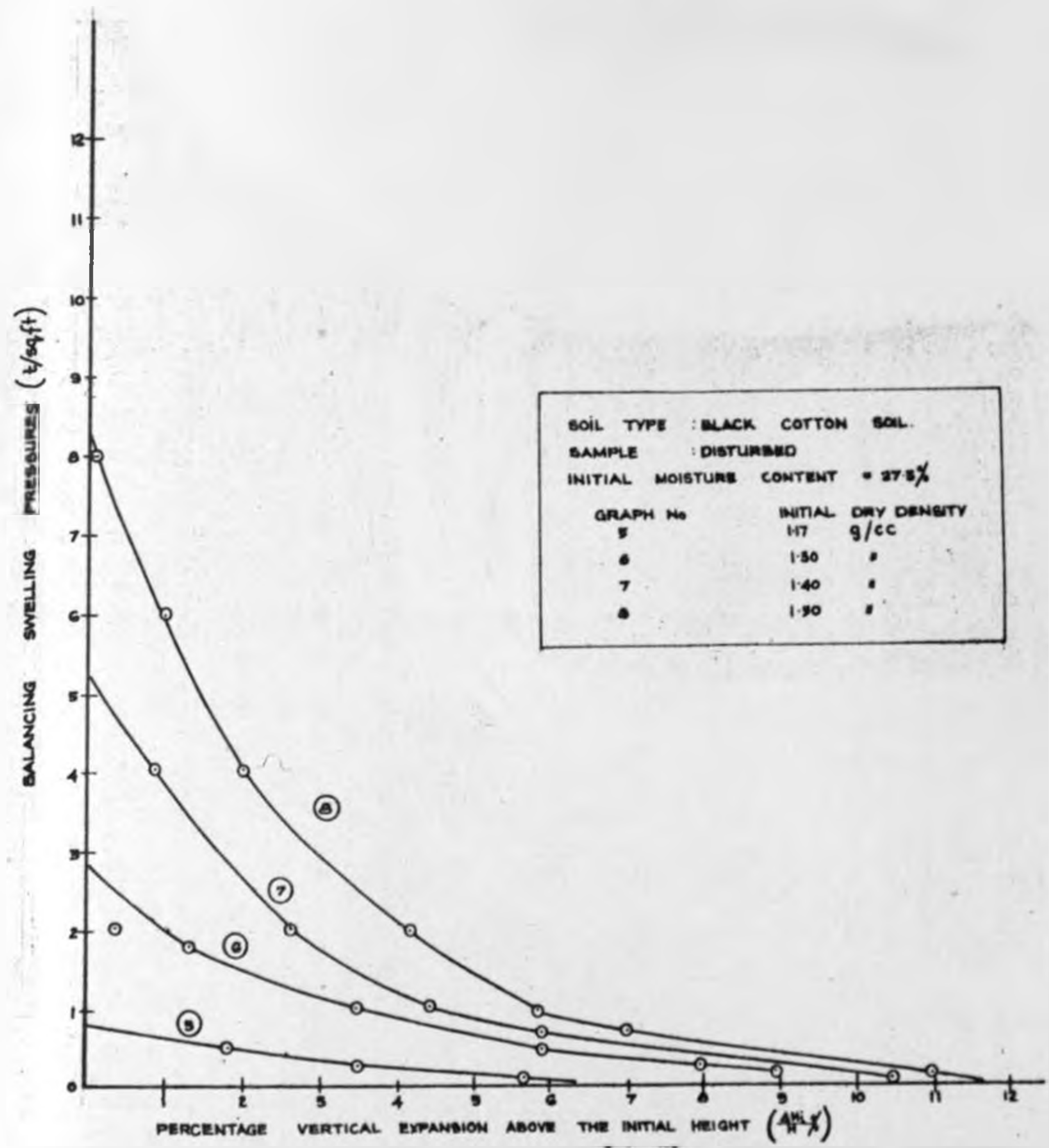

FIG. 5·44. BALANCING SWELLING PRESSURES (t/sqft) v. PERCENTAGE VERTICAL EXPANSION ABOVE THE INITIAL HEIGHT ($\frac{\Delta H}{H}$%)
INDIRECT METHOD TEST SERIES A-1 TEST No 5-1 TO 8-8

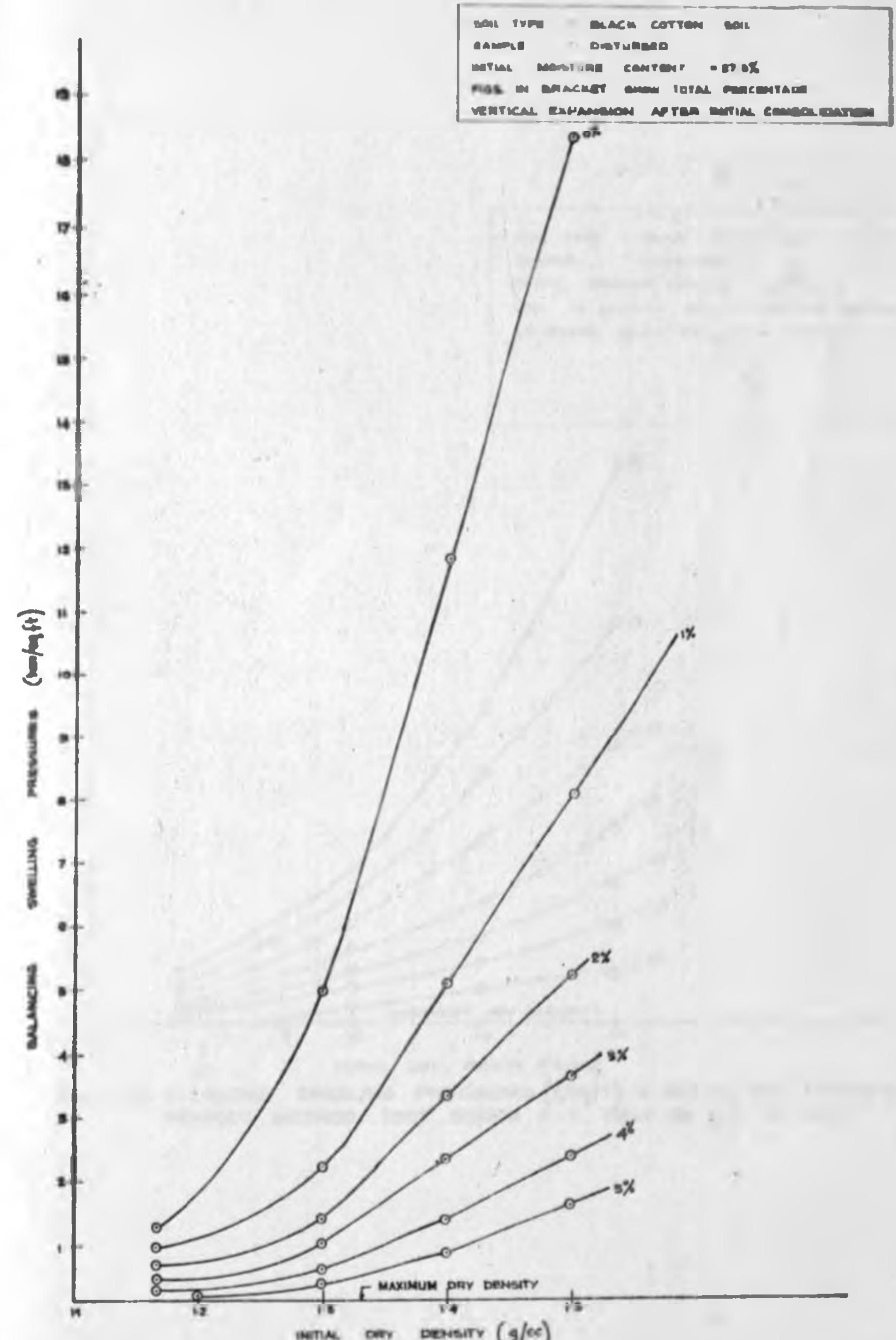

FIG. 5.45. BALANCING SWELLING PRESSURES (t/sq.ft) v. INITIAL DRY DENSITY (g/cc)
INDIRECT METHOD TEST SERIES A-1 TEST No 5-1 TO 6-6

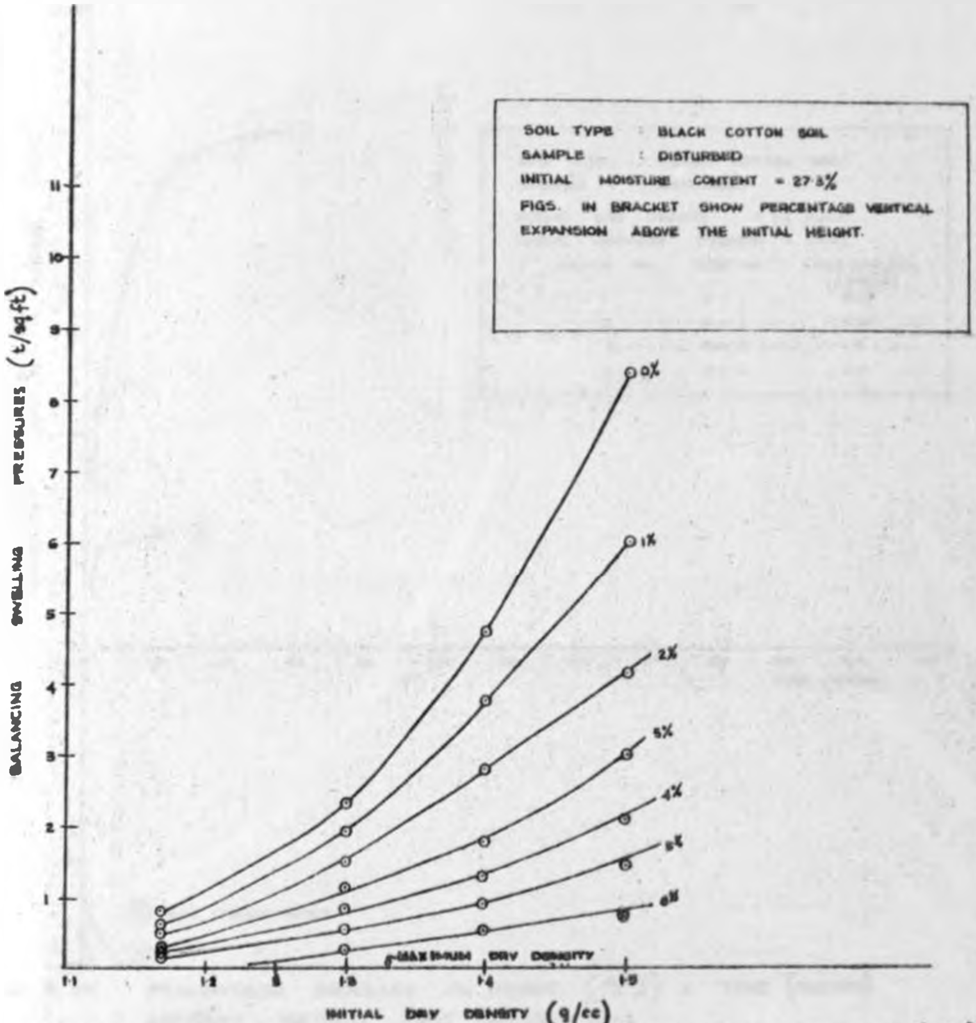

FIG. 5.46. BALANCING SWELLING PRESSURES (t/sqft) v. INITIAL DRY DENSITY (g/cc) INDIRECT METHOD. TEST SERIES A-1. TEST No 5-1 TO 5-6

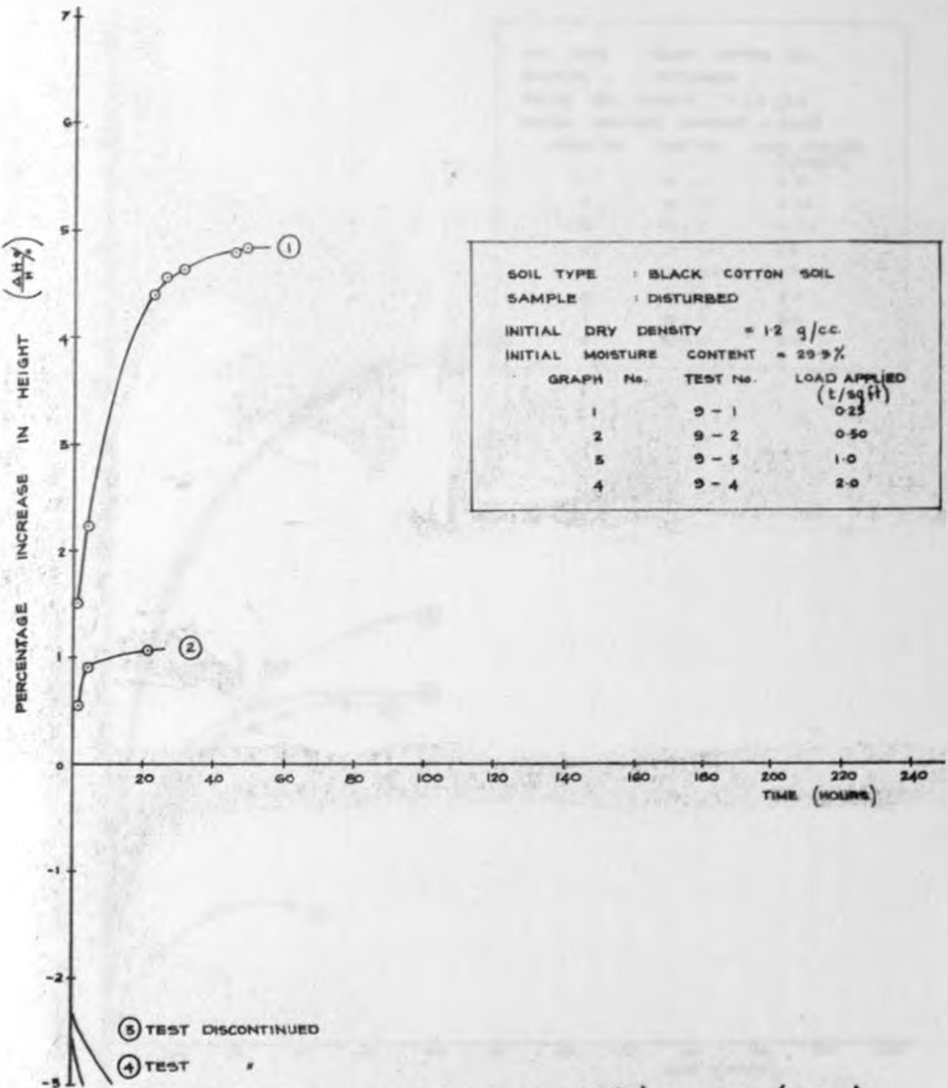

FIG. 5.47. PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}\%$) v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1

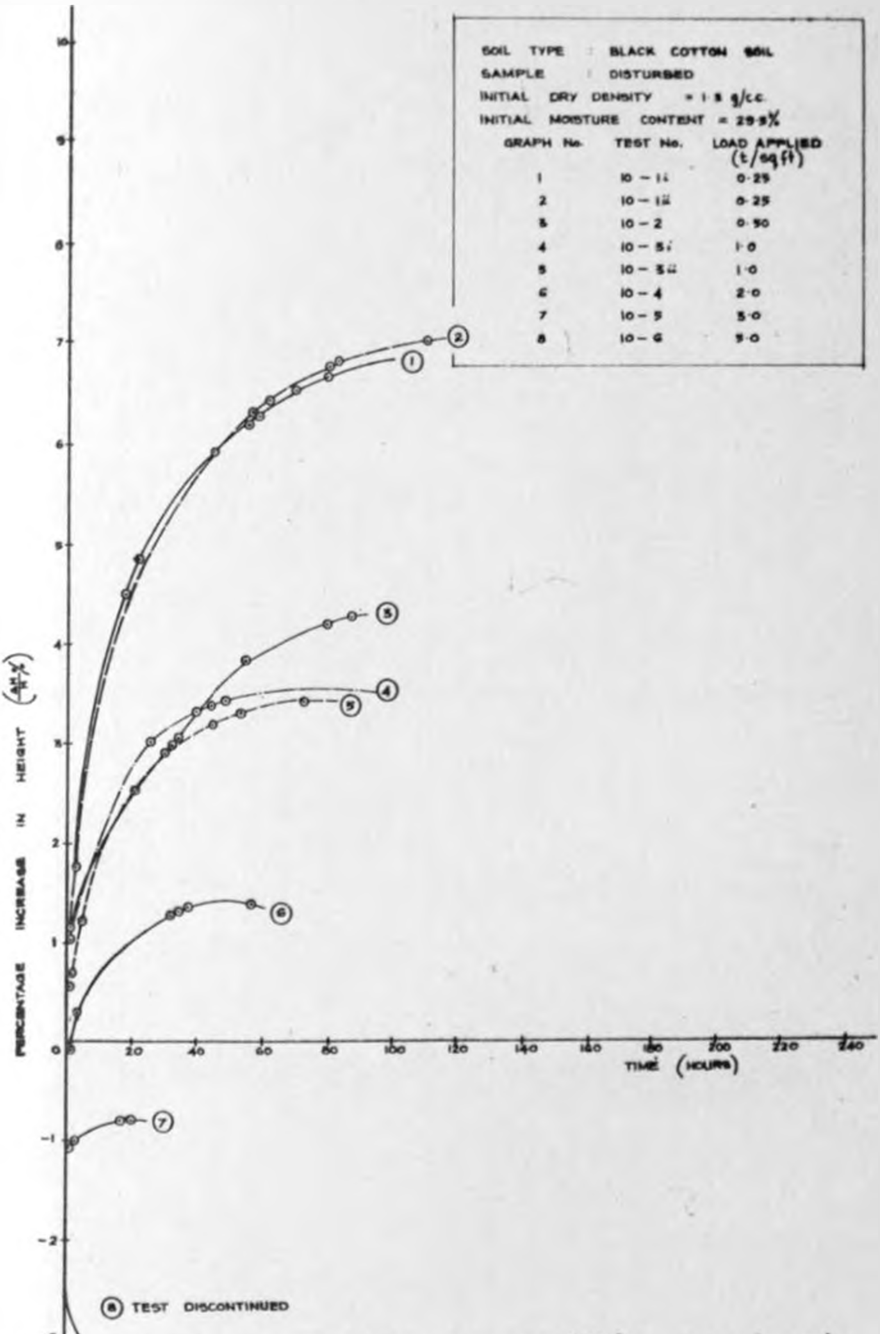

FIG. 5.48. PERCENTAGE INCREASE IN HEIGHT (ΔH/H %) v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1.

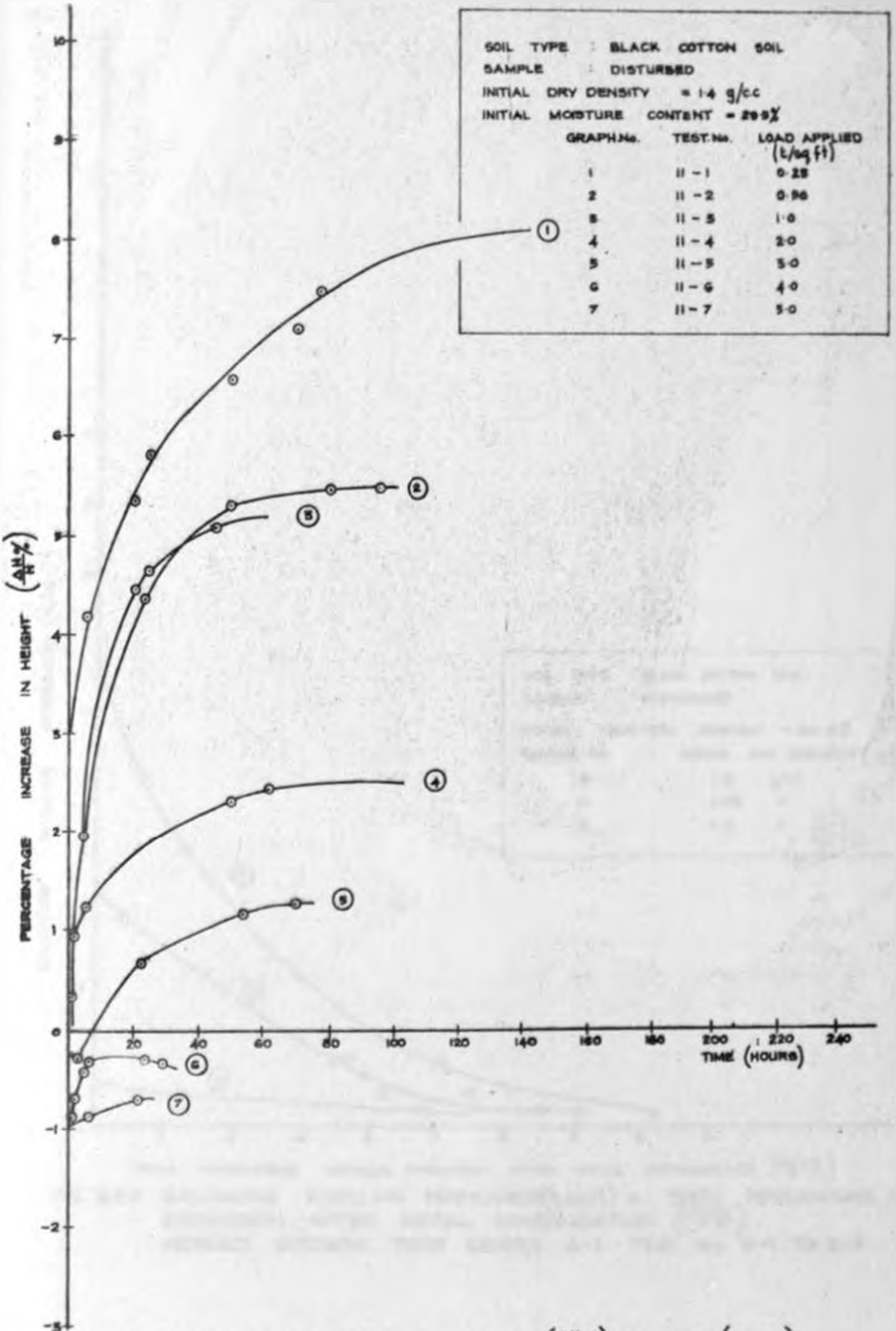

FIG. 5.48. PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H}{H}\%\right)$ v. TIME (HOURS) INDIRECT METHOD. TEST SERIES A-1

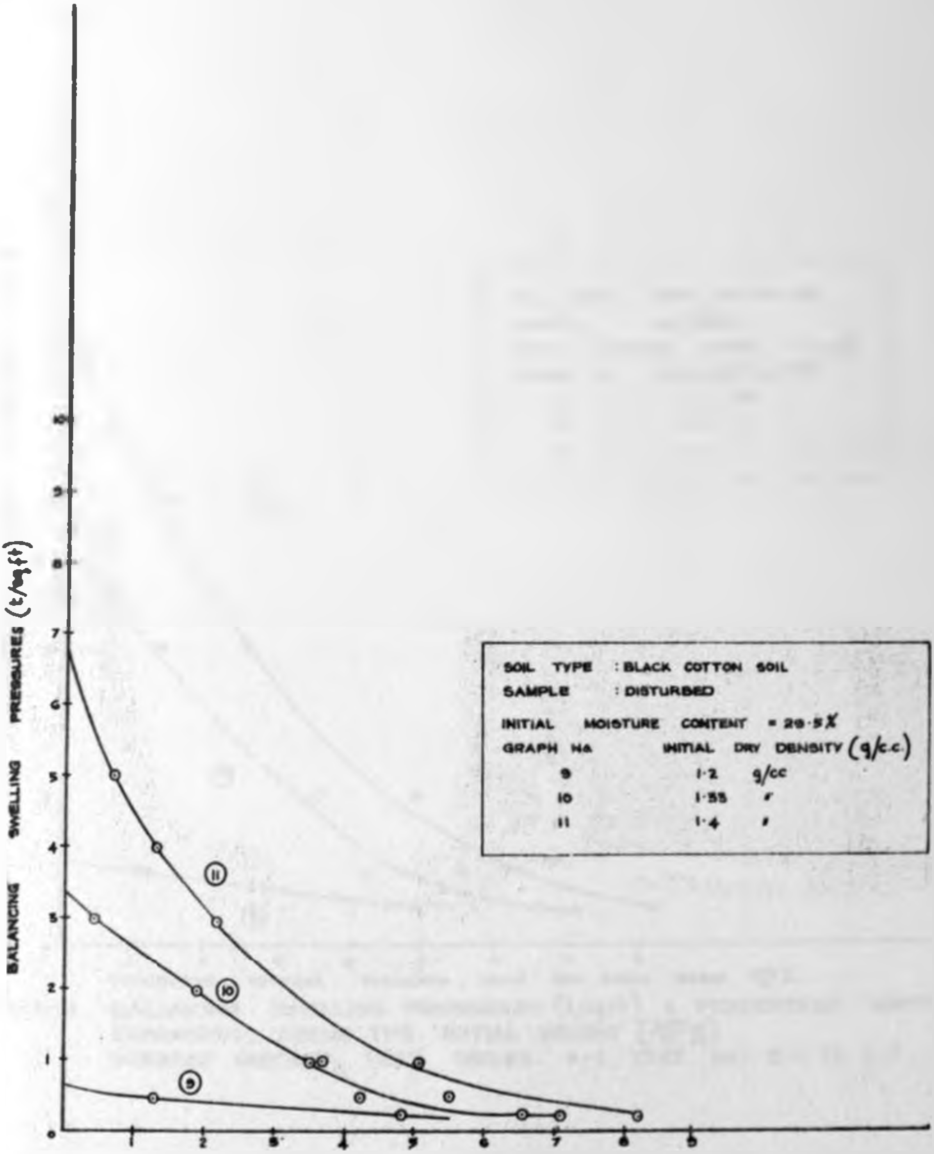

FIG. 5.50. BALANCING SWELLING PRESSURES (t/sq ft) v. TOTAL PERCENTAGE VERTICAL EXPANSION, AFTER INITIAL CONSOLIDATION
INDIRECT METHOD. TEST SERIES A-1 TEST No 9-1 TO 11-7

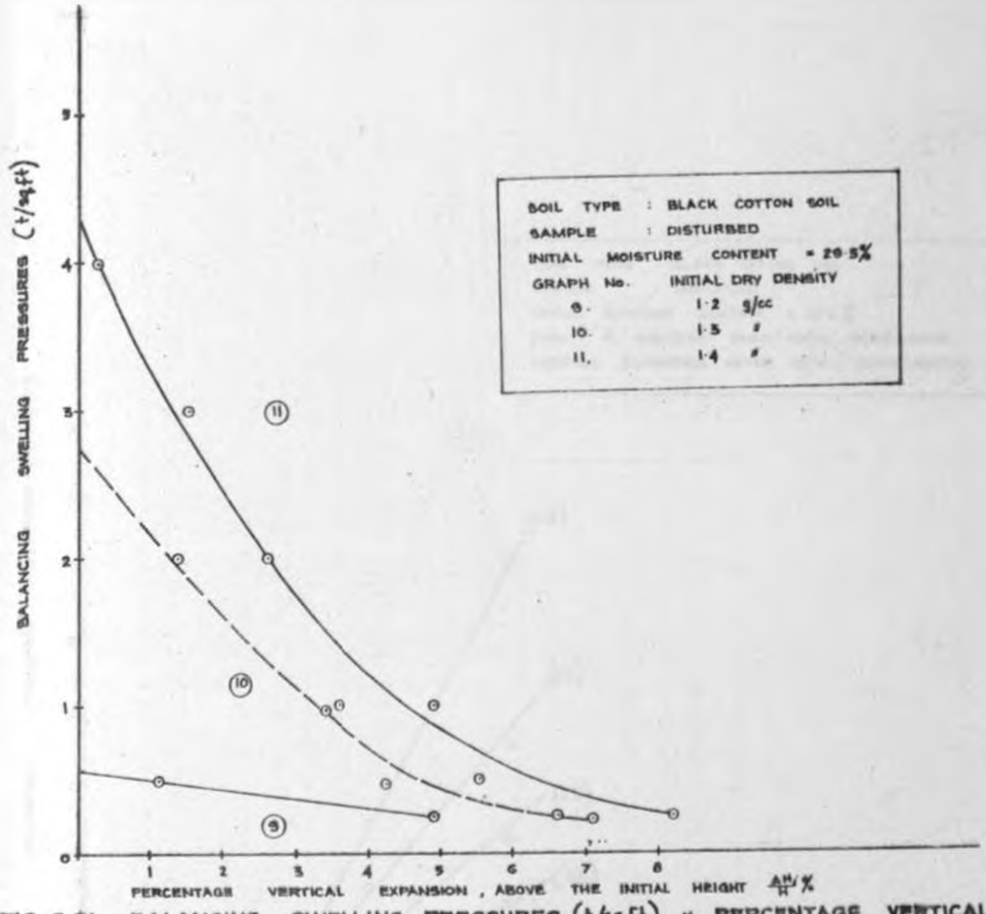

FIG. 5.51 BALANCING SWELLING PRESSURES (t/sq ft) v. PERCENTAGE VERTICAL EXPANSION, ABOVE THE INITIAL HEIGHT $\left(\frac{\Delta Hi}{H}\%\right)$ INDIRECT METHOD TEST SERIES A-1 TEST No. 9-1 TO 11-7

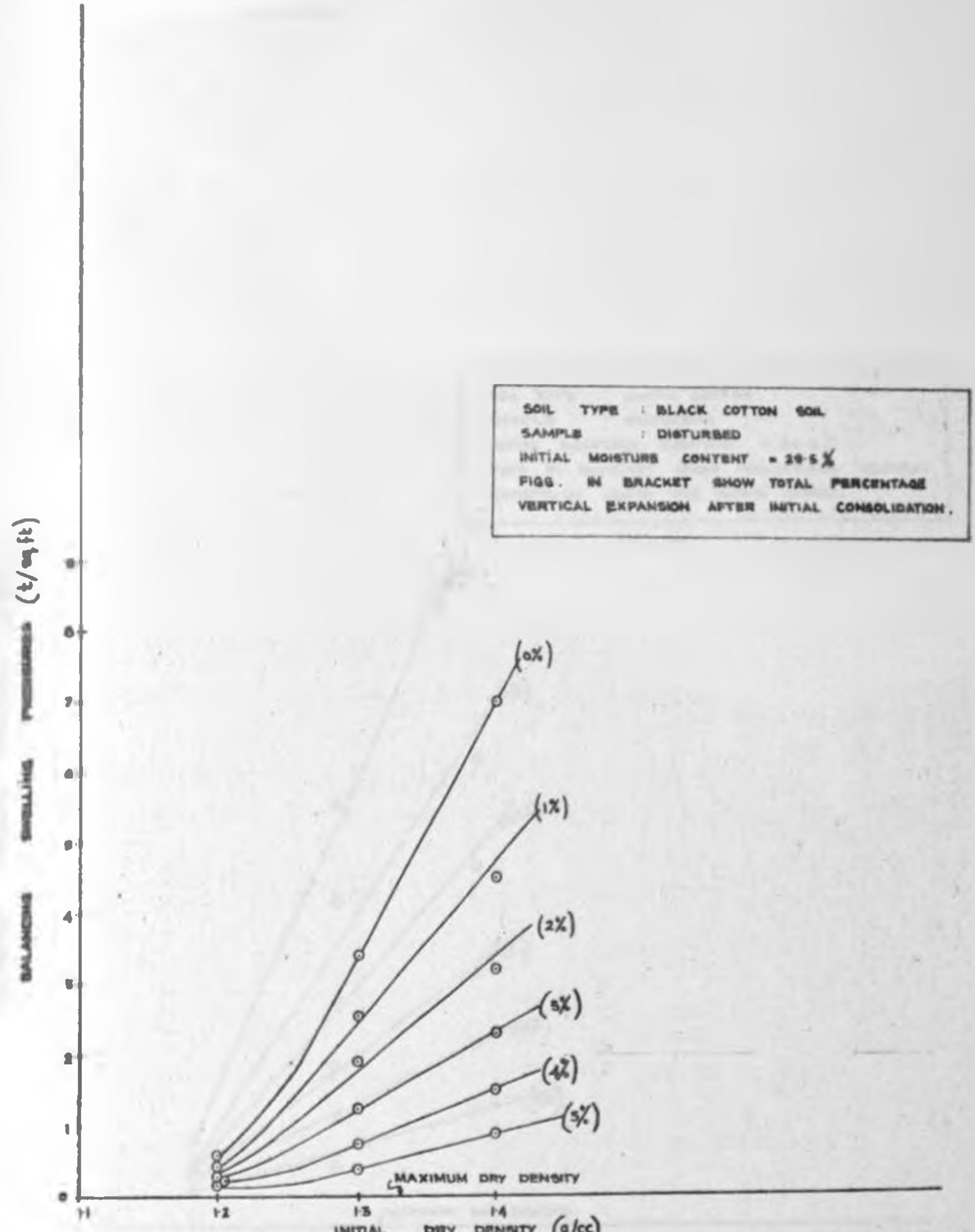

FIG. 5·52. BALANCING SWELLING PRESSURES (t/sq.ft) v. INITIAL DRY DENSITY (g/cc) INDIRECT METHOD. TEST SERIES A-1 TEST No. 9-1 TO 11-7

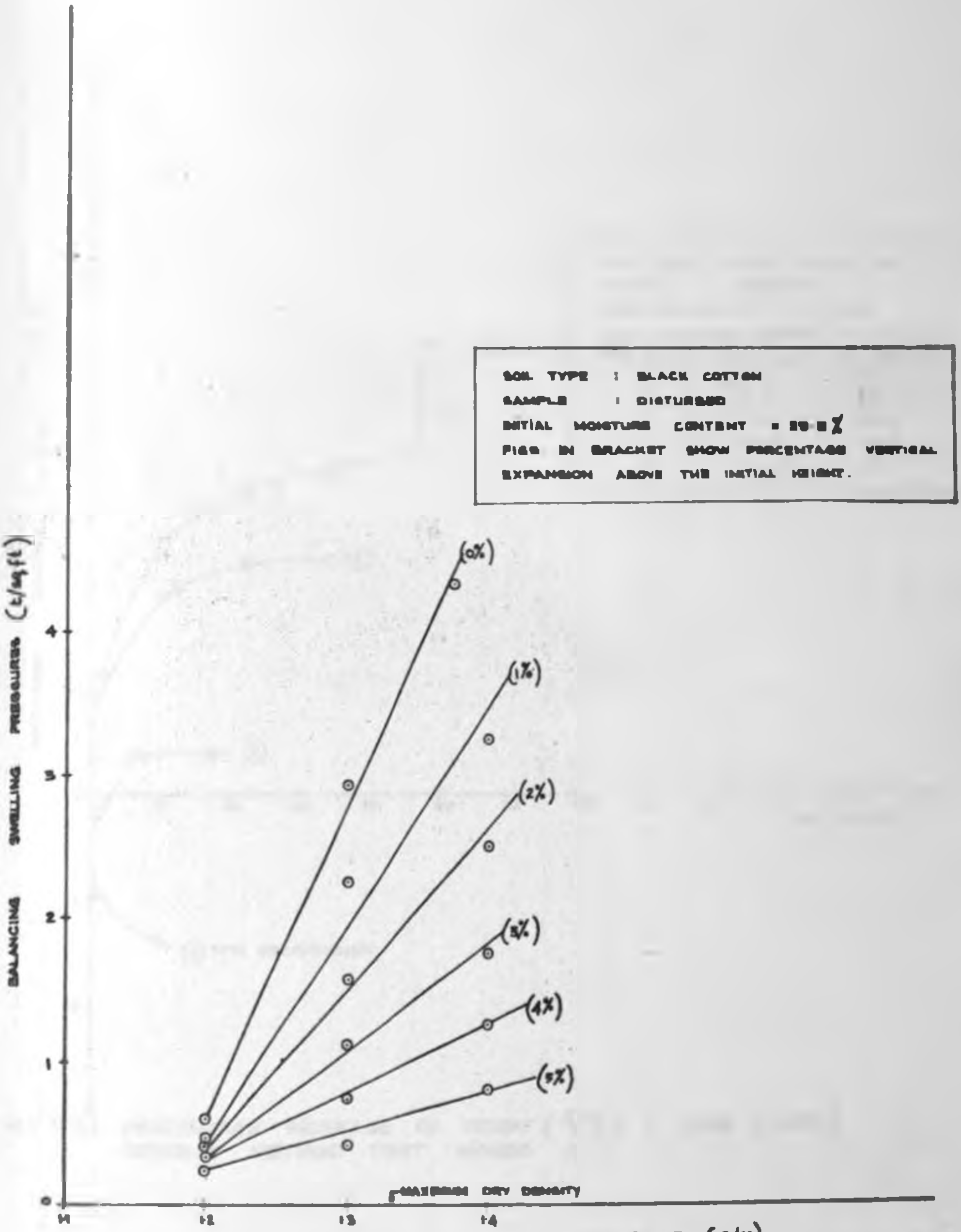

FIG. 5·53. BALANCING SWELLING PRESSURES (t/sq ft) v. INITIAL DRY DENSITY (g/cc) INDIRECT METHOD. TEST SERIES A-1 TEST No. 9-1 TO 11-7

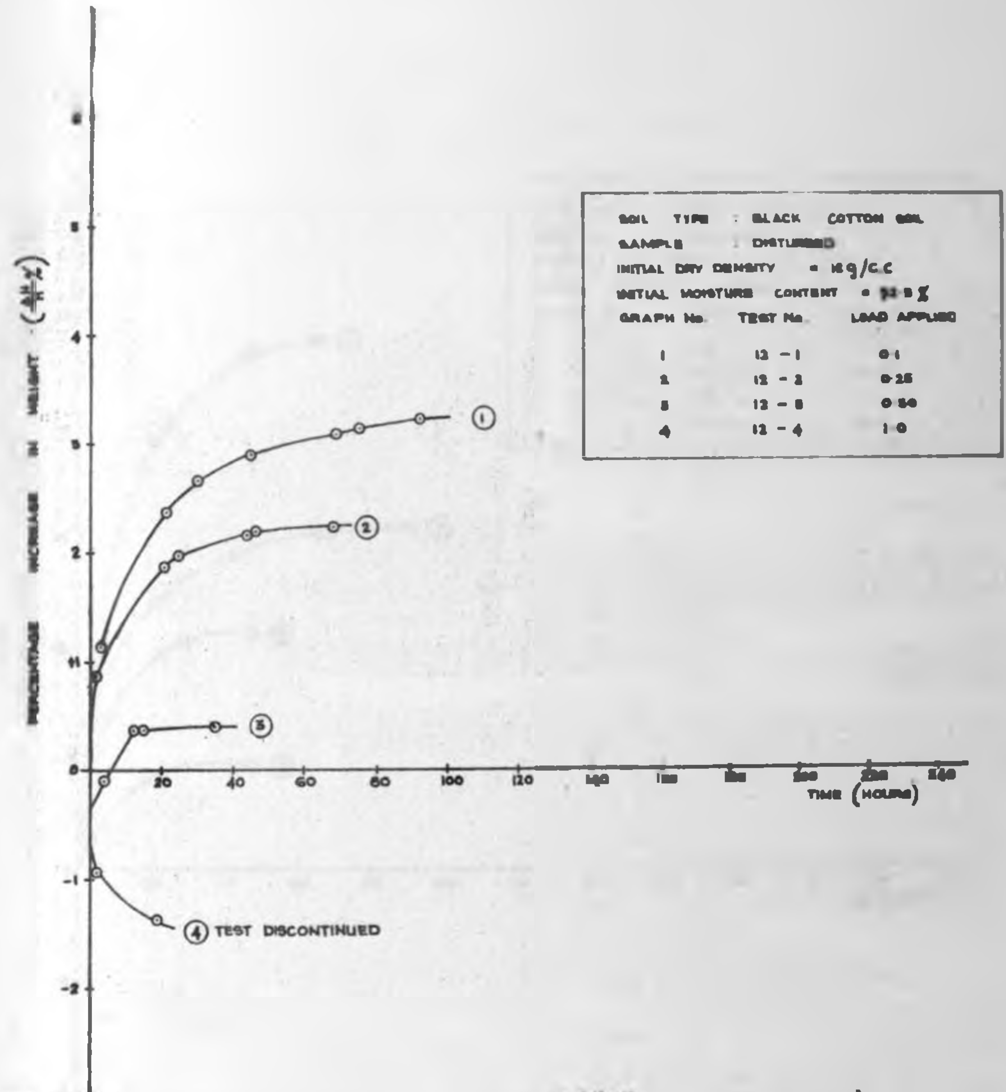

FIG. 5·34. PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}\%$) v. TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

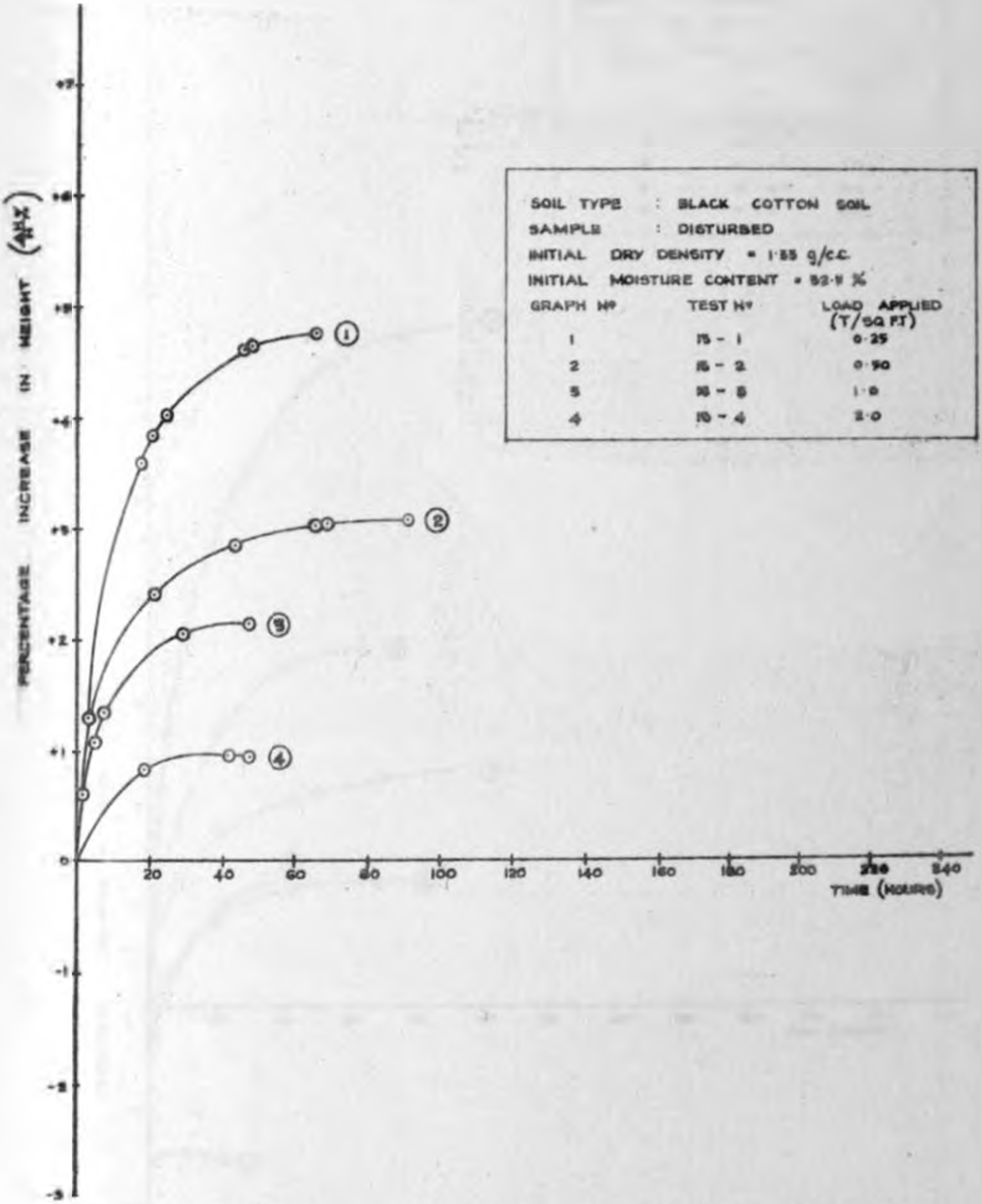

FIG.5·55 PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}$%) v. TIME (HOURS)
INDIRECT METHOD TEST SERIES (A-1)

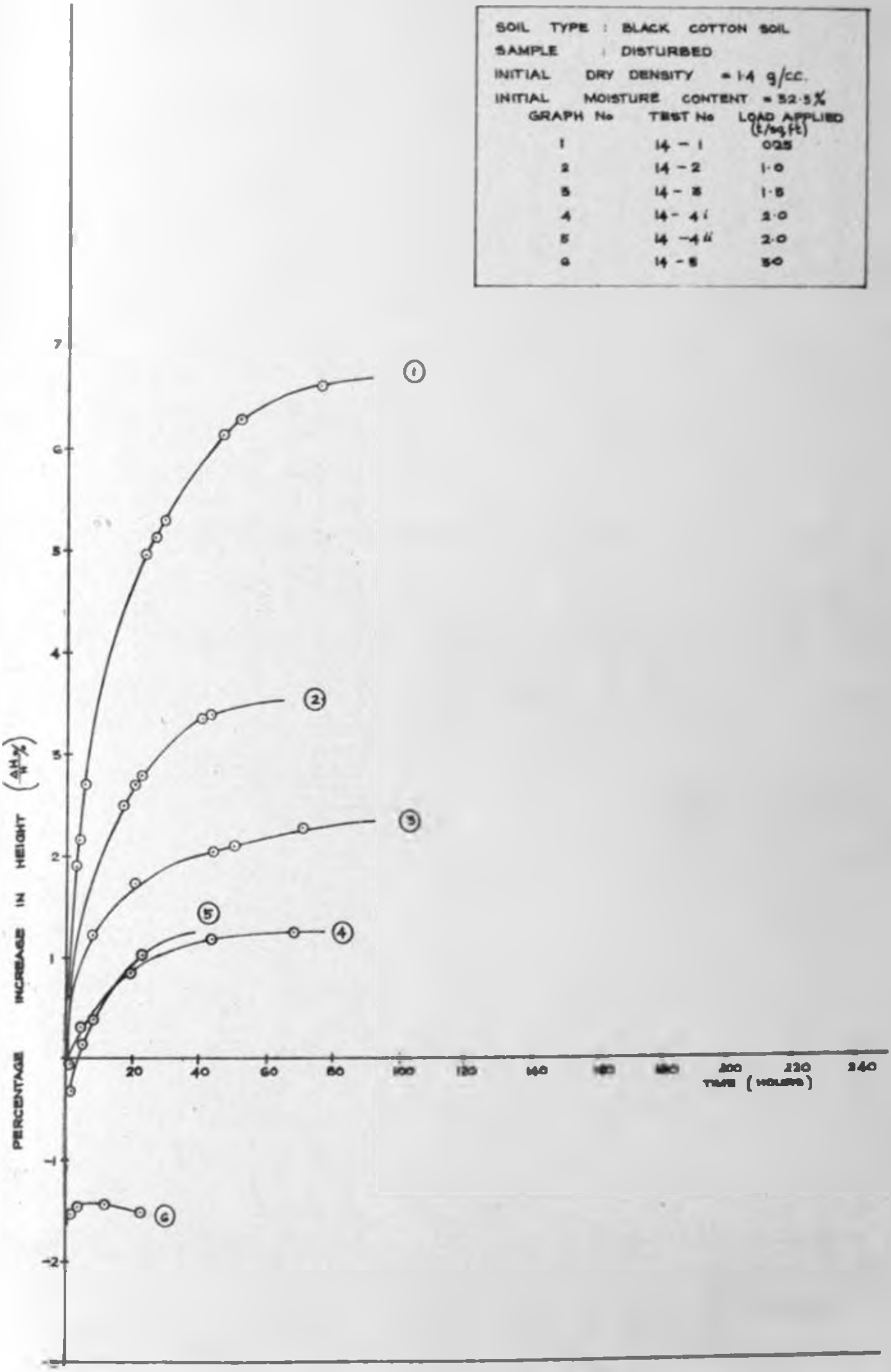

FIG. 3.56. PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}\%$) v TIME (HOURS)
INDIRECT METHOD TEST SERIES A-1

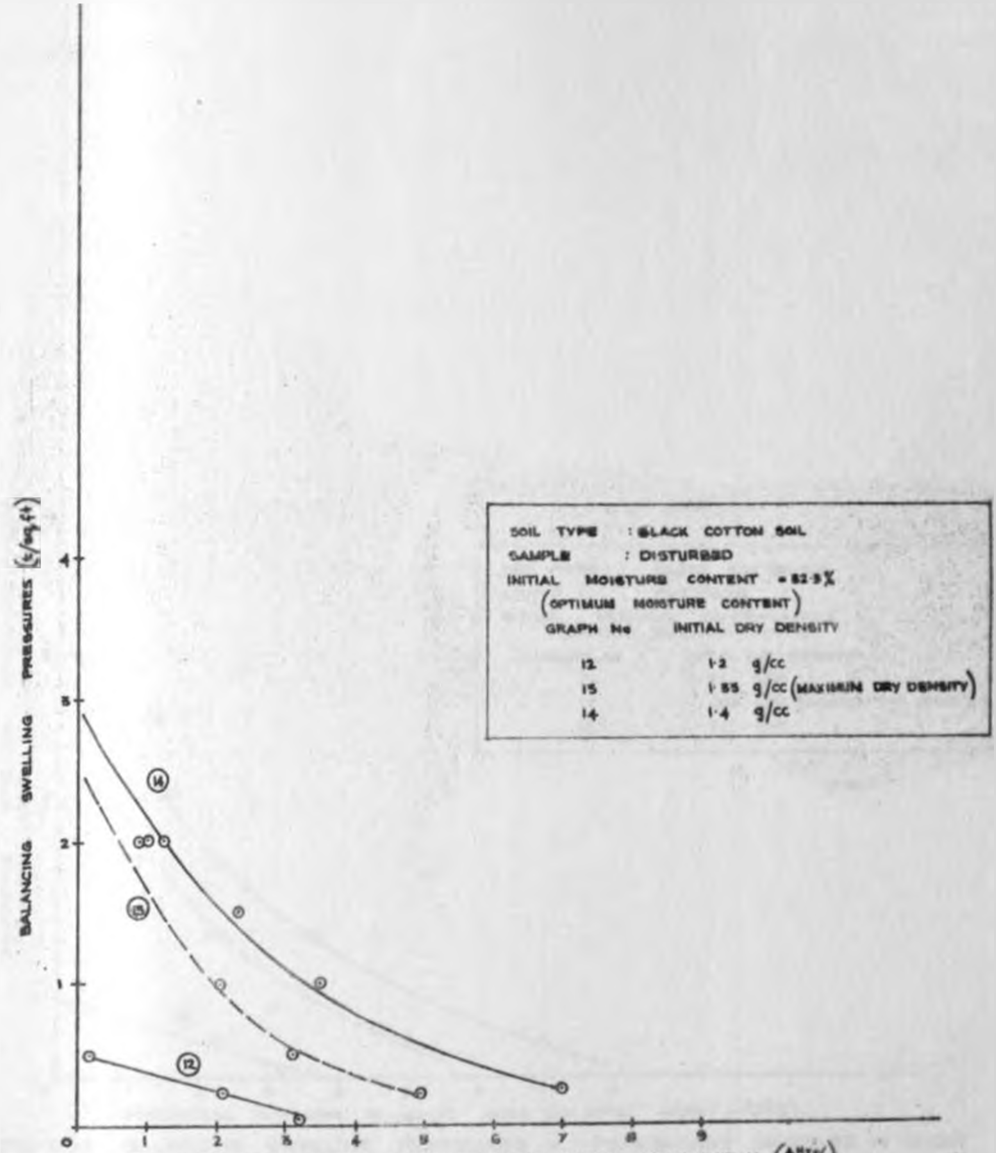

FIG. 5·57 BALANCING SWELLING PRESSURES (t/sq ft) v. TOTAL PERCENTAGE VERTICAL EXPANSION AFTER INITIAL CONSOLIDATION ($\frac{\Delta H_T}{H}$%) INDIRECT METHOD. TEST SERIES A-1 TEST No 12-1 TO 14-5

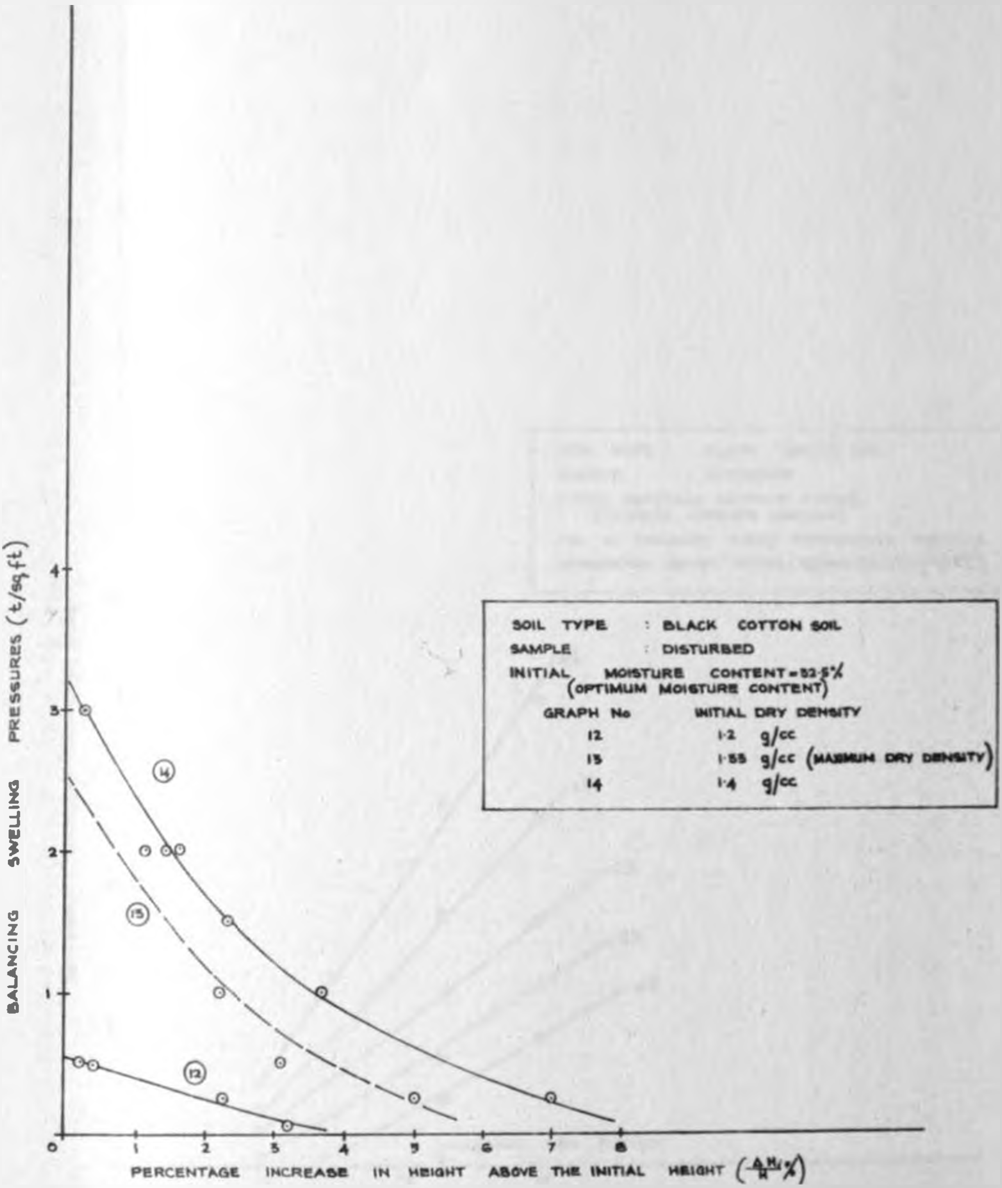

FIG. 5·58. BALANCING SWELLING PRESSURES v. PERCENTAGE INCREASE IN HEIGHT ABOVE THE INITIAL HEIGHT ($\frac{\Delta H}{H}$%)
INDIRECT METHOD. TEST SERIES A-1 TEST No 12-1 TO 14-5

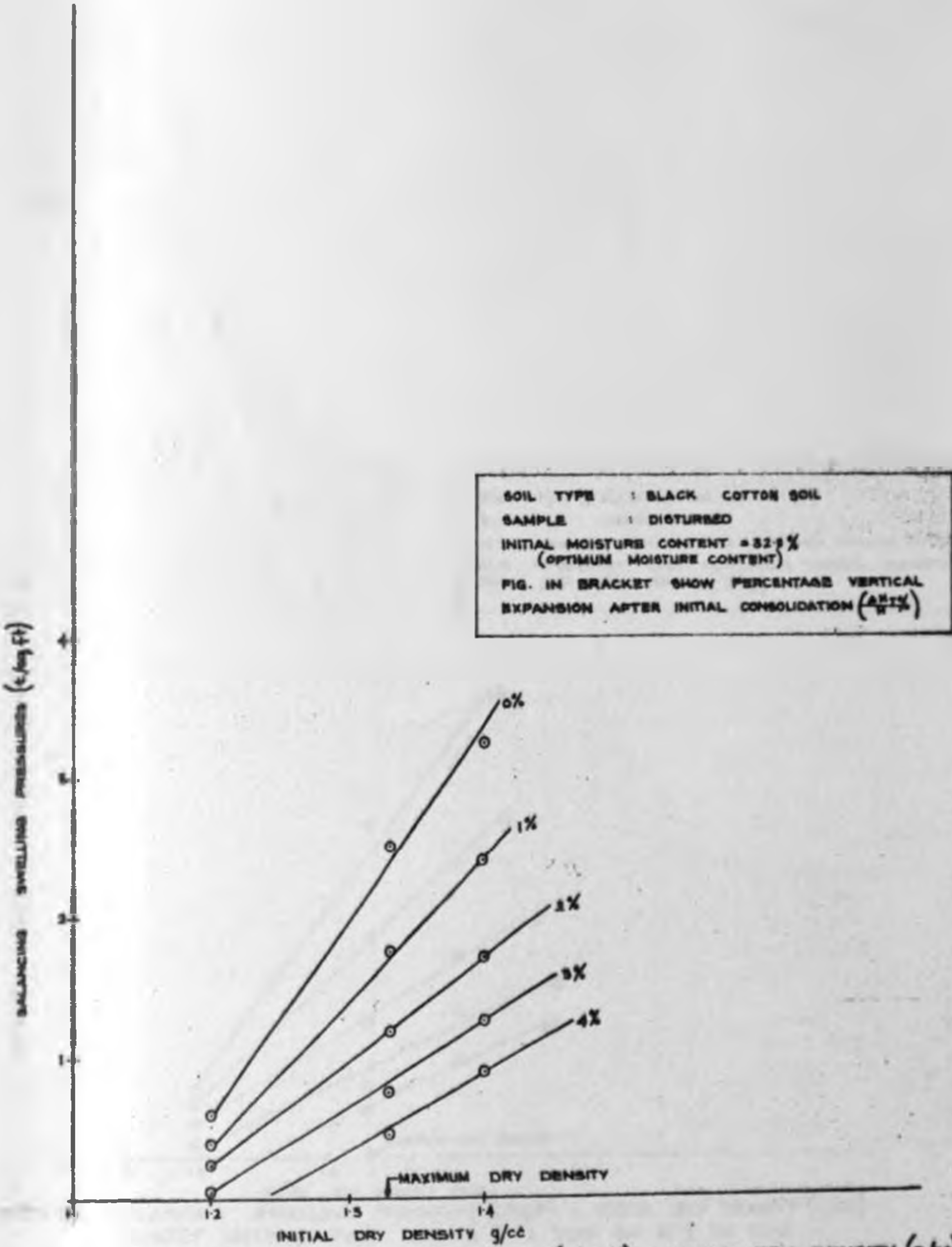

FIG. 5·59 BALANCING SWELLING PRESSURES (t/sq ft) v. INITIAL DRY DENSITY (g/cc) INDIRECT METHOD TEST SERIES A-1 TEST No 12-1 TO 14-5

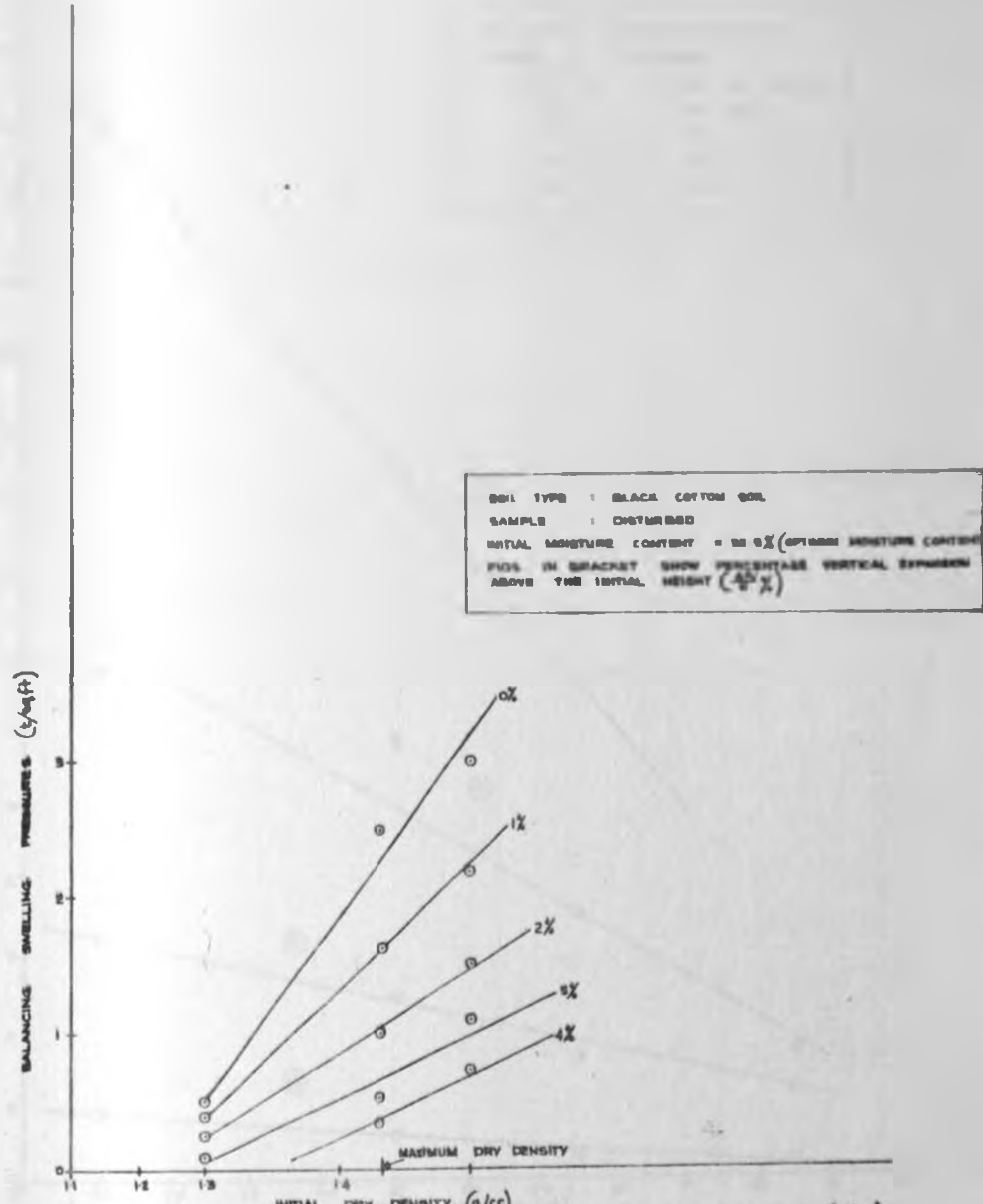

FIG. 5·60. BALANCING SWELLING PRESSURES (t/sqft) v. INITIAL DRY DENSITY (g/cc) INDIRECT METHOD TEST SERIES A-1 TEST No. 12-I TO 14-V

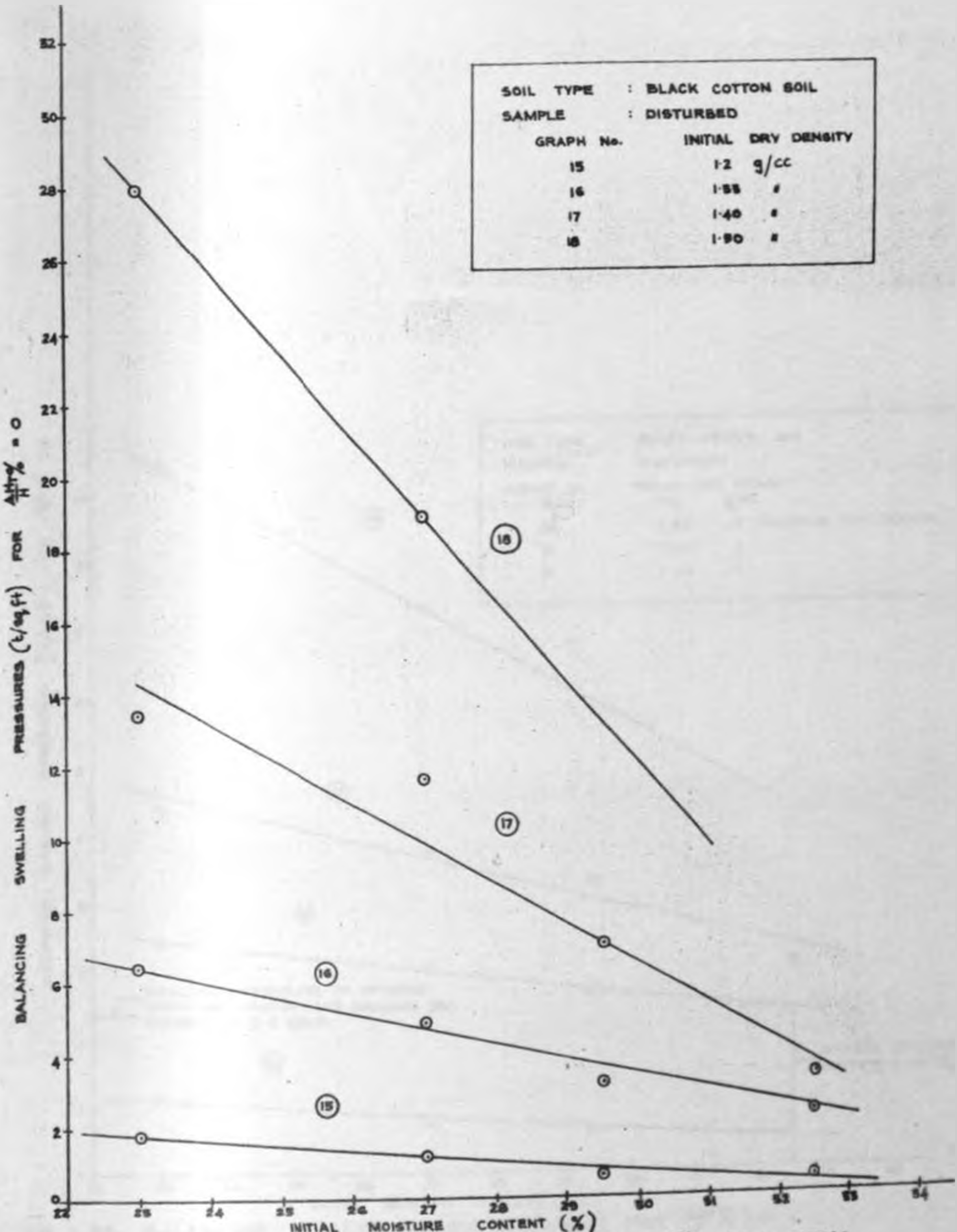

FIG. 5.61 BALANCING SWELLING PRESSURES (t/sq.ft) FOR $\frac{\Delta H}{H} = 0$ v. INITIAL MOISTURE CONTENT (%)

INDIRECT METHOD TEST SERIES A-1 TEST No. 1-1 TO 14-5

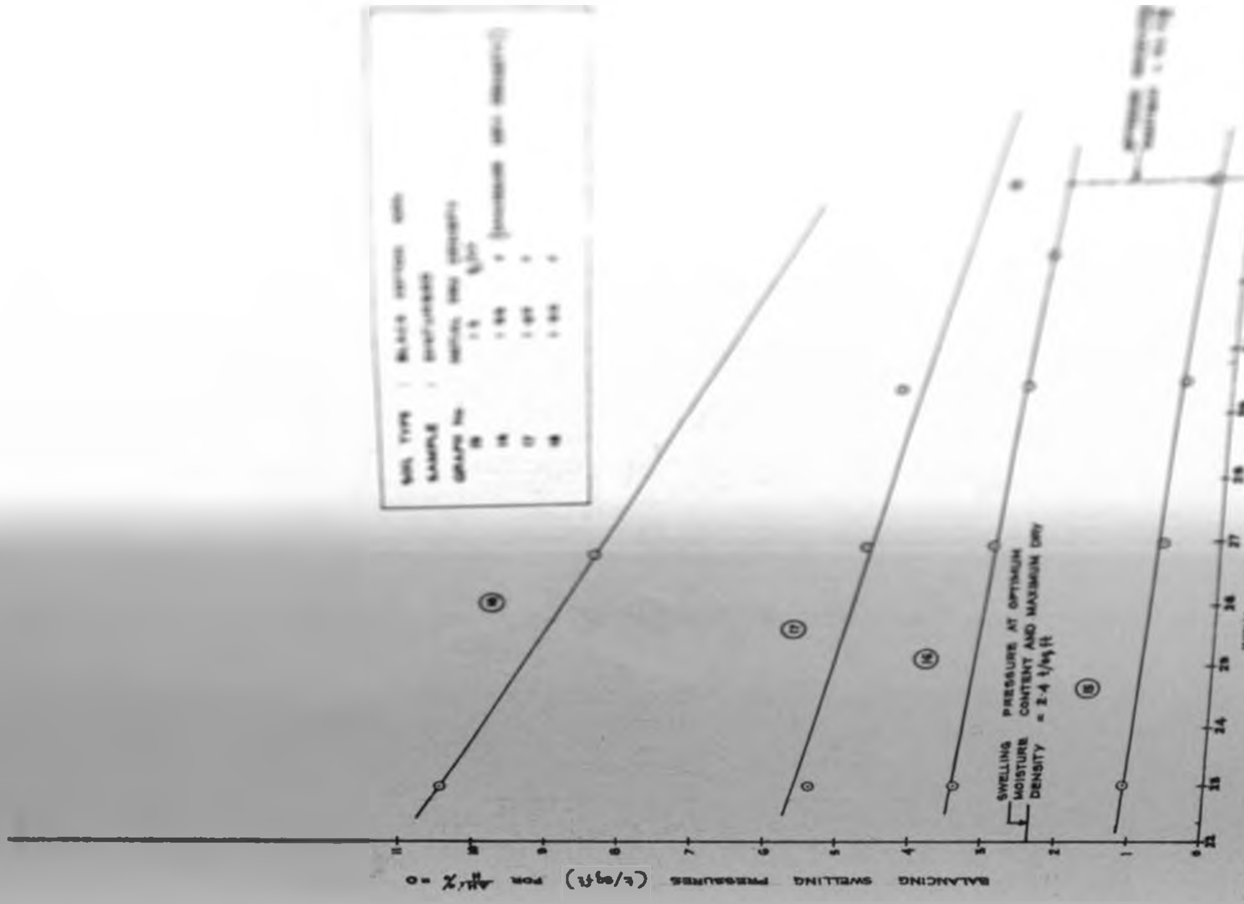

FIG. 5.62. BALANCING SWELLING PRESSURES (t/sq ft) FOR $\frac{\Delta H}{H}$ % = 0 &
INITIAL MOISTURE CONTENT (%)
INDIRECT METHOD. TEST SERIES A-1 TEST No. 1-1 TO 15-4

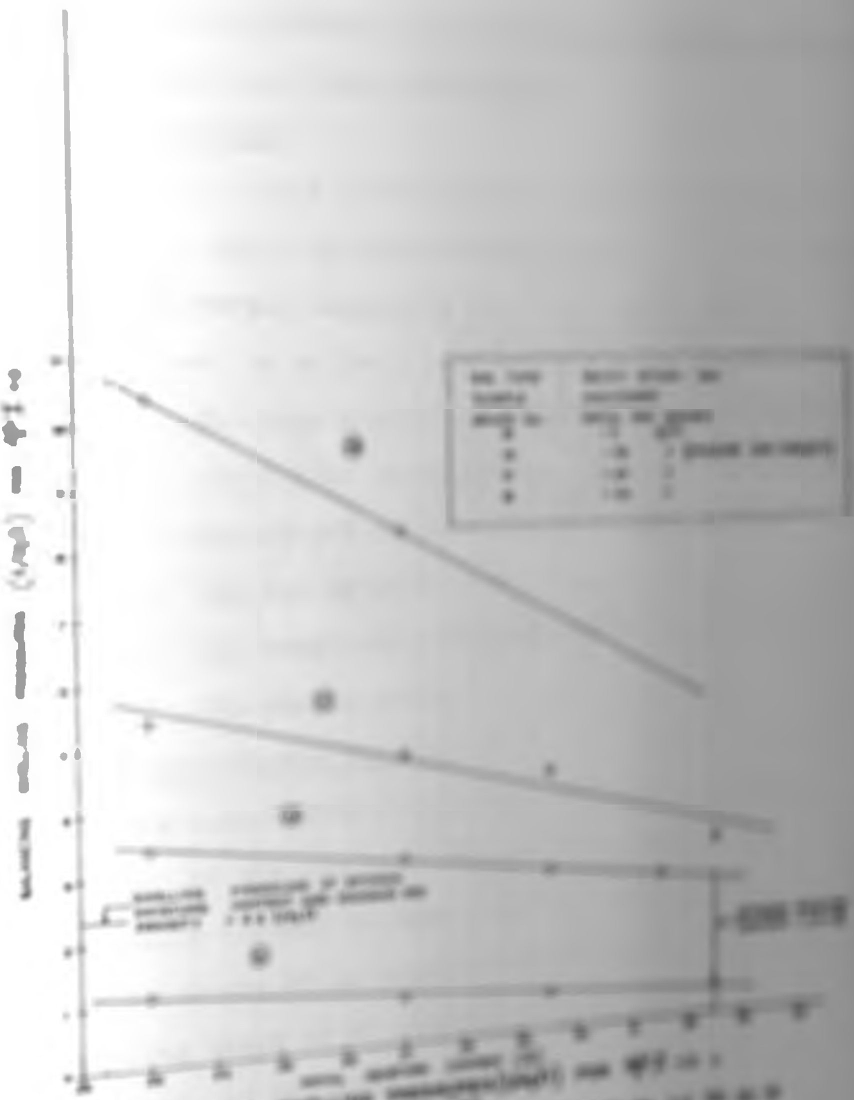

FIG. 5.43 BALANCING SWELLING [illegible]

5.7 Indirect Method. Test Series A-2: Tests on undisturbed Samples of Black Cotton Soil.

5.7a Apparatus:

The apparatus consisted of a standard consolidometer, a burette and a specially designed cell. The cell designed for undisturbed samples is as shown in Fig. 5.64 and 5.65 and is made up of the following parts:

(a) The base A with $\frac{3''}{16}$ diameter hole for water inlet from the burette.

(b) The ring B which contains the specimen.

(c) The top cover C.

(d) The metal plate E fitted with a porous stone.

(e) The porous stone.

5.7b Undisturbed Samples.

The undisturbed samples were obtained by digging a pit. At the bottom of the pit, the soil was very carefully trimmed to form a cylinder of undisturbed soil approximately 8" in diameter and 9" high. A cylinderical sampling mould of 4" diameter was then pushed gently into the soil, at the same time trimming the sides to the size of the cylinder. Care was taken to see that there was minimum disturbance and that the cylinder was pushed vertically. After taking the sample in the cylinder the top and bottom were trimmed flush and the sides coated.

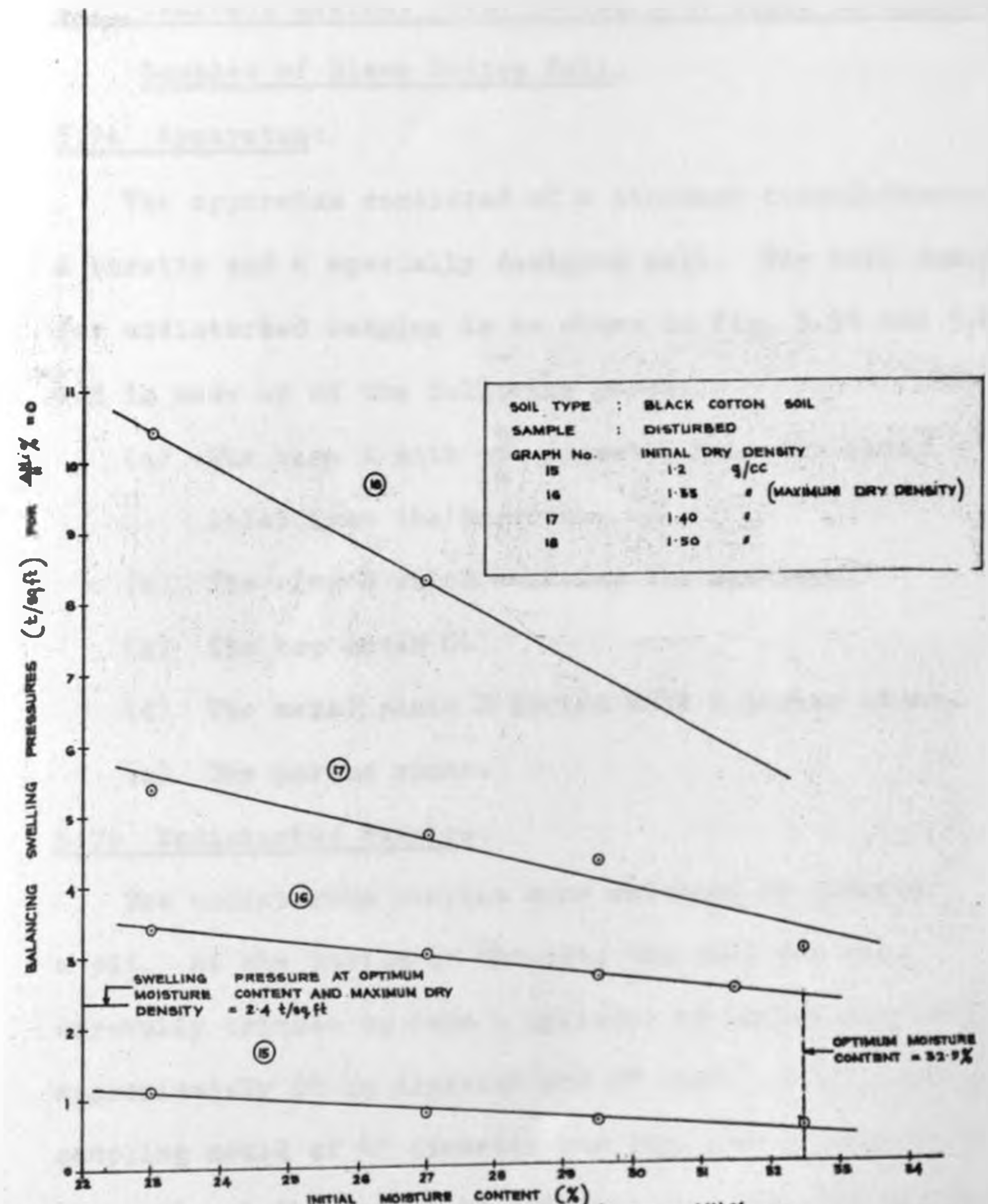

FIG. 5.62. BALANCING SWELLING PRESSURES (t/sq.ft) FOR $\frac{\Delta H_i}{H}\% = 0$ v. INITIAL MOISTURE CONTENT (%)
INDIRECT METHOD. TEST SERIES A-1 TEST No. 1-1 TO 14-5

with wax, so as to retain the original moisture content. At the same time disturbed soil samples were taken from the same pit to carry out some of the other tests.

5.7c Procedure.

The consolidometer ring was gently pushed into the undisturbed sample by carefully trimming the sides. The top and bottom of the sample in the consolidometer ring were then trimmed flush with the ring. The initial moisture content of the soil and the weight of the ring and the sample were taken.

The ring containing the sample and fitted with a rubber O-ring was placed into the base A containing the saturated porous stone. A loading plate fitted with a porous stone was then placed on top. The top cover C was then bolted to the base A. (Figs. 5.63, 5.64 and 5.65).

The cell was then placed in the consolidometer. After this the procedure similar to that outlined in the Indirect Method, Test Series A-1, was followed.

These tests were carried out on the undisturbed samples at the initial moisture content and initial dry density conditions listed below. These conditions were attained by air drying the prepared undistrubed samples from the natural moisture content.

with wax, so as to retain the original moisture content. At the same time disturbed soil samples were taken from the same pit to carry out some of the other tests.

5.7c Procedure.

The consolidometer ring was gently pushed into the undisturbed sample by carefully trimming the sides. The top and bottom of the sample in the consolidometer ring were then trimmed flush with the ring. The initial moisture content of the soil and the weight of the ring and the sample were taken.

The ring containing the sample and fitted with a rubber O-ring was placed into the base A containing the saturated porous stone. A loading plate fitted with a porous stone was then placed on top. The top cover C was then bolted to the base A. (Figs. 5.63, 5.64 and 5.65).

The cell was then placed in the consolidometer. After this the procedure similar to that outlined in the Indirect Method, Test Series A-1, was followed.

These tests were carried out on the undisturbed samples at the initial moisture content and initial dry density conditions listed below. These conditions were attained by air drying the prepared undistrubed samples from the natural moisture content.

FIG.5.63. MODIFIED CELL FOR UNDISTURBED SOIL SAMPLE

INDIRECT METHOD TEST SERIES A-2

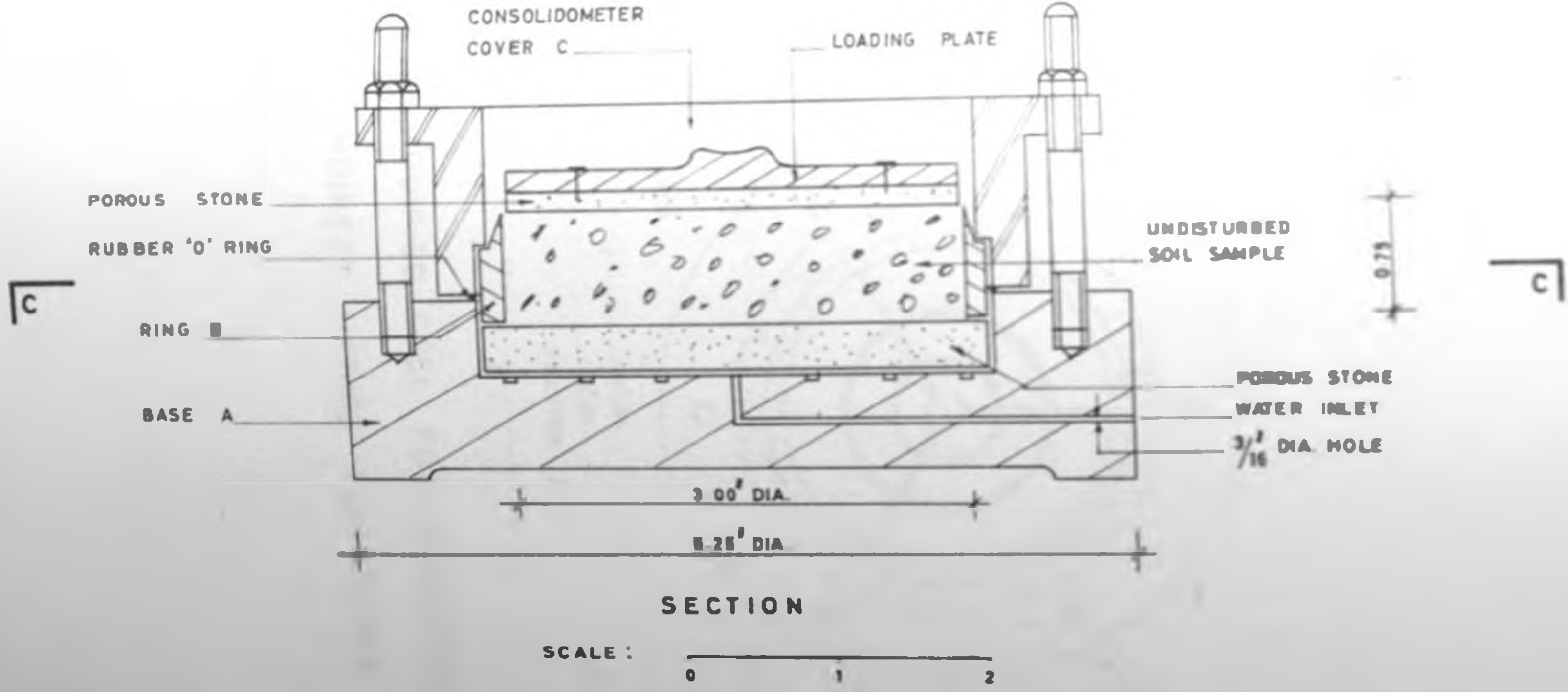

FIG. 5.64. MODIFIED CELL FOR UNDISTURBED SOIL SAMPLE INDIRECT METHOD - TEST SERIES A-2

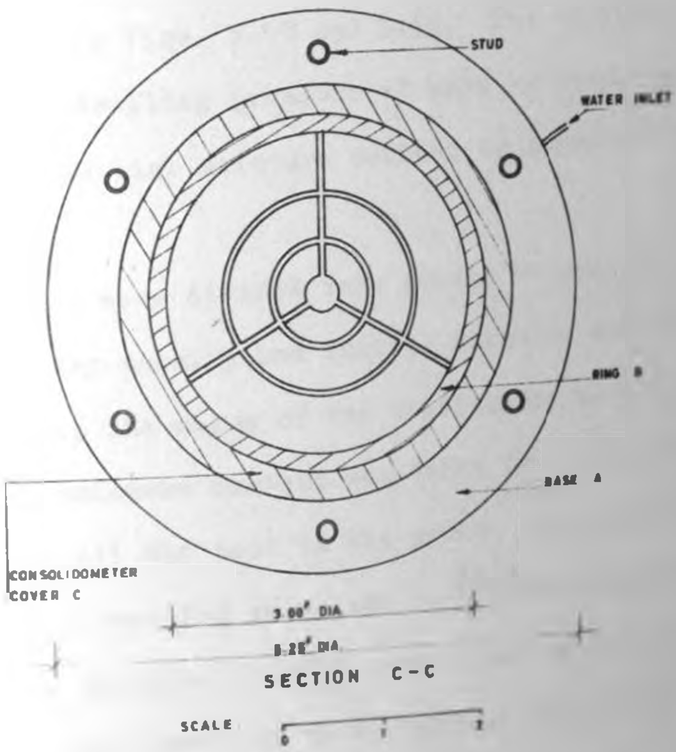

FIG.5.65. MODIFIED CELL FOR UNDISTURBED SOIL SAMPLE

INDIRECT METHOD TEST SERIES A-2

5.7d Results:

For each of the tests, graphs of percentage increase in height $\frac{(\Delta H\%)}{H}$ against time were drawn. These graphs are shown in Figs. 5.65 and 5.66. The variation of the Balancing Swelling pressure at zero vertical expansion, with the initial moisture content is shown in Figs. 5.67 and 5.68.

Tests were divided into groups depending on their initial dry density and initial moisture content. In each group the value of the initial dry density and initial moisture content was taken to be the average value of all the test in the group. The graphs of balancing swelling pressures v. $\frac{\Delta Hi\%}{H}$ and balancing swelling pressures v. $\frac{\Delta H_T\%}{H}$ were drawn as before.

As the number of tests carried out in this series were few due to the difficulty in getting the undisturbed samples, no definite values of the balancing swelling pressures when $\frac{\Delta H_T\%}{H}$ is zero and when $\frac{\Delta Hi\%}{H}$ is zero could be obtained. The probable values are shown in Table 5.7.

Table 5.6

Results:

Indirect Method Test Series A-2

	Test No	Load t/sq.ft.	Wi	γ_{di}	Wf %	Si %	Sf %	ΔHT/H %	ΔHi/H %
Group 1	1	2	22.5	1.46	31.5	73	91	6.0	5.2
	9	5	22.0	1.47		74.3		1.95	0.71
Group 2	6	2	26.2	1.30	35.7	67.5	92.5	0.80	0.05
	7	2	24.6	1.36	33.0	69	93	1.5	00
	13	2	26.0	1.31	35.8	68.5	94	1.2	0.01
Group 3	2	1	31.0	1.34	35.0	85	90	3.2	2.9
	8	2	31.0	1.29	34.5	79	89	0.6	-0.67
Group 4	3	¼	37.5	1.22	40.4	85	90	0.8	0.8
	4	½	35.4	1.27	38.8	86.2	95	0.9	0.65
	5	½	35.0	1.22	41.4	78	91.5	0.62	0.35
	10	½	36.0	1.21	40.5	80	90	0.70	0.50
	11	1	36.4	1.23	40.8	85	95	0.75	0.35
	12	1	39.5	1.2	47.1	84	100	0.35	0.15

Table 5.7

Indirect Method Test Series A-2

Summary of Results.

Undisturbed Sample.

Average Initial Moisture Content %	Average Initial Dry Density (g/cc)	Balancing Swelling Pressures When $\frac{\Delta HT\%}{H} = 0$ (t/sq.ft.)	Balancing Swelling Pressures When $\frac{\Delta Hi\%}{H} = 0$ (t/sq.ft)
25.6	1.32	3.8	2.0
35.6	1.22	2.0	1.3

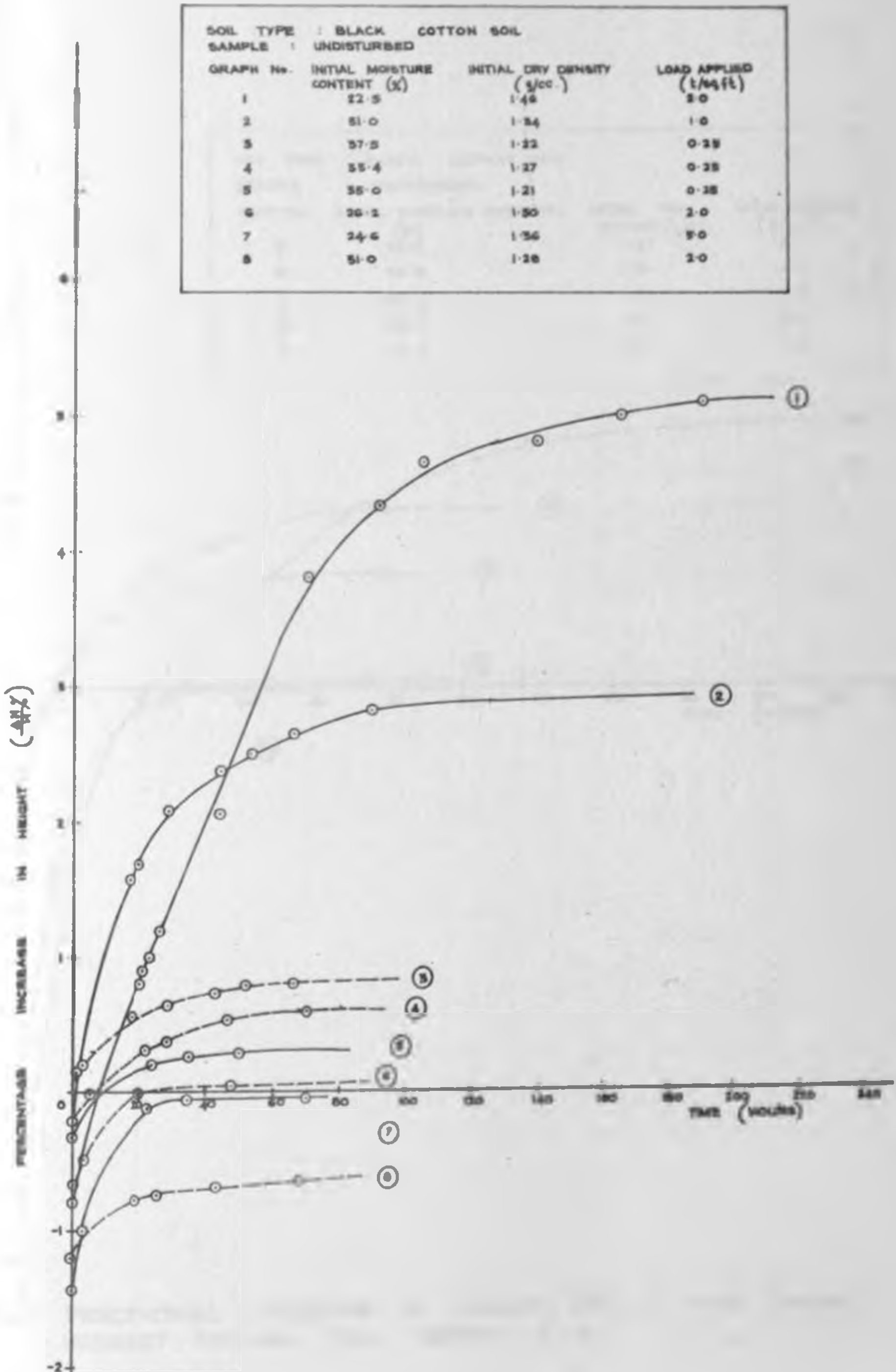

FIG. 5.66 PERCENTAGE INCREASE IN HEIGHT $\left(\frac{\Delta H}{H}\%\right)$ v. TIME (HOURS)
INDIRECT METHOD TEST SERIES A-2

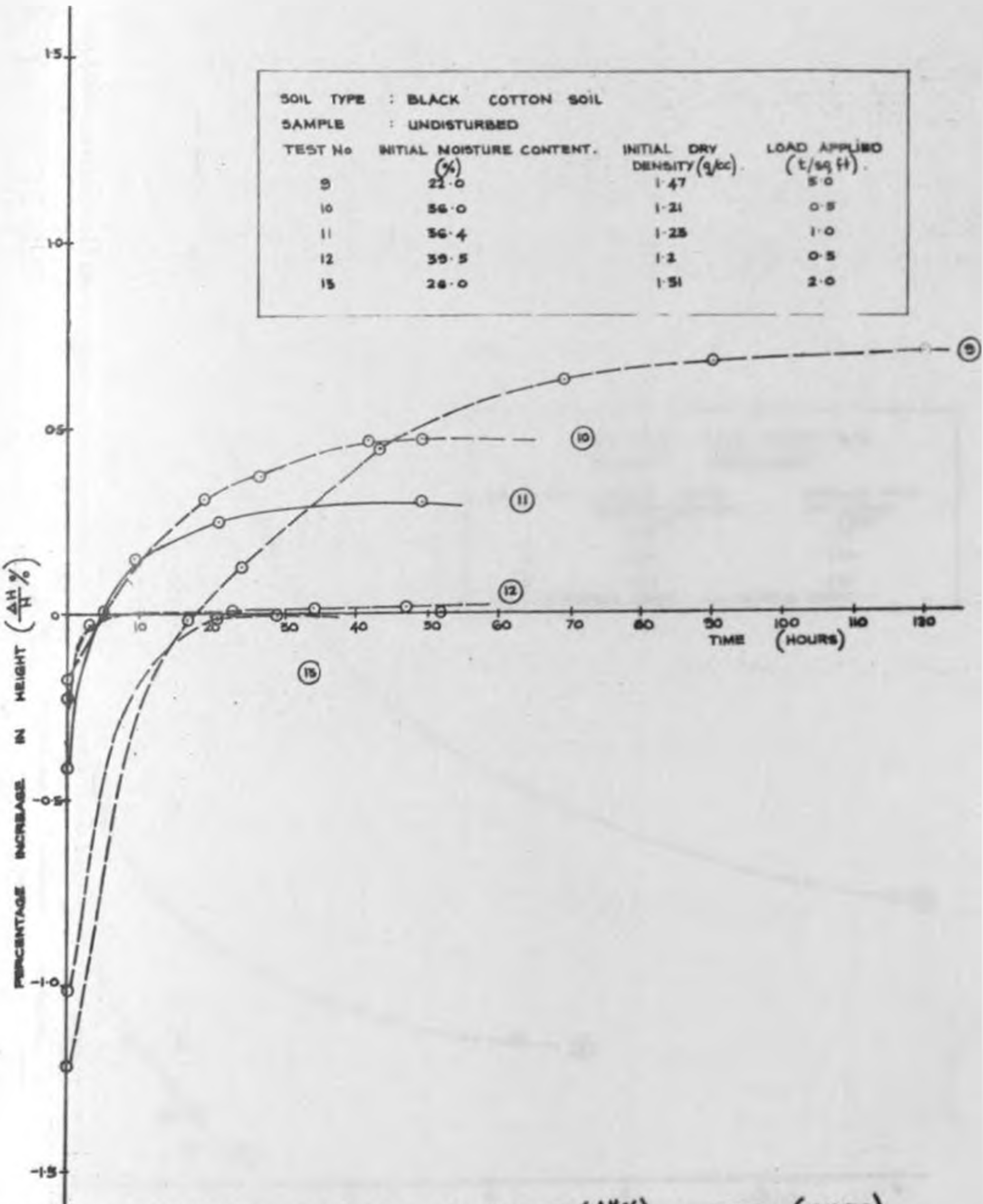

FIG. 5·67 PERCENTAGE INCREASE IN HEIGHT ($\frac{\Delta H}{H}$%) v. TIME (HOURS)
INDIRECT METHOD. TEST SERIES A - 2

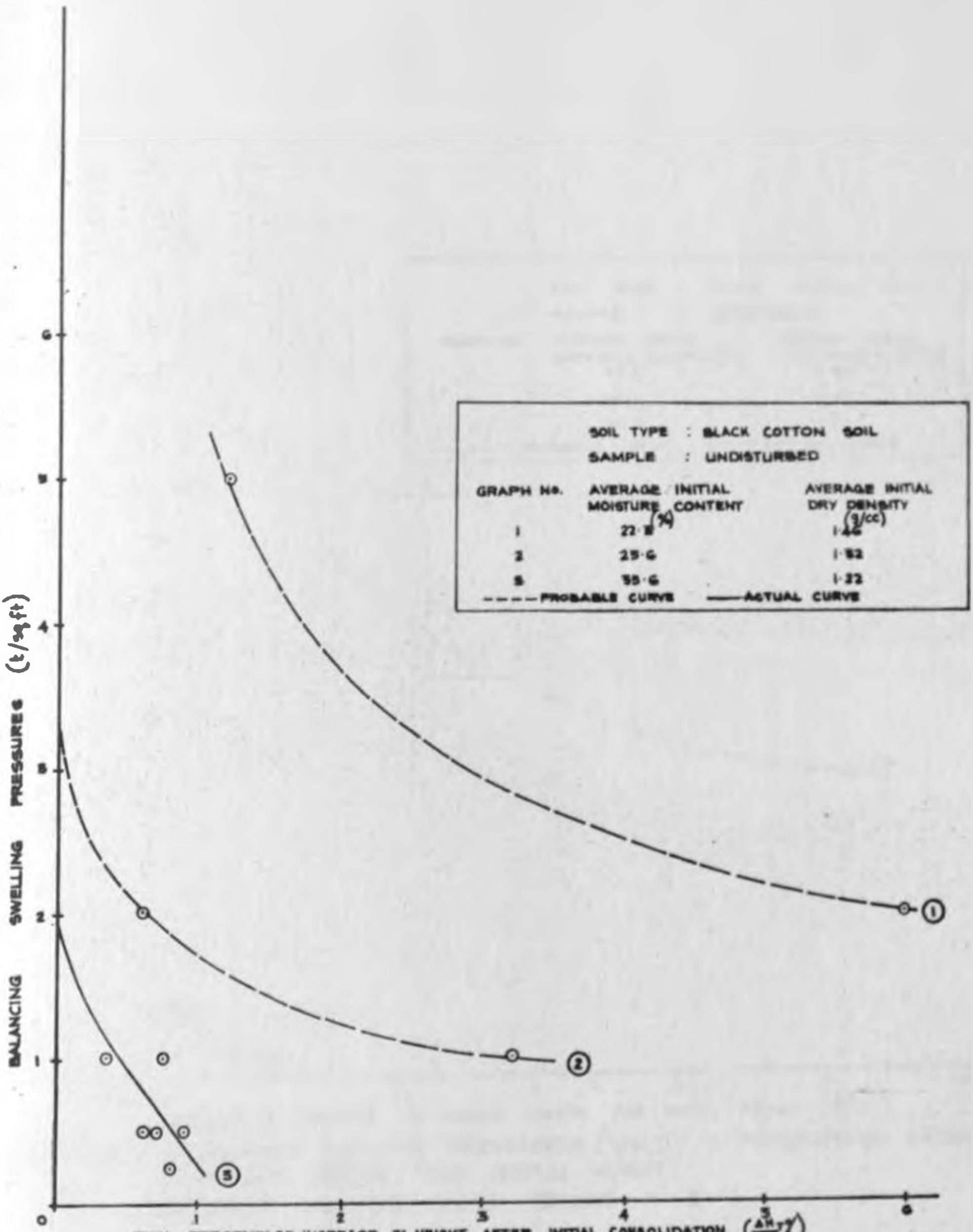

FIG. 5. 68. BALANCING SWELLING PRESSURES (t/sq ft) v. TOTAL PERCENTAGE INCREASE IN HEIGHT, AFTER INITIAL CONSOLIDATION $\left(\frac{\Delta H_T}{H}\%\right)$ INDIRECT METHOD TEST - SERIES A-2

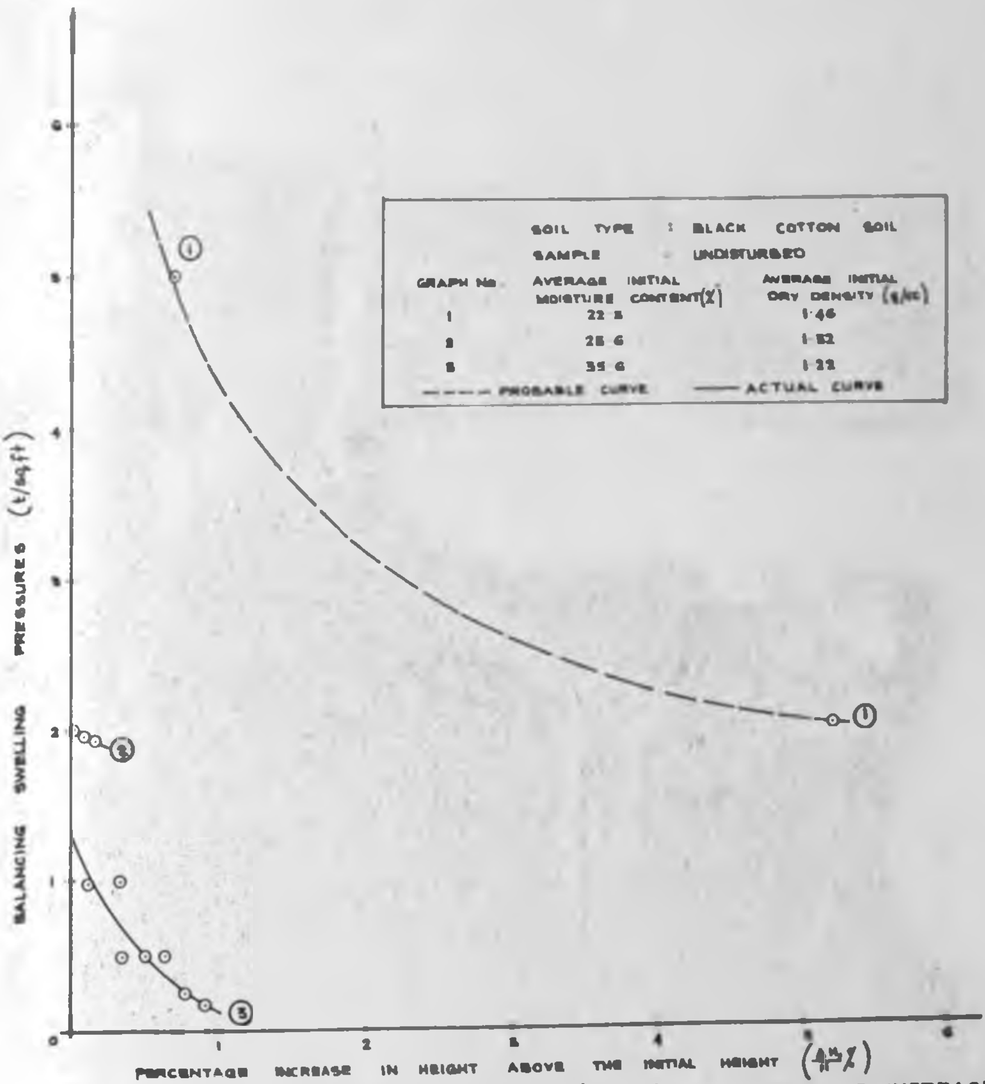

FIG. 5.69. BALANCING SWELLING PRESSURES (t/sq ft) v. PERCENTAGE INCREASE IN HEIGHT ABOVE THE INITIAL HEIGHT. INDIRECT METHOD. TEST SERIES A - 2

5.8 Direct Method. Test Series B1: Tests on disturbed samples of Black Cotton Soil.

Tests were carried out at the following initial dry density conditions-1.2g/cc, 1.33 g/cc, 1.4 g/cc and 1.5 g/cc. At each initial dry density, a number of tests were carried out at various initial moisture contents.

5.8a Apparatus for the measurement of Swelling Pressure.

The specially designed apparatus to study the swelling pressure of a soil under controlled horizontal and vertical confinement is shown in the Figs. 5.73 and 5.74. It is made up of the following parts:

(1) Top steel plate A.

(2) Bottom Steel plate B fitted with studs.

(3) Top Cover C.

(4) The ring E in which the specimen is compacted.

(5) The porous stone.

(6) Load Cell or High Strength Aluminimum Bar fitted with at least 3 strain gages at 120°.

(7) Strain measuring equipment.

(8) 2 No. Cover G. used for statically compacting the specimen.

(9) Rod K used for Calibrating the instrument.

(10) Load applying device.

5.8b Calibration of the Apparatus.

The apparatus was set up as shown in the Fig. 5.71. It was placed in a loading device (Fig. 5.76). The load was applied in desired increments and the corresponding strain readings indicated by the strain gage apparatus were noted. A calibration curve, applied pressure v. strain, was plotted (Fig. 5.77).

5.8 c Preparation of Sample.

The method adopted for the preparation of the sample was similar to that outlined for the statically compacted samples in the Indirect Method, Test Series A-1.

5.8d Procedure.

The calculated amount of soil at desired moisture content to achieve the desired dry density was statically compacted in the ring E by using covers G (Fig. 5.72). After the removal of covers G, a dry porous stone was placed on top and a saturated porous stone was placed at the bottom of the ring. This assembly was then placed on top of bottom plate B. The top cover C was placed on the ring, followed by the load cell. Top plate A was then placed on the load cell and the nuts were tightened so as to apply some initial load to ensure that the cover C was resting on top of ring E (Figs. 5.73 and 5.74).

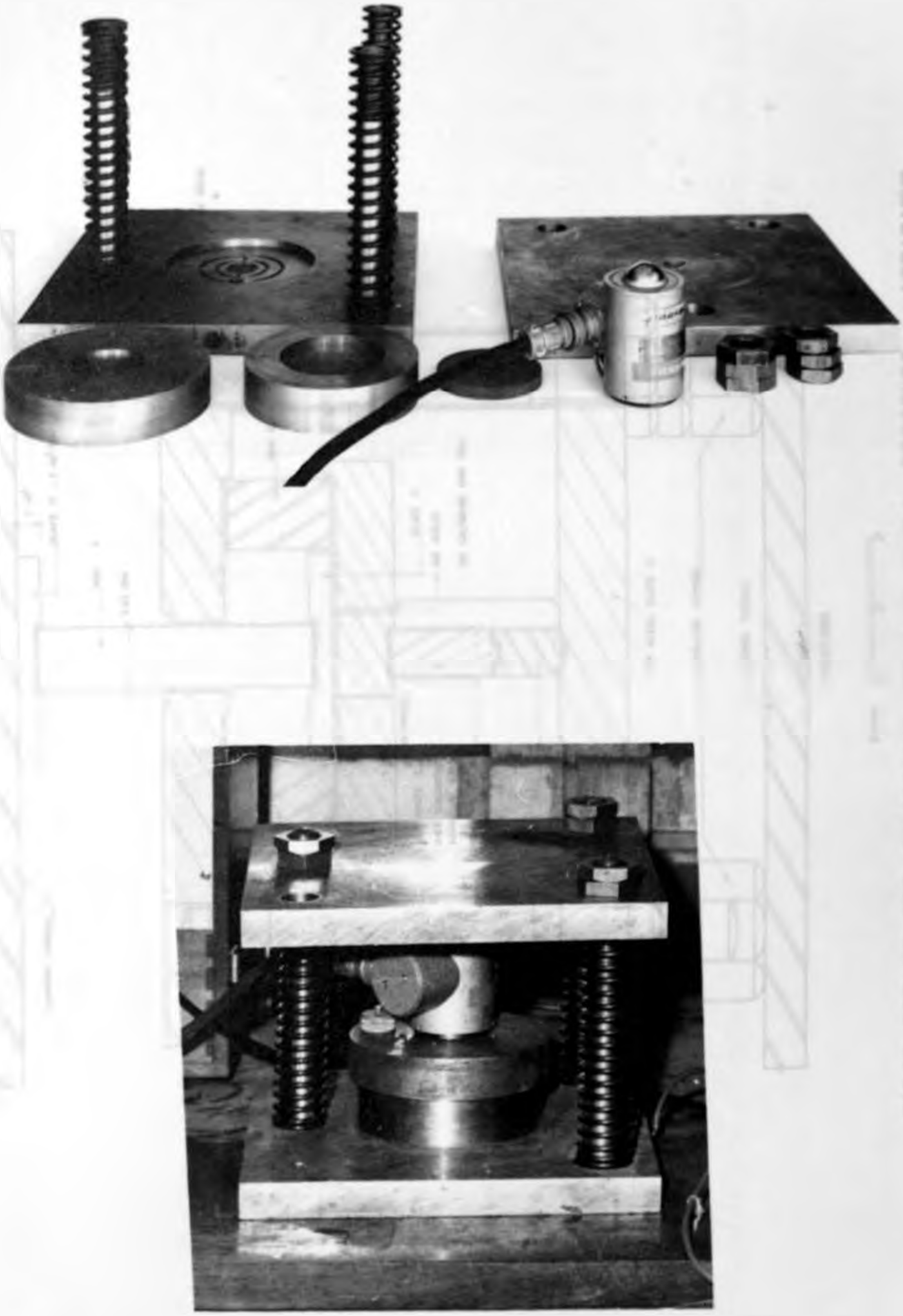

FIG.5.70. APPARATUS FOR THE MEASUREMENT OF THE SWELLING PRESSURE DEVELOPED IN A SOIL UNDER CONTROLLED CONDITIONS OF LATERAL AND VERTICAL CONFINEMENT.

DIRECT METHOD TEST SERIES B.

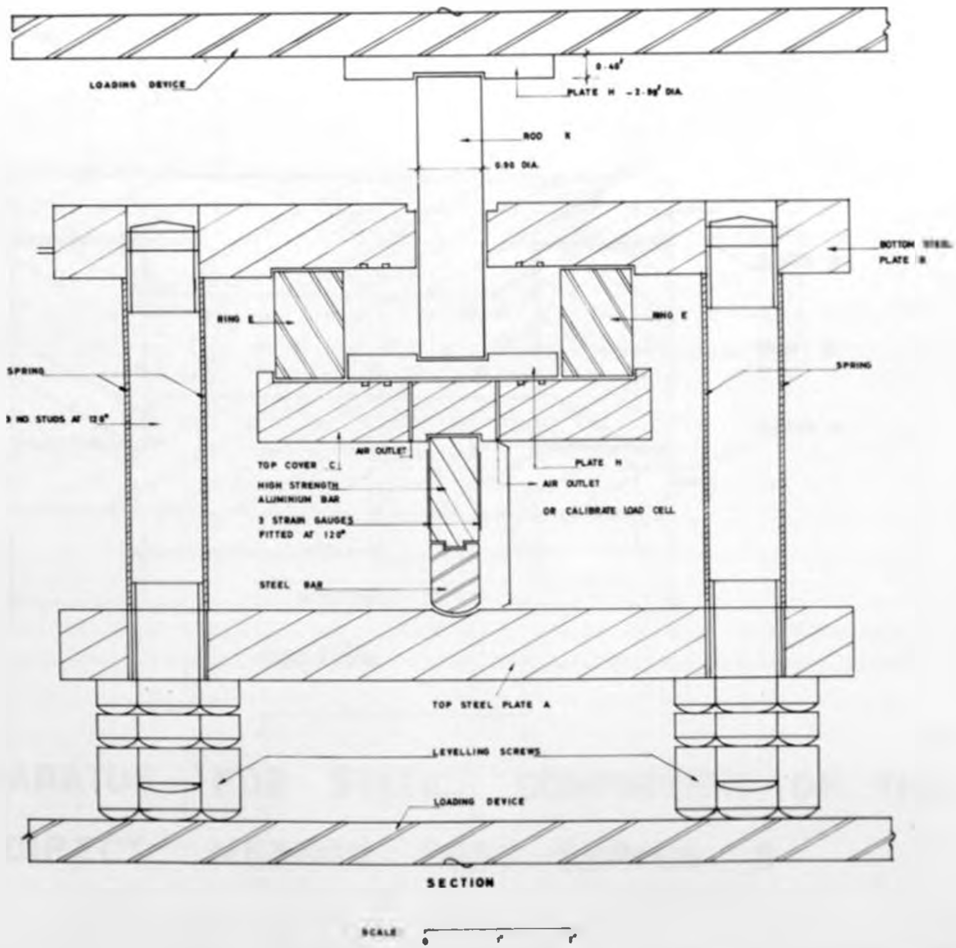

FIG.5.11. SET UP FOR CALIBRATION OF THE SWELL PRESSURE APPARATUS
DIRECT METHOD TEST SERIES B

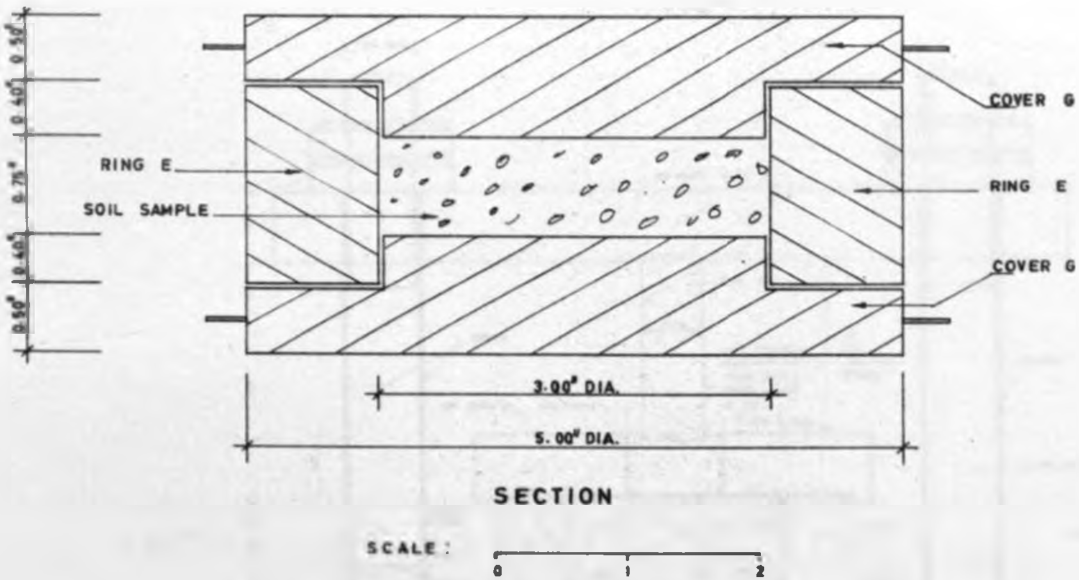

FIG. 5.72. APPARATUS FOR STATIC COMPACTION OF THE SPECIMEN
DIRECT METHOD TEST SERIES B

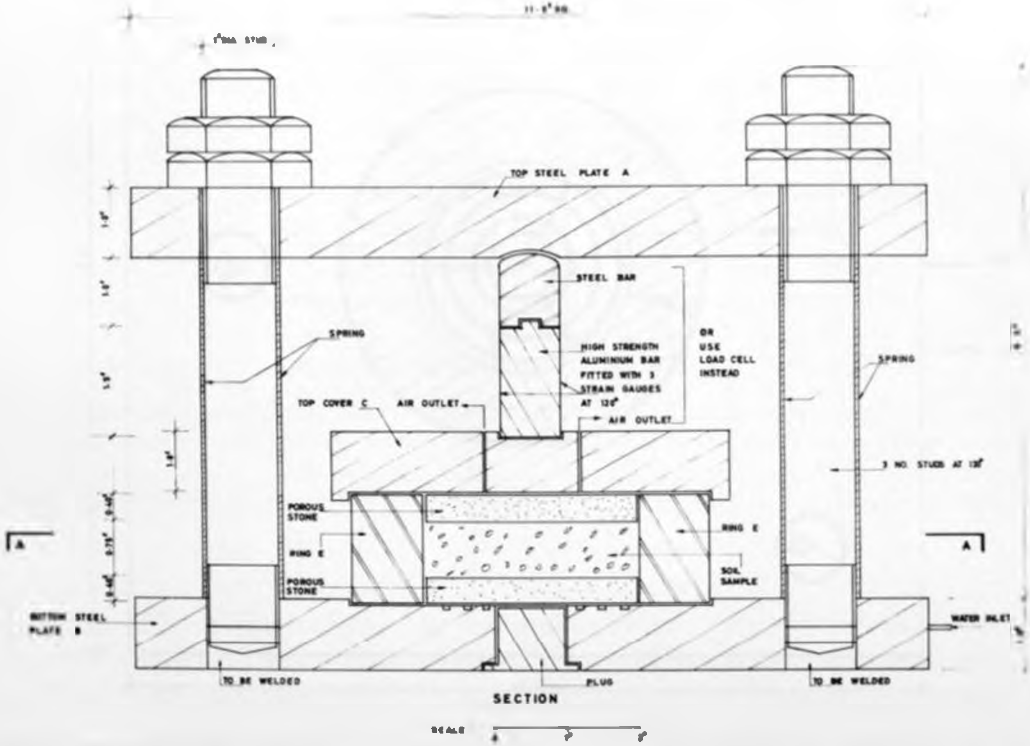

FIG. 5.73. APPARATUS FOR THE MEASUREMENT OF SWELLING PRESSURE DEVELOPED IN A SOIL UNDER CONTROLLED CONDITIONS OF LATERAL AND VERTICAL CONFINEMENT

DIRECT METHOD TEST SERIES B

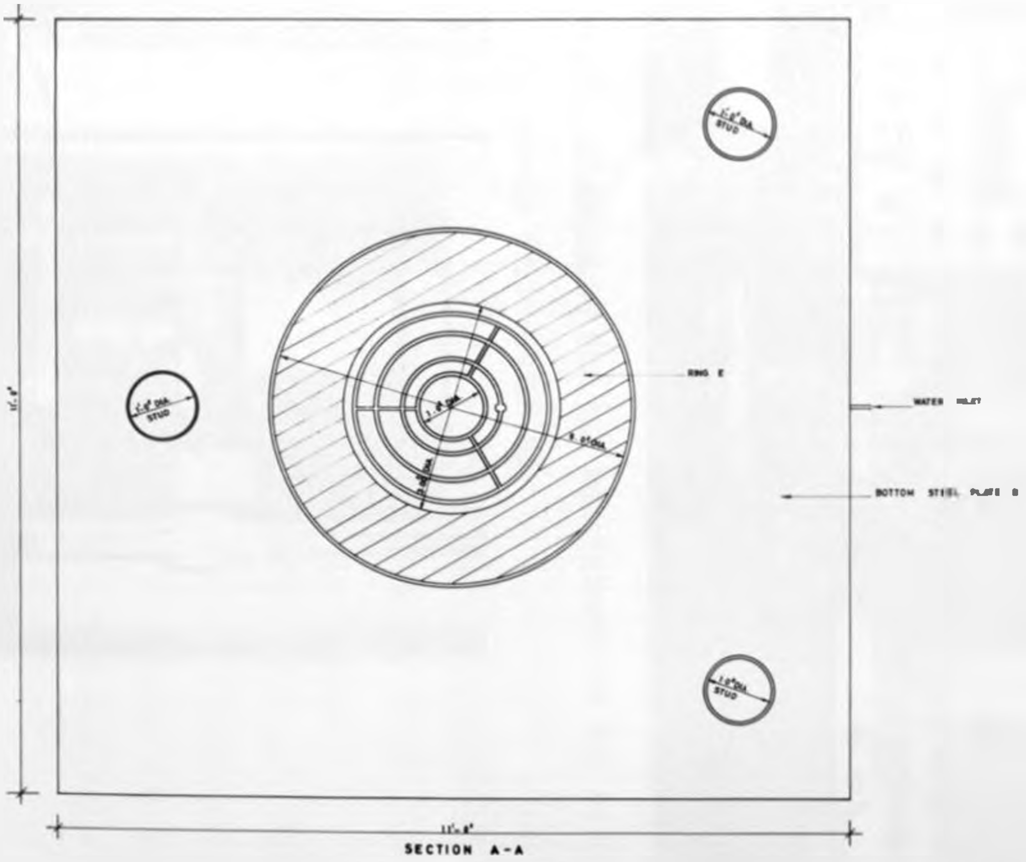

FIG. 5.74. APPARATUS FOR THE MEASUREMENT OF SWELLING PRESSURE DEVELOPED IN A SOIL UNDER CONTROLLED CONDITIONS OF LATERAL AND VERTICAL CONFINEMENT

DIRECT METHOD TEST SERIES B

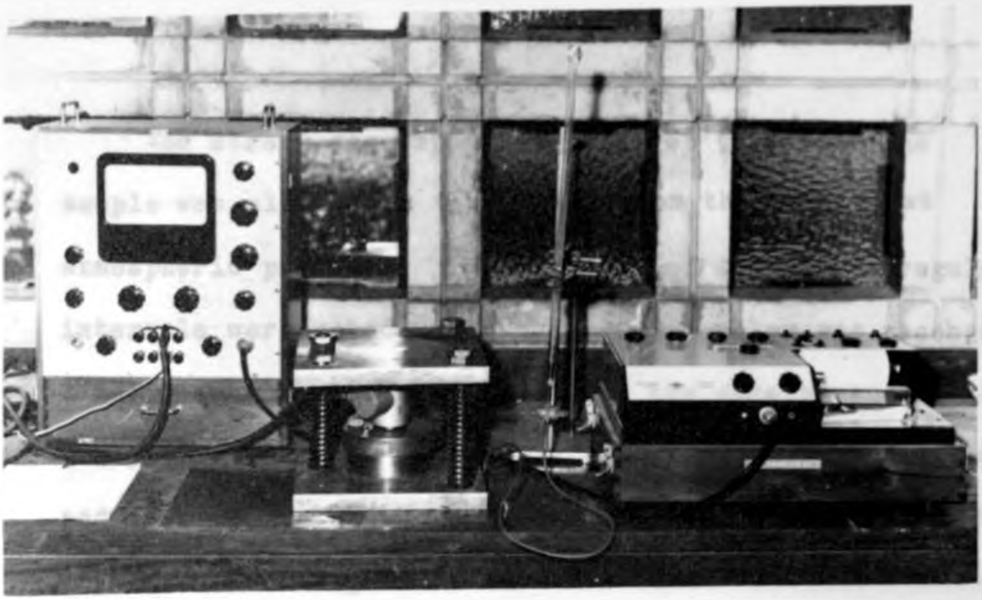

FIG.5.75. TYPICAL ARRANGEMENT FOR THE MEASUREMENT OF THE SWELLING PRESSURE DIRECT METHOD TEST SERIES B.

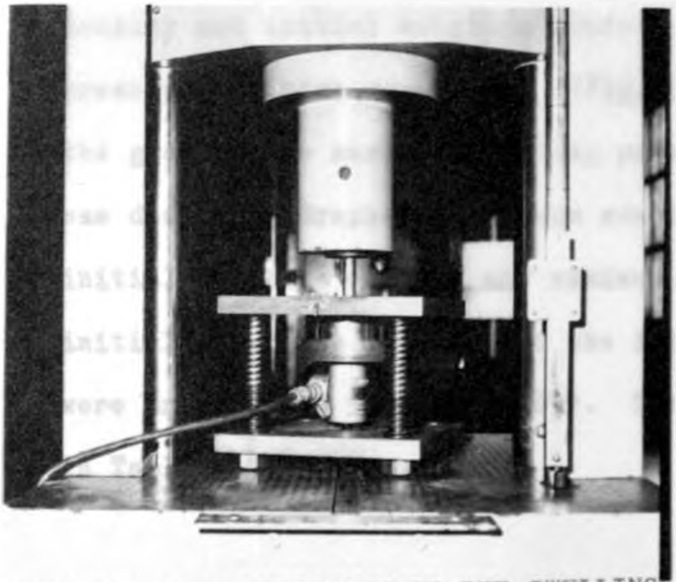

FIG.5.76 SET UP FOR THE CALIBRATION OF THE SWELLING PRESSURE APPARATUS.

DIRECT METHOD TEST SERIES B.

The strain gage apparatus was set to zero. The sample was allowed to take water from the burette at atmospheric pressure. The readings of strains at regular intervals were noted until the maximum value was reached. A recorder was also used to record the strains automatically (Fig. 5.75). The sample was then removed, weighed and left to dry in the oven at 110°C for at least 24 hours. It was then reweighed to find the final moisture content.

5.8e Results.

For each of the tests at a particular initial dry density and initial moisture content, graphs of swelling pressures v. time were drawn (Fig. 5.78 to 5.82). From the graphs, the maximum swelling pressure for each test was deduced. Graphs of maximum swelling pressures v. initial moisture content and maximum swelling pressure v. initial air-voids for each of the initial dry densities were drawn (Figs 5.83 and 5.84). The results are tabulated in Table 5.8.

Table 5.8

Results

Direct Method Test Series B-1

Initial Dry Density = 1.5 g/cc.

Test No	Wi %	Wf %	Si %	Sf %	SP max t/sq. ft.
1-1	8.9	21.2	30.9	73.5	13.9
1-2	13.4	21.9	46.5	76.0	12.8
1-3	16.6		57.5		11.2
1-4	16.8	24.1	58.5	83.5	14.80
1-5	16.8	24.1	58.5	83.5	14.30
1-6	18.10	26.0	63.0	90.1	12.8
1-7	19.3	26.7	67.0	92.5	13.5
1-8	20.2	25.1	70.0	87.0	14.1
1-9	22.4	27.6	78.0	95.5	12.8
1-10	24.5	27.5	85.0	95.5	11.3
1-11	25.6	29.0	88.5	100	9.7
1-12	27.5		95.5		9.2
1-13	27				9.4

Initial Dry Density = 1.4 g/cc.

Test No	Wi %	Wf %	Si %	Sf %	SP max t/sq. ft.
2-1	13.6	23.6	40.6	70.5	6.7
2-2	16.8	24.7	50.1	74.0	8.00
2-3	18.9	26.4	56.5	79.0	7.1
2-4	19.0	27.2	57.0	81.4	7.5
2-5	22.4	29.4	67.0	88.0	6.1
2-6	24.8	26.7	74.0	80.0	5.5
2-7	24.5	29.0	73.2	87.0	5.7
2-8	27.5	33.4	82.2	100.0	5.00
2-9	28.6	32.0	85.5	96.0	5.4
2-10	29.4	33.0	87.9	98.5	4.4
2-11	30.7	32.6	92.0	97.5	5.0

Initial Dry Density = 1.33 g/cc.

Test	Wi	Wf	Si	Sf	SP max
3-1	13.7	25.7	36.8	69.1	3.45
3-2	16.8	26.4	45.2	71.0	4.45
3-3	18.5	27.3	49.8	73.2	4.65
3-4	22.5	29.0	60.5	78.0	3.62
3-5	24.5	27.5	66.0	74.0	4.05
3-6	25.0	29.7	67.2	80.0	3.50
3-7	27.4	32	73.5	86.0	3.06
3-8	28.0	32.5	75.2	87.5	2.84
3-9	29.2	36.0	78.5	97.0	1.89
3-10	31.7	35.4	85.0	95.2	2.50
3-11	32.5	34.8	87.5	93.5	2.70

Initial Dry Density = 1.2 g/cc

Test No	Wi %	Wf %	Si %	Sf %	SP max t/sq.ft.
4-1	16.8	28.2	36.8	62.0	1.95
4-2	18.5	28.6	40.6	62.8	1.56
4-3	22.5	34	49.4	74.5	0.94
4-4	24.0	36	52.5	79.0	0.78
4-5	27.3	38	60.0	83.5	0.45
4-6	27.8	37.4	61.0	82.0	0.43
4-7	30.7	38.0	67.2	83.5	0.33
4-8	32.6	43.0	71.5	94.2	0.13

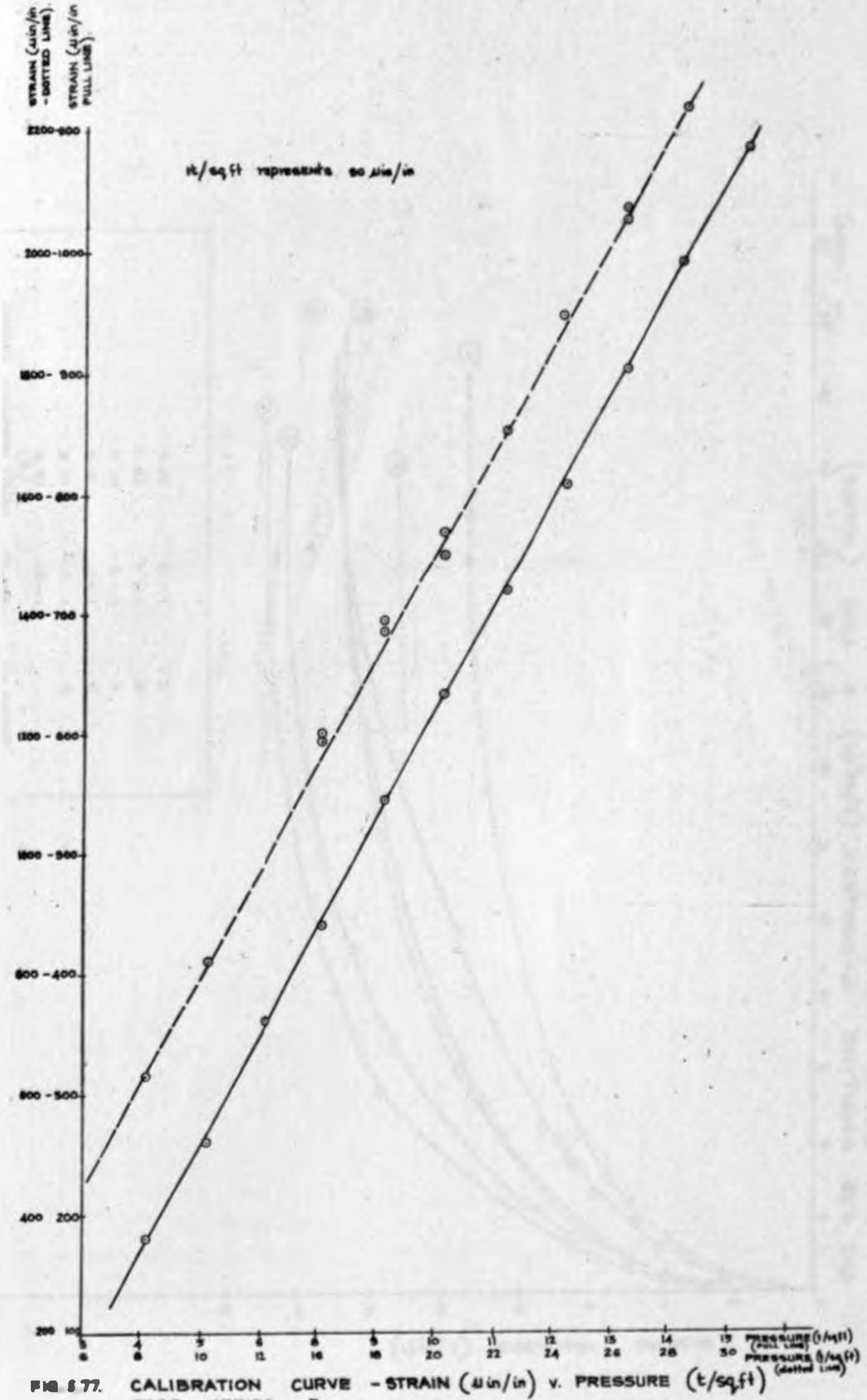

FIG. 8.77. CALIBRATION CURVE - STRAIN (μin/in) v. PRESSURE (t/sq ft)
TEST SERIES B

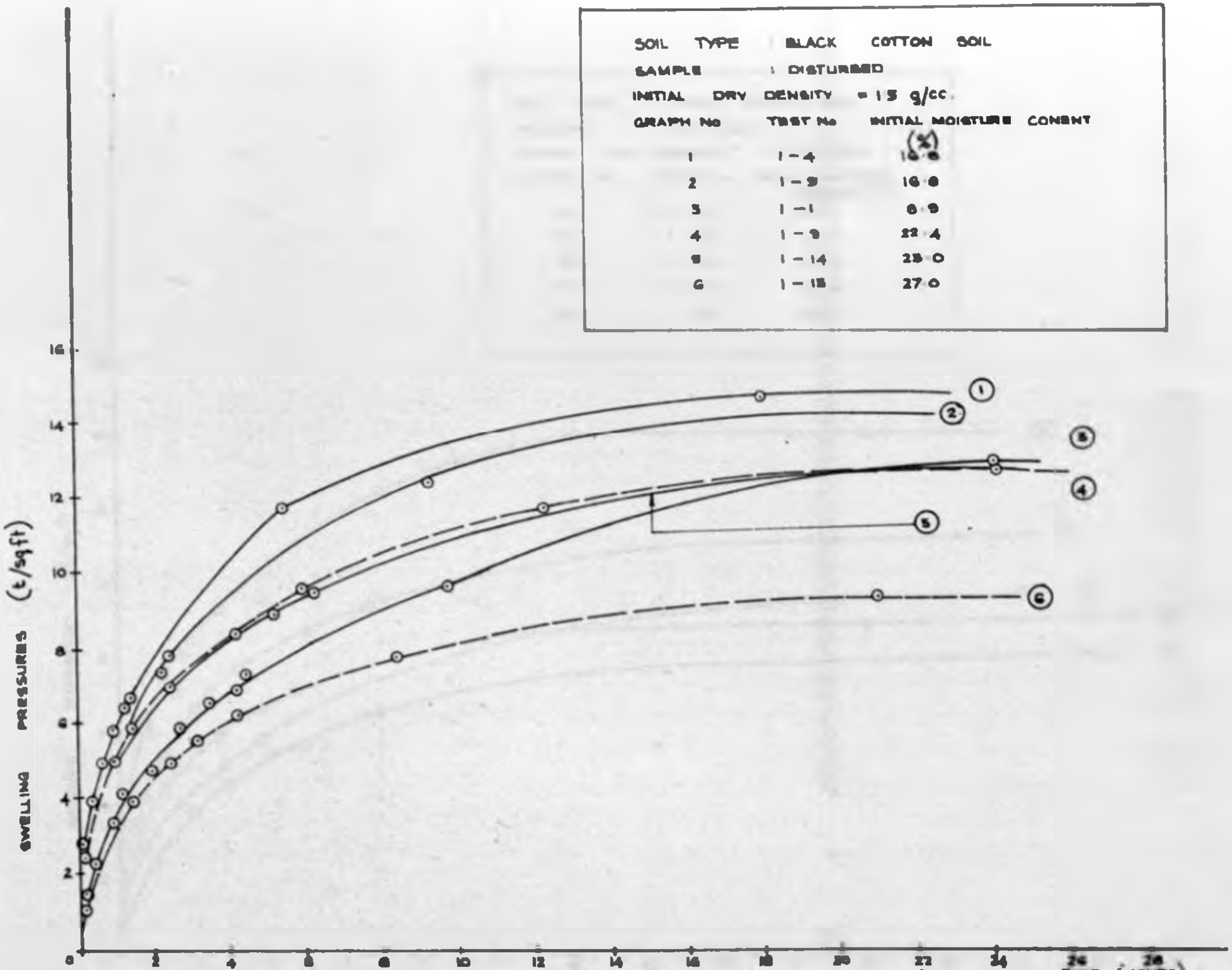

FIG 4.78 SWELLING PRESSURES (t/sq ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B - 1

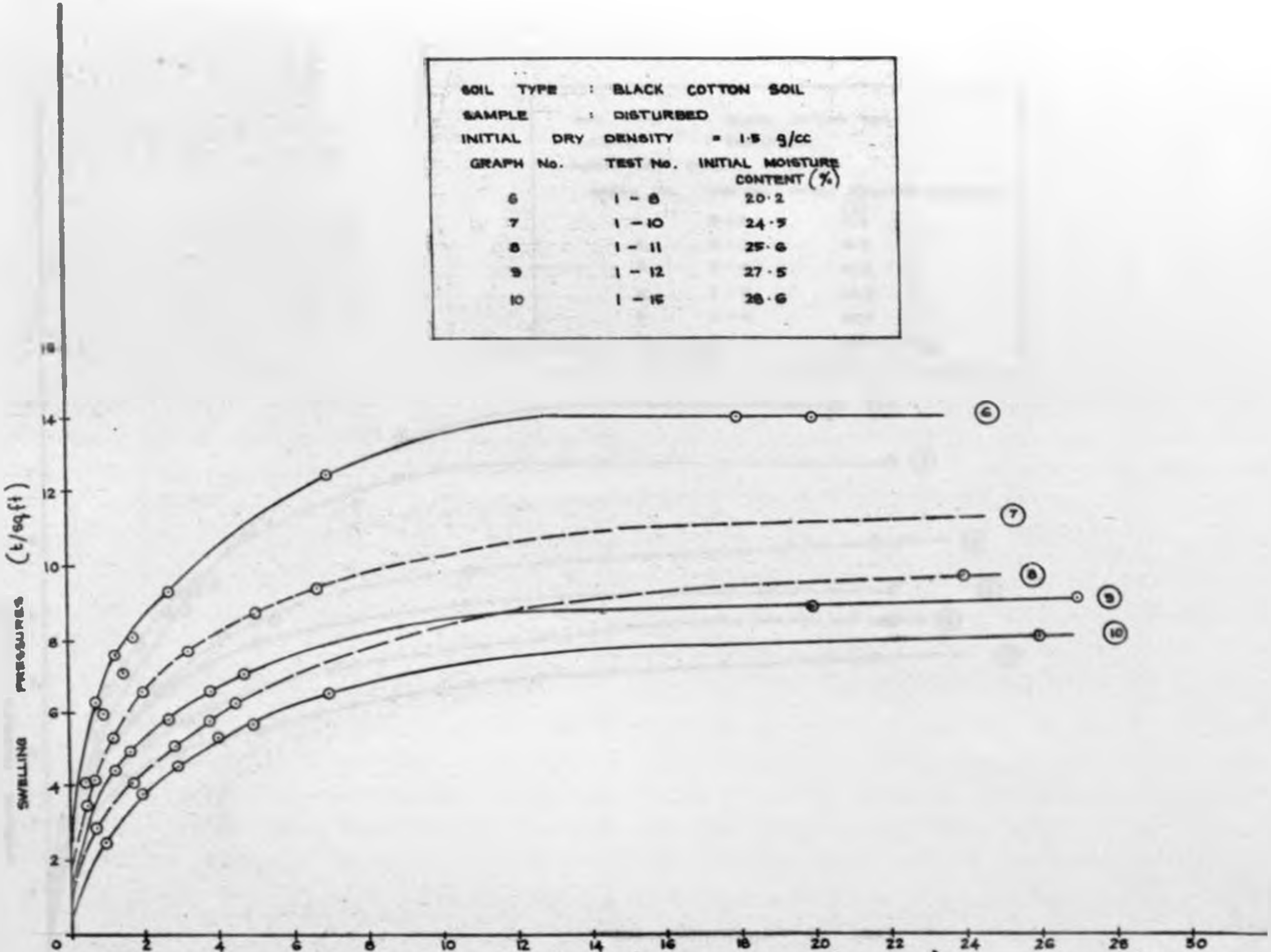

FIG. 5·79. SWELLING PRESSURES (t/sq.ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B-1.

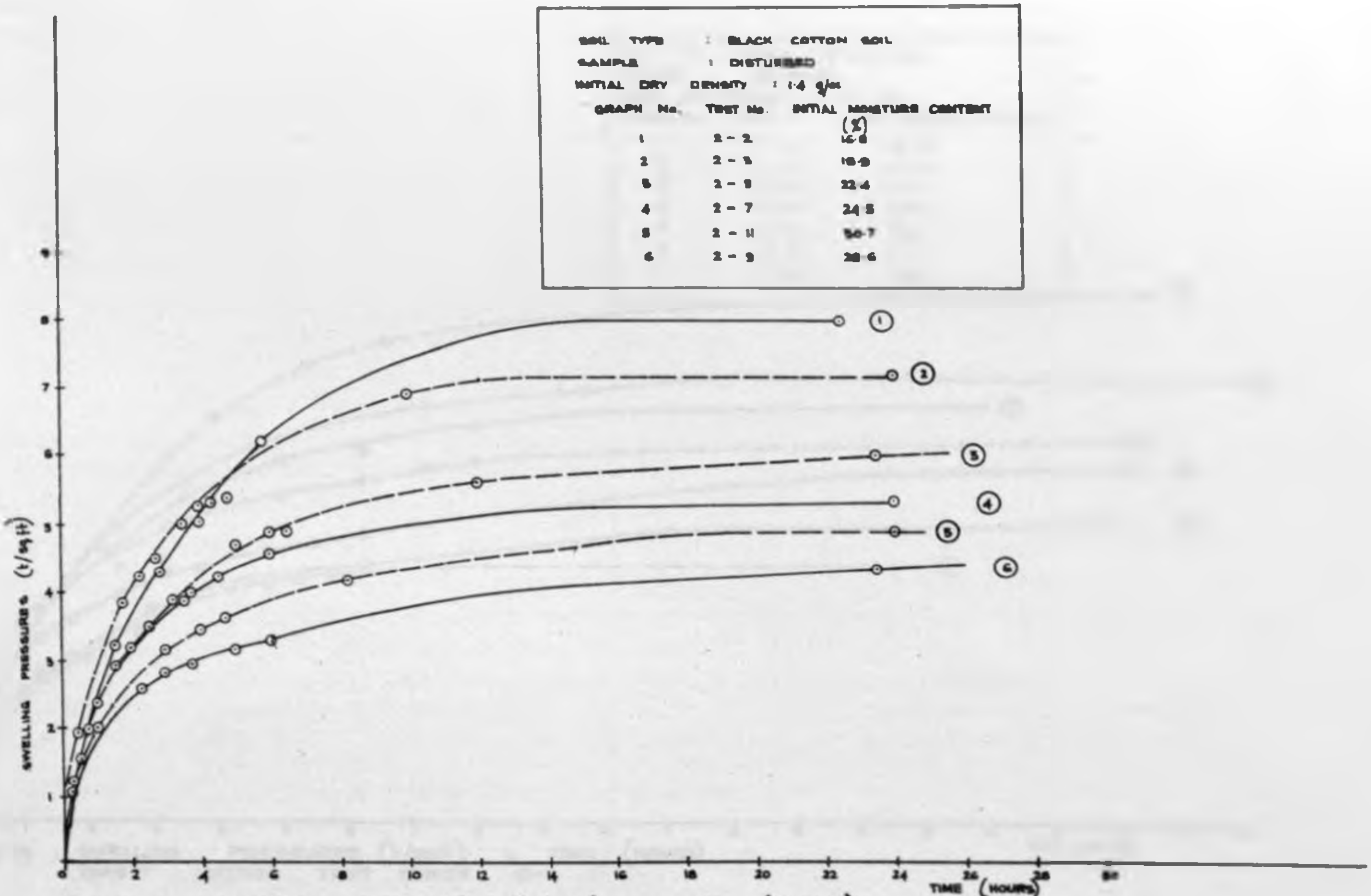

FIG. 5.80. SWELLING PRESSURES (t/sq ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B-1

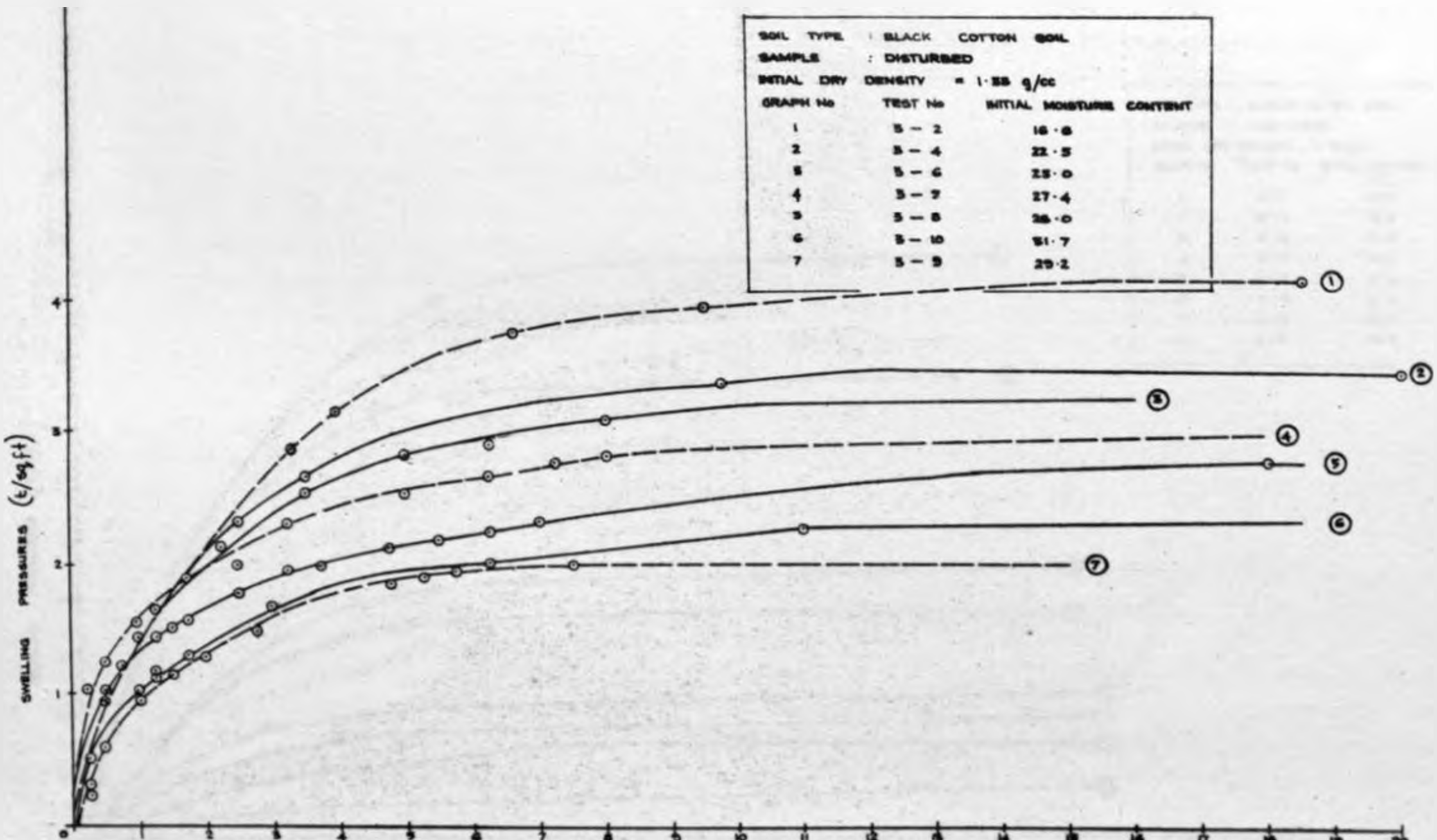

FIG. 5.61 SWELLING PRESSURES (t/sqft) v TIME (HOURS)
DIRECT METHOD TEST SERIES S-1

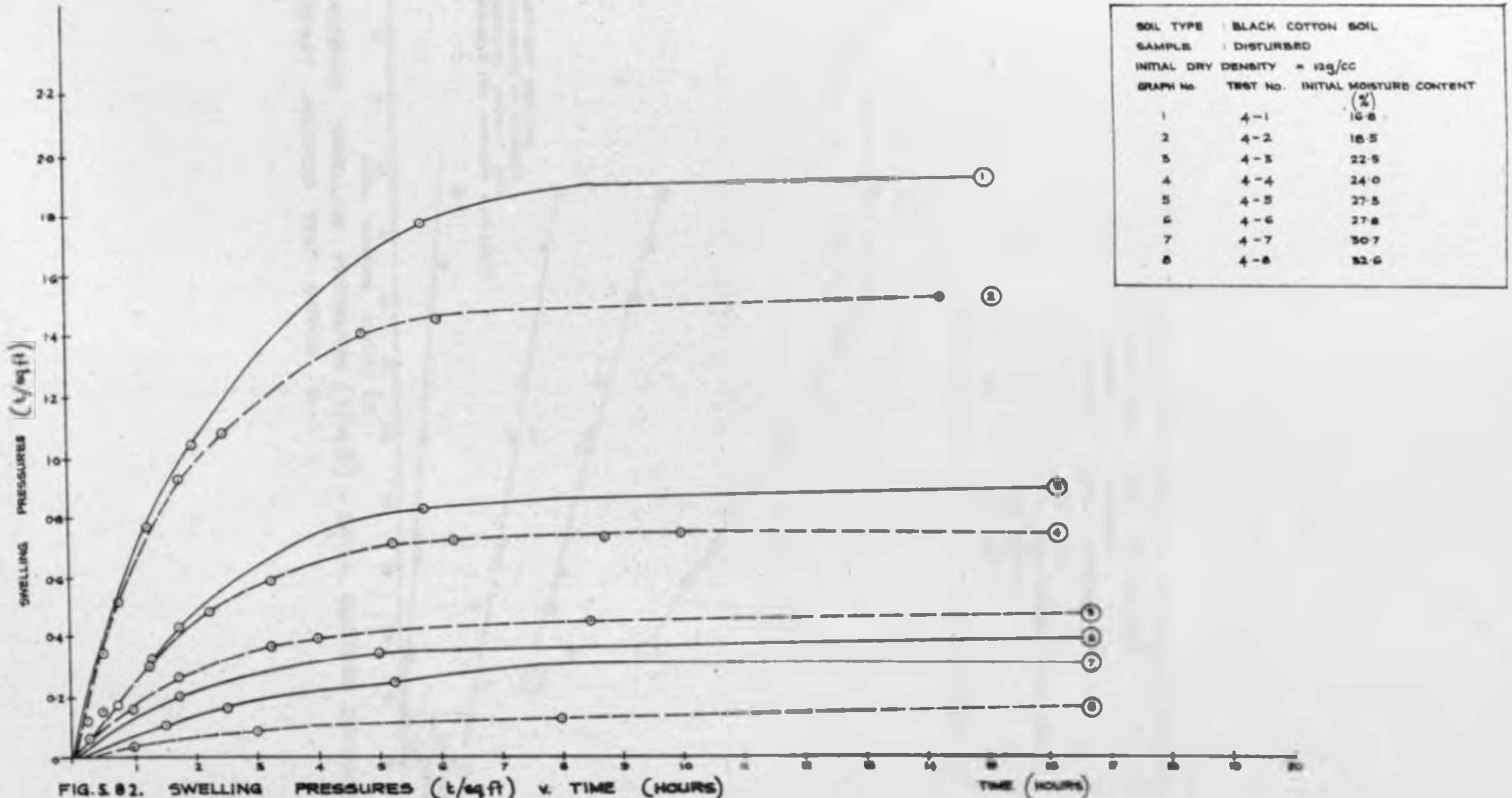

FIG. 5.82. SWELLING PRESSURES (t/sq ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B-1

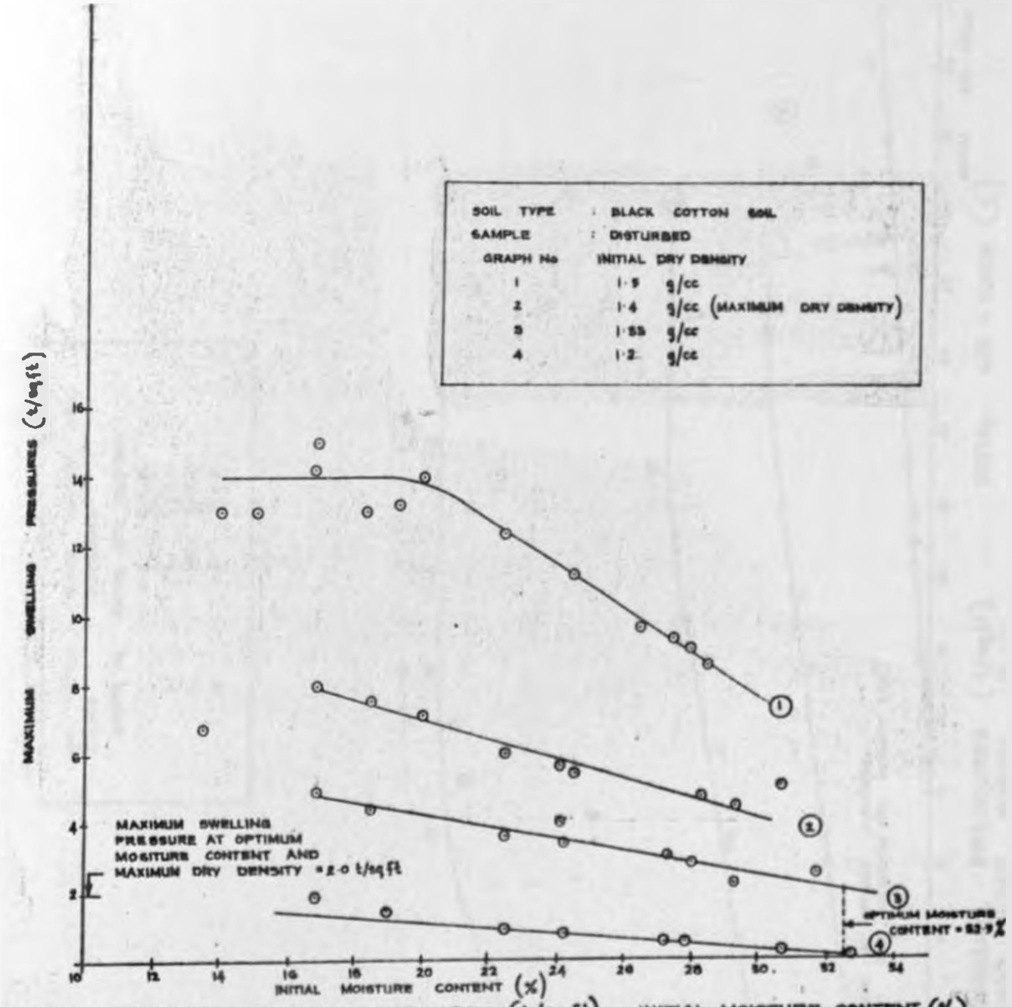

FIG. 5·65. MAXIMUM SWELLING PRESSURES (t/sq ft) v. INITIAL MOISTURE CONTENT (%) DIRECT METHOD TEST SERIES B-1

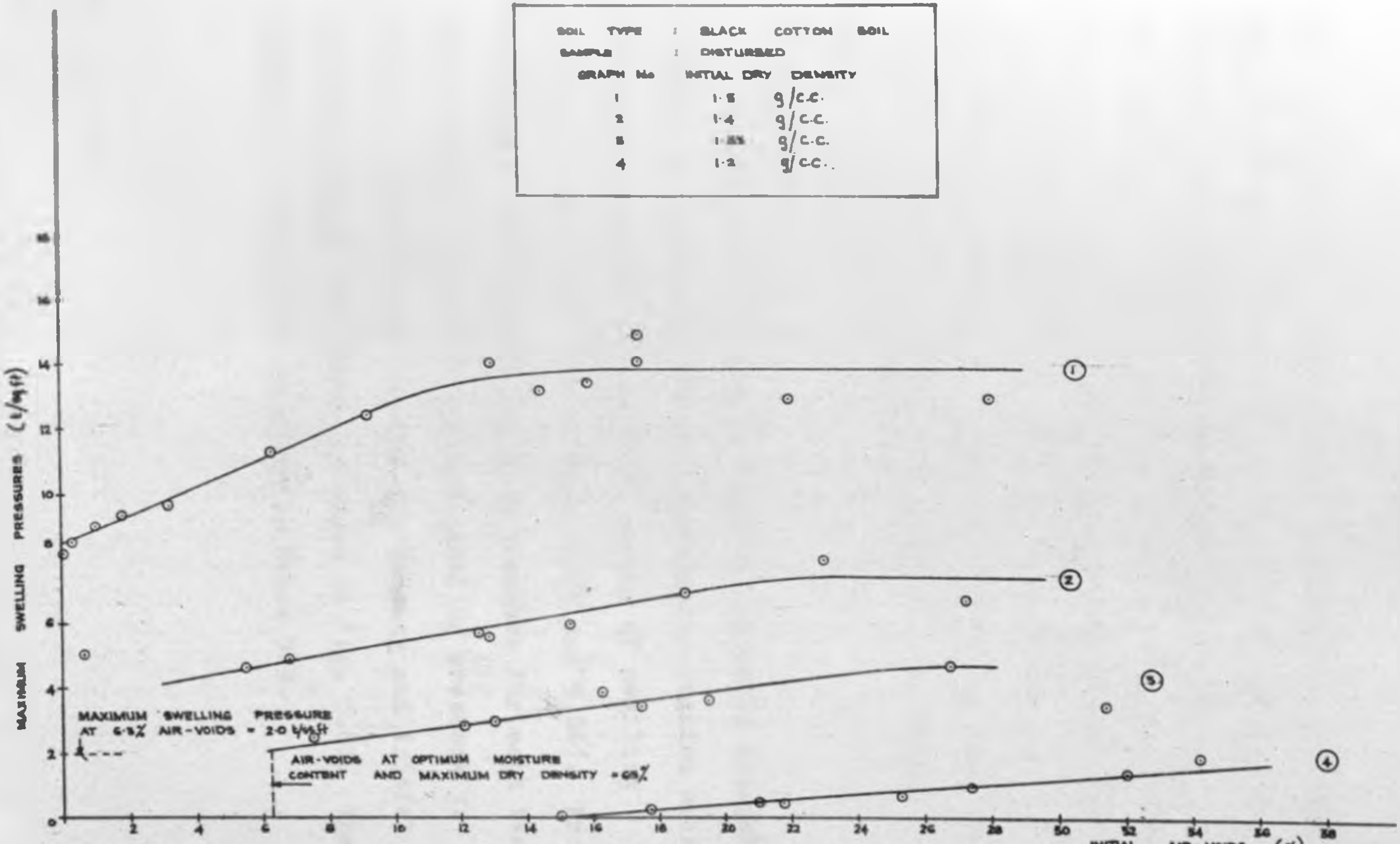

FIG. 5.84. MAXIMUM SWELLING PRESSURES (t/sqft) v. INITIAL AIR - VOIDS (%)
DIRECT METHOD TEST SERIES B - 1

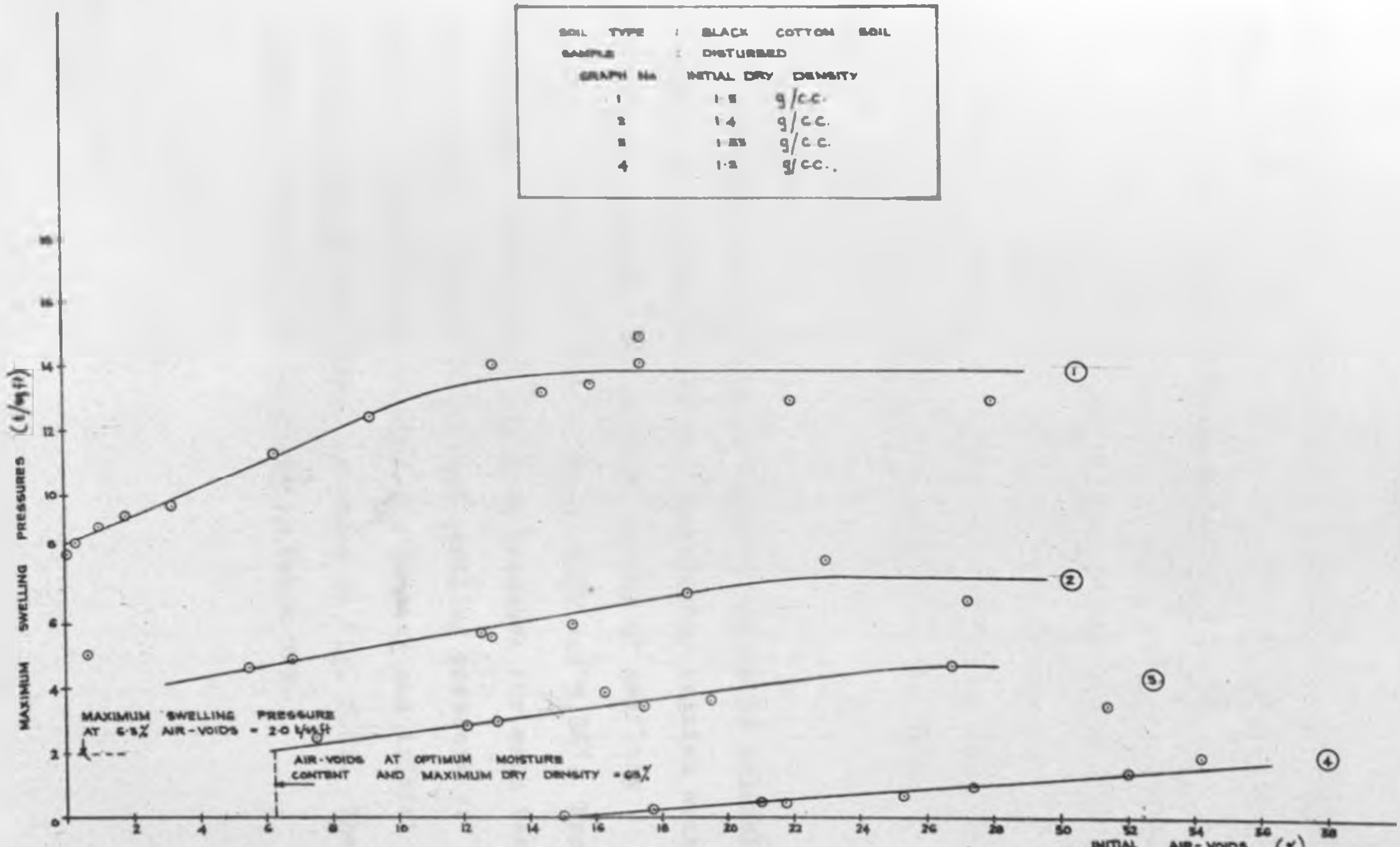

FIG. 5.84. MAXIMUM SWELLING PRESSURES (t/sq ft) v. INITIAL AIR - VOIDS (%)
DIRECT METHOD TEST SERIES B - 1

5.9 Direct Method. Test Series B-2. Tests on disturbed samples of Black Cotton Soil, to study the influence of height of sample on the maximum swelling pressure.

These tests were carried out at initial dry density of 1.4 g/cc. and initial moisture content of 20%, but the height of the sample was varied from 0.35 inch to 0.75 inch in equal intervals of 0.1 inch. The apparatus used and testing procedure adopted, was similar to the Direct Method, Test Series B-1.

5.9a Results.

For each of the tests at various values of initial height of the specimen and at a particular initial moisture content and initial dry density, graphs of swelling pressure v. time were drawn (Figs. 5.85 and 5.86). From these graphs the maximum swelling pressure for each test was deduced. A graph of maximum swelling pressure v. height at a particular initial dry density and initial moisture content was drawn, as shown in Fig. 5.87. The summary of the results is shown in Table 5.9.

Table 5.9

Summary of Results

Direct Method Test Series B-2

Initial dry density = 1.4 g/cc. Initial moisture content = 20.0%		
Test No	Ht. of sample (IN)	Maximum Swelling Pressure (t/sq.ft.)
1	0.65	6.1
2	0.55	5.45
3	0.45	4.11
4	0.35	3.00
5	0.65	6.0
6	0.55	4.8
7	0.45	4.5

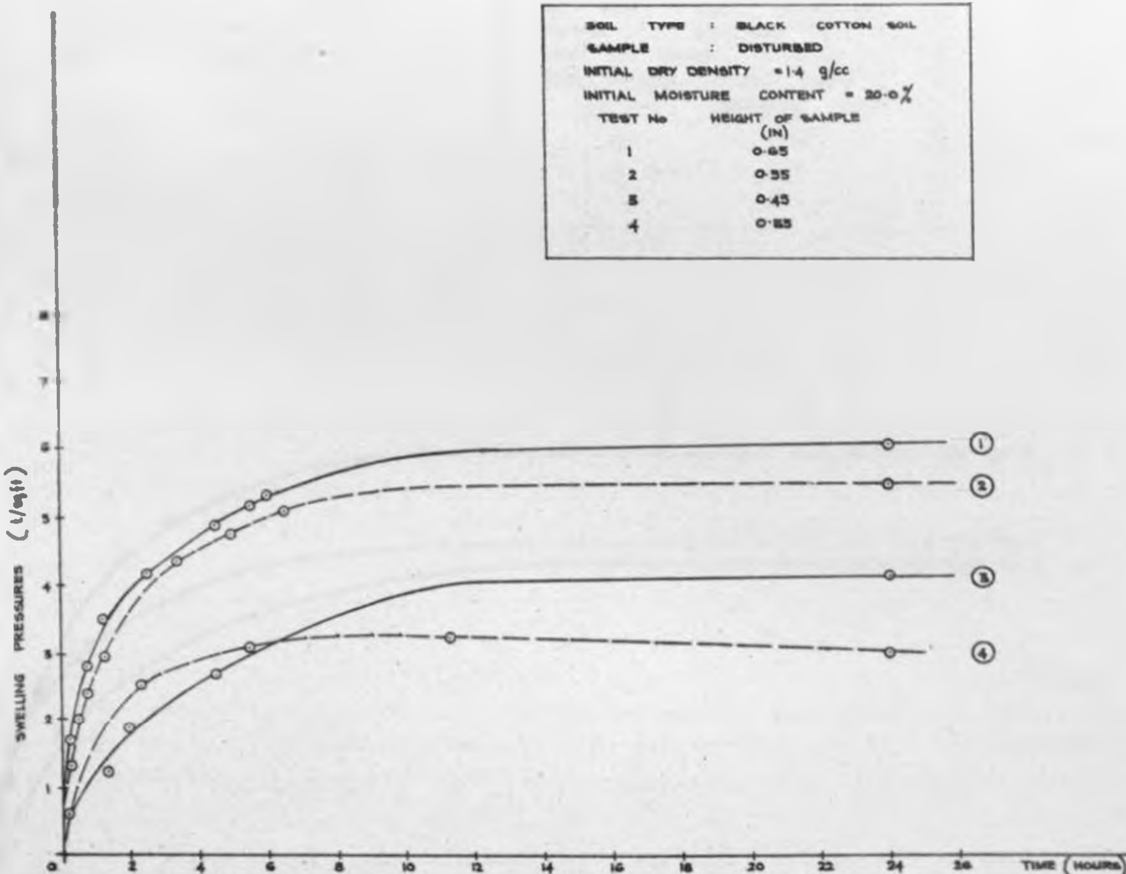

FIG. 5.85. SWELLING PRESSURES (t/sq ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B-2

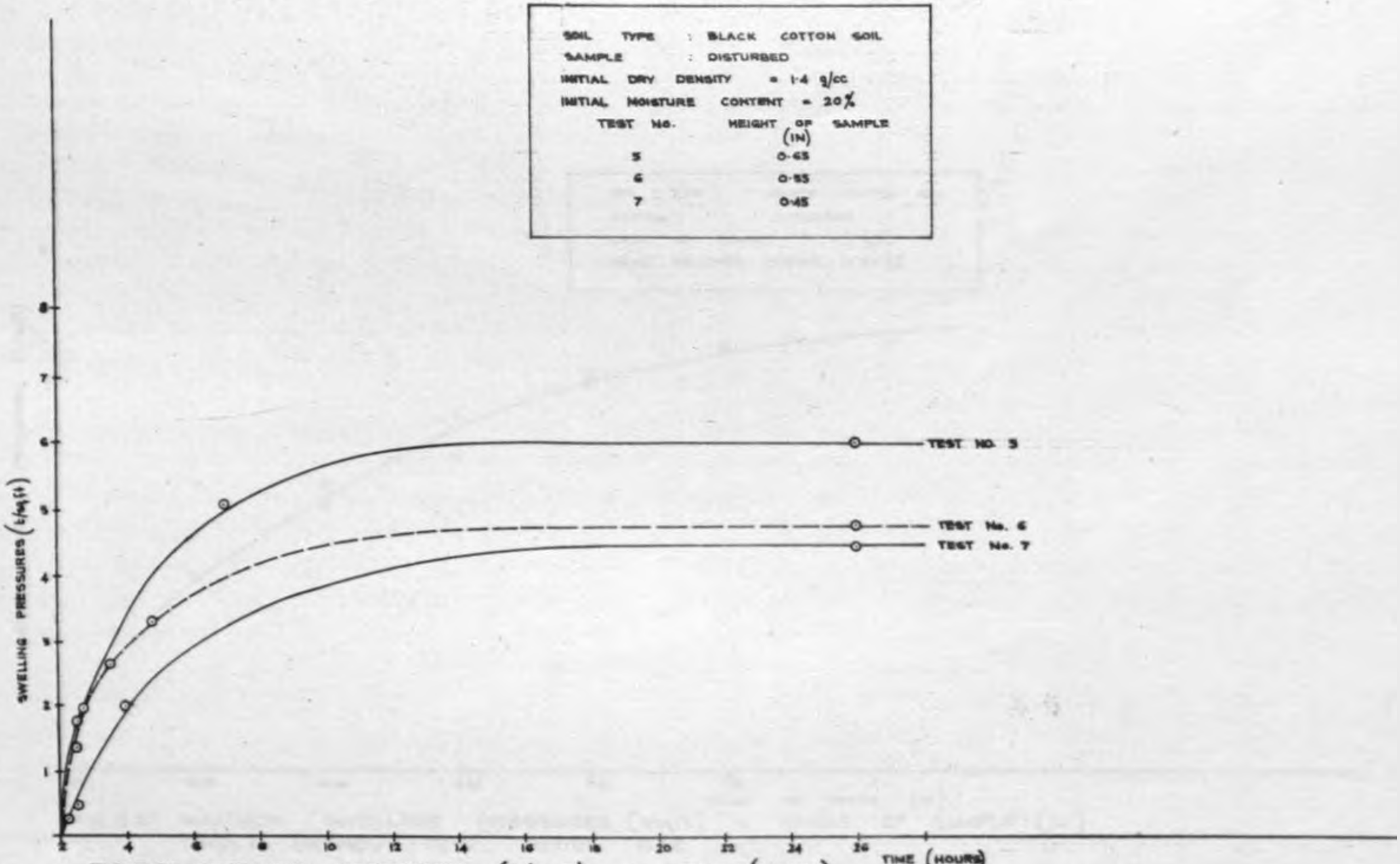

FIG. 5.86. SWELLING PRESSURES (t/sq ft) v. TIME (HOURS)
DIRECT METHOD TEST SERIES B - 2

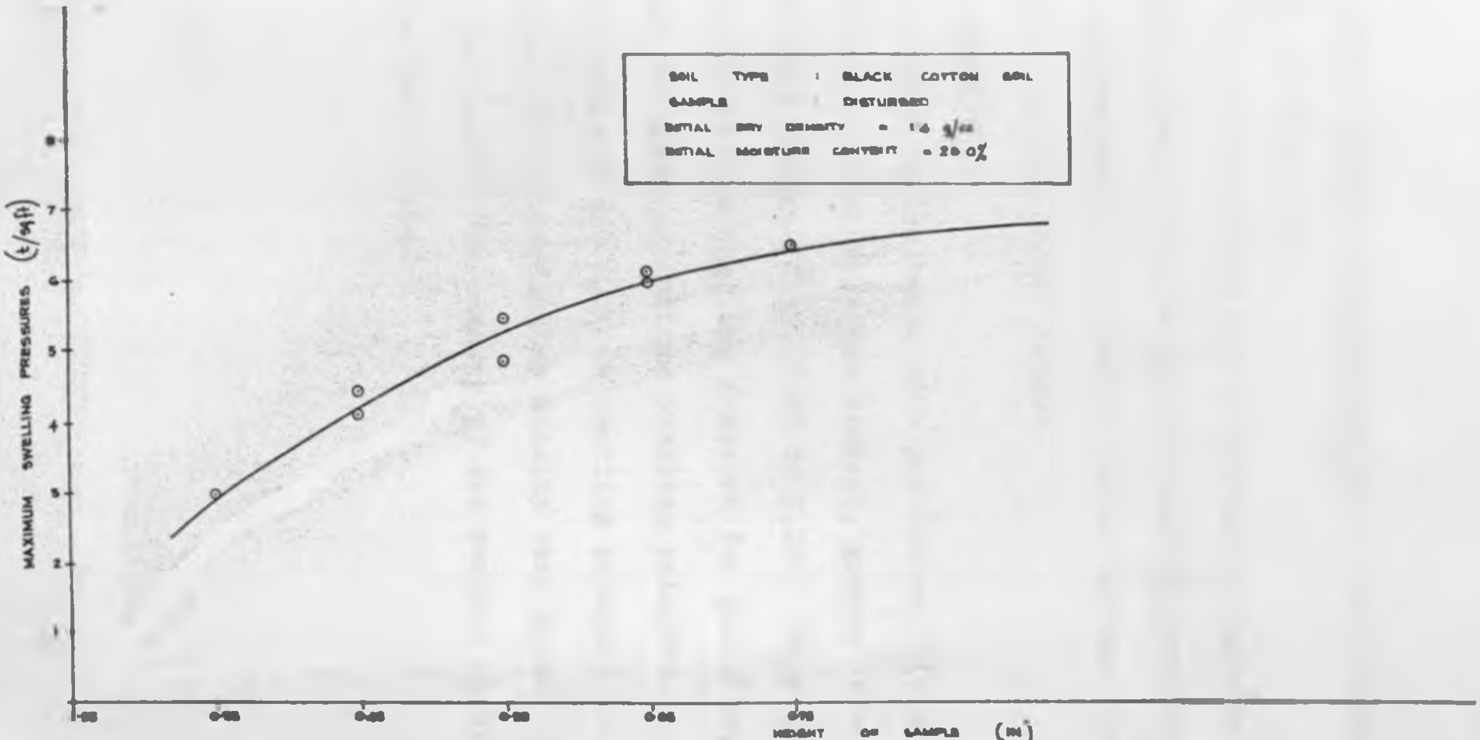

FIG 5.67. MAXIMUM SWELLING PRESSURES (t/sqft) v. HEIGHT OF SAMPLE (IN.)
DIRECT METHOD TEST SERIES B-2

5.10 Direct Method. Test Series B-3: Tests on disturbed samples of Green Clay.

Tests were carried out at initial dry density of 1.02 g/cc, and at various initial moisture contents, using the specially designed apparatus and the normal procedure for the Direct Method.

5.10a Results.

For each of the tests at a particular initial dry density and initial moisture content, graphs of strain v. time were drawn (Figs. 5.88 to 5.101) From the graphs the maximum swelling pressure for each test was deduced. A graph of maximum swelling pressure v. initial moisture content and maximum swelling pressure v. initial air-voids for the particular density were drawn (Figs. 5.102 and 5.103). The summary of the results obtained is shown in Table 5.10.

Table 5.10

Summary of Results

Direct Method Test Series B-3

Initial Dry Density = 1.02 g/cc					
Test No	Wi %	Wf %	Si %	Sf %	SP max t/sq. ft.
1	38.0	51.4	63.0	85.5	3.34
2	37.0	56.0	61.5	93.0	3.56
3	40.3	58.5	67.0	97.0	2.94
4	42.5	60.5	71.0	100	2.61
5	43.0	63.8	72.0	100	2.80
6	45.0	59.8	75.0	100	2.40
7	46.8	62.0	78.0	100	2.30
8	48.0	61.5	80.0	100	2.10
9	49.0	60.00	81.5	100	1.83
10	49.5	61	82.1	100	1.94
11	52.0	61.5	87.5	100	1.84
12	55.0		95.0	100	1.30

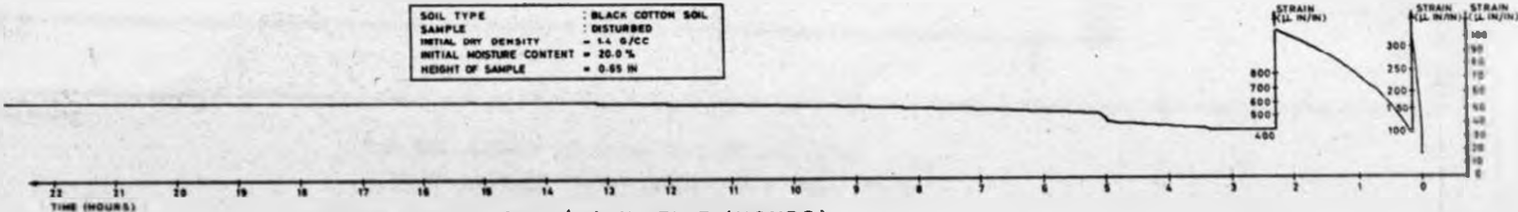

FIG. 5.88. STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-2 TEST No. 1

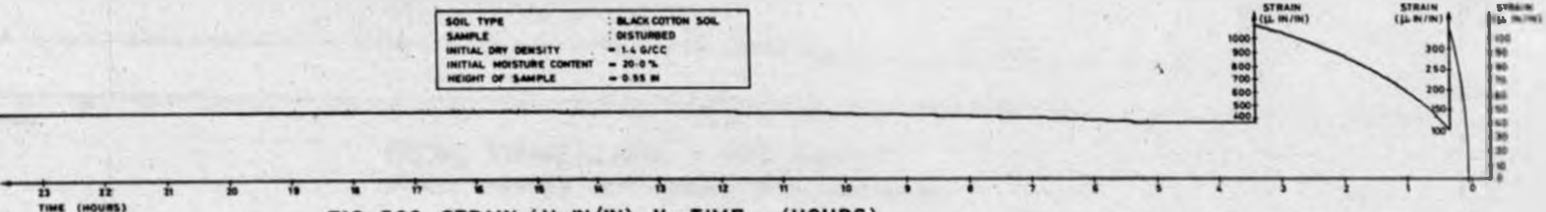

FIG. 5.89. STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-2 TEST No. 2

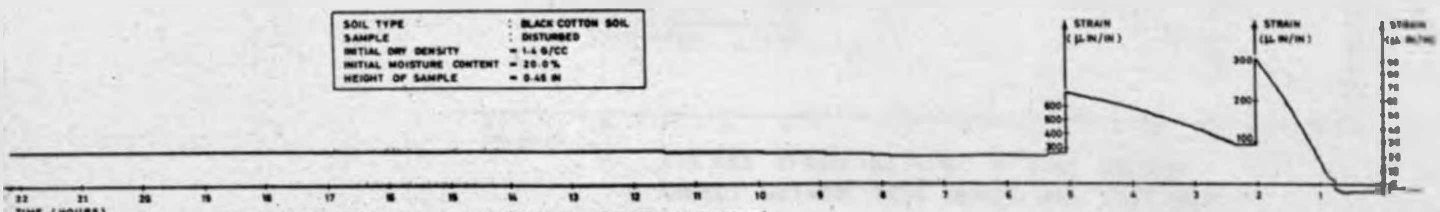

FIG. 5.90. STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-2 TEST No. 3

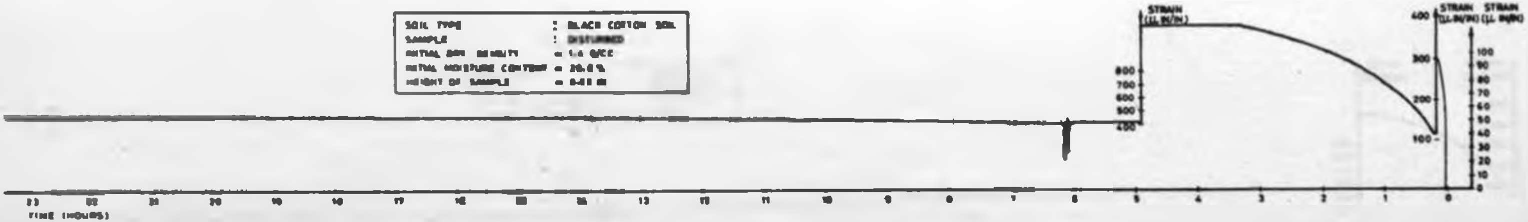

FIG. 5.91 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-2 TEST No. 5

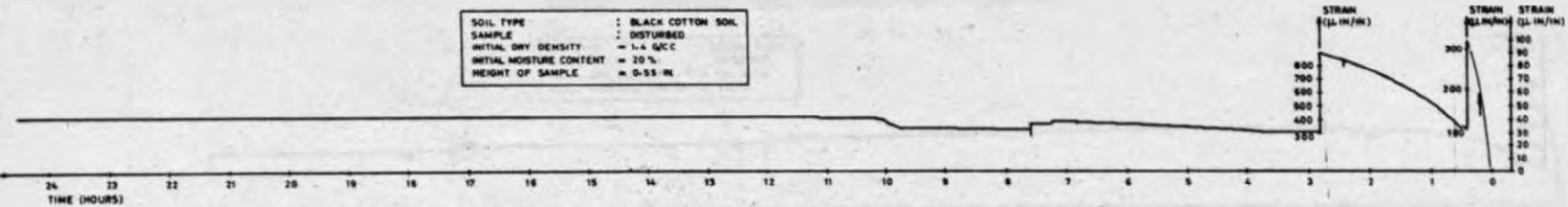

FIG. 5.92 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-2 TEST No. 6

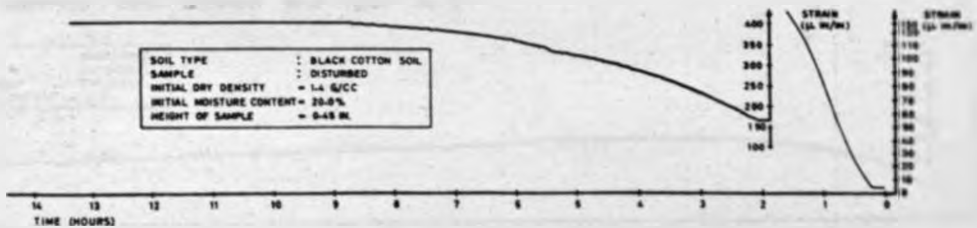

FIG. 5.93 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD. TEST SERIES B-2 TEST No. 7

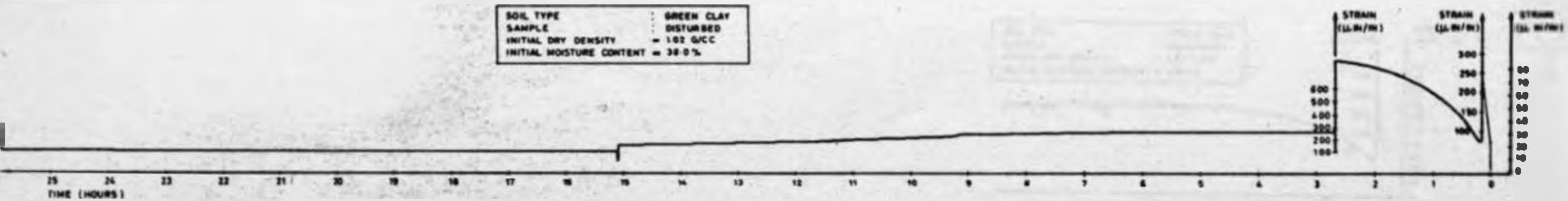

FIG. 5.94 STRAIN (µ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 1

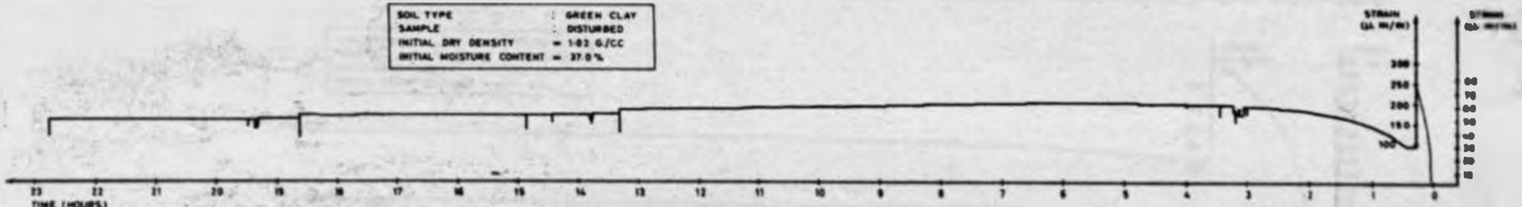

FIG. 5.95 · STRAIN (µ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 2

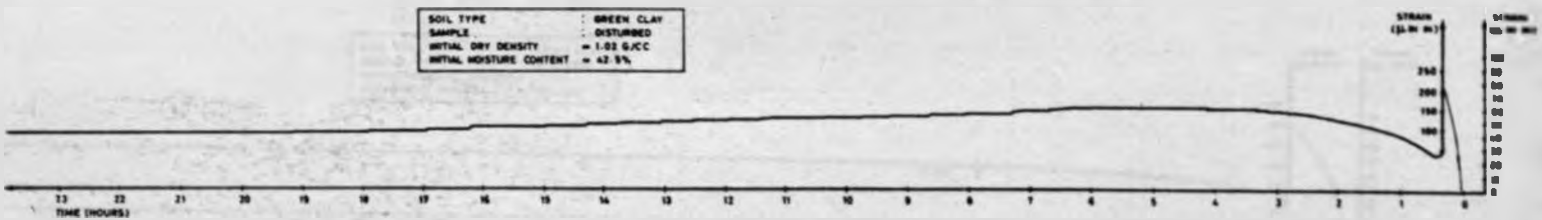

FIG. 5.96 STRAIN (µ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 4

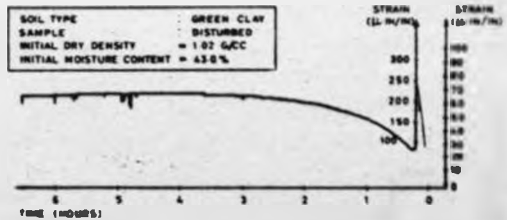

FIG. 5.97 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 5

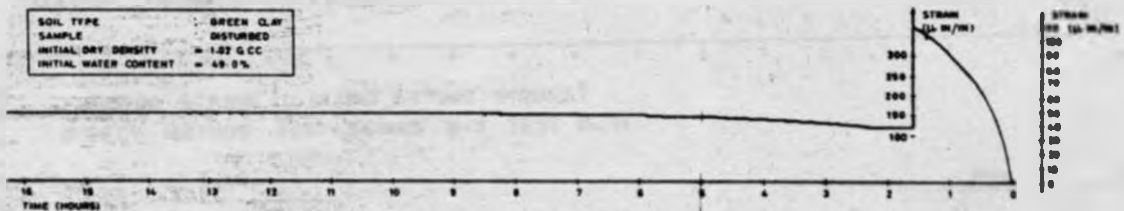

FIG. 5.98 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 9

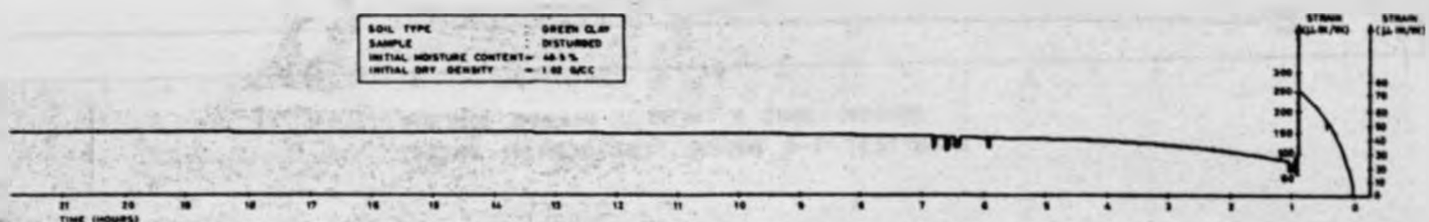

FIG. 5.99 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 10

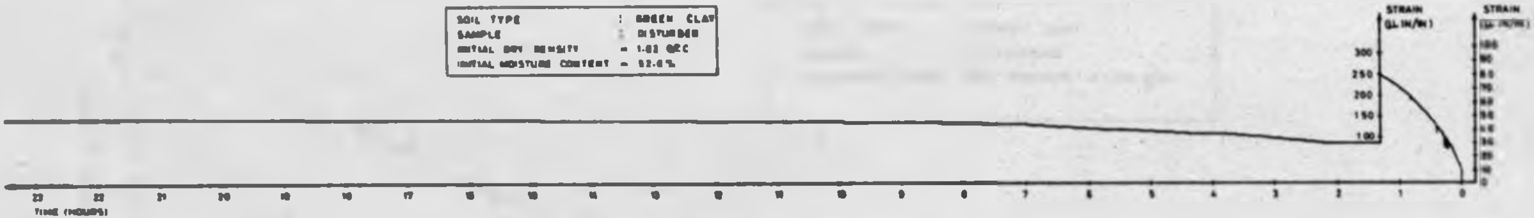

FIG. 5.100 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 11

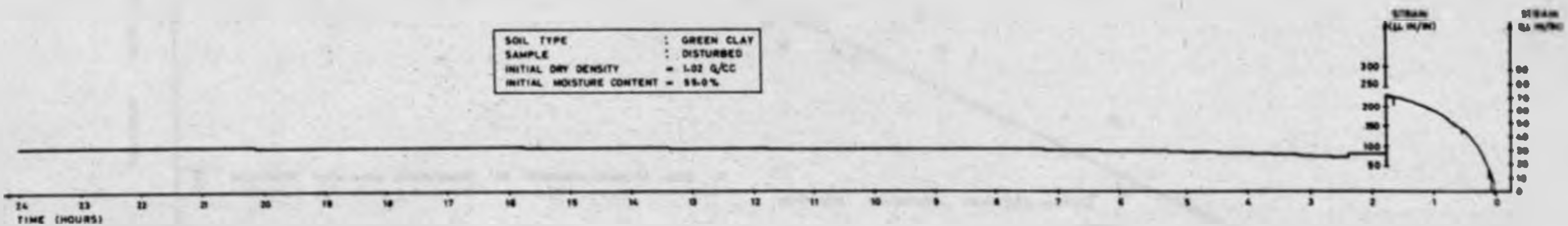

FIG. 5.101 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-3 TEST No. 12

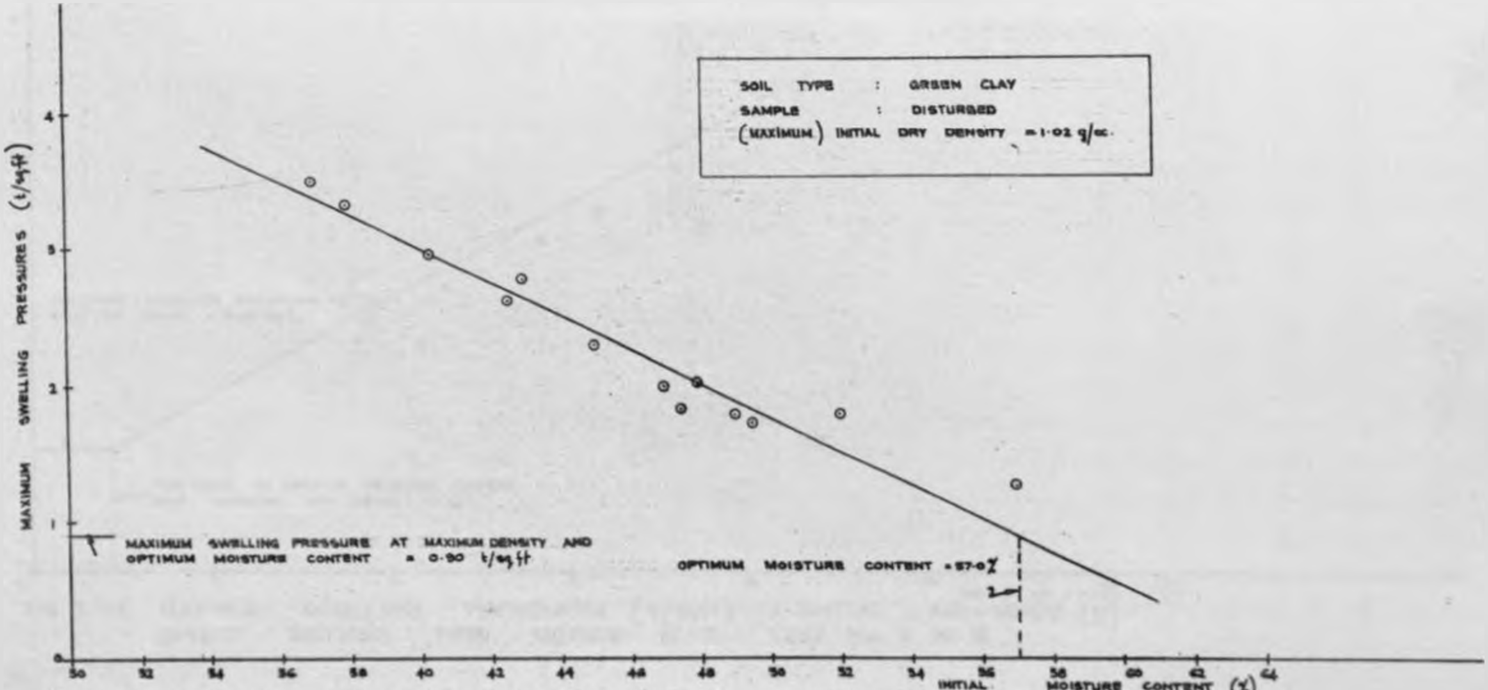

FIG 5.102. MAXIMUM SWELLING PRESSURES (t/sq ft) v. INITIAL MOISTURE CONTENT (%)
DIRECT METHOD TEST SERIES B-3 TEST No 1TO12

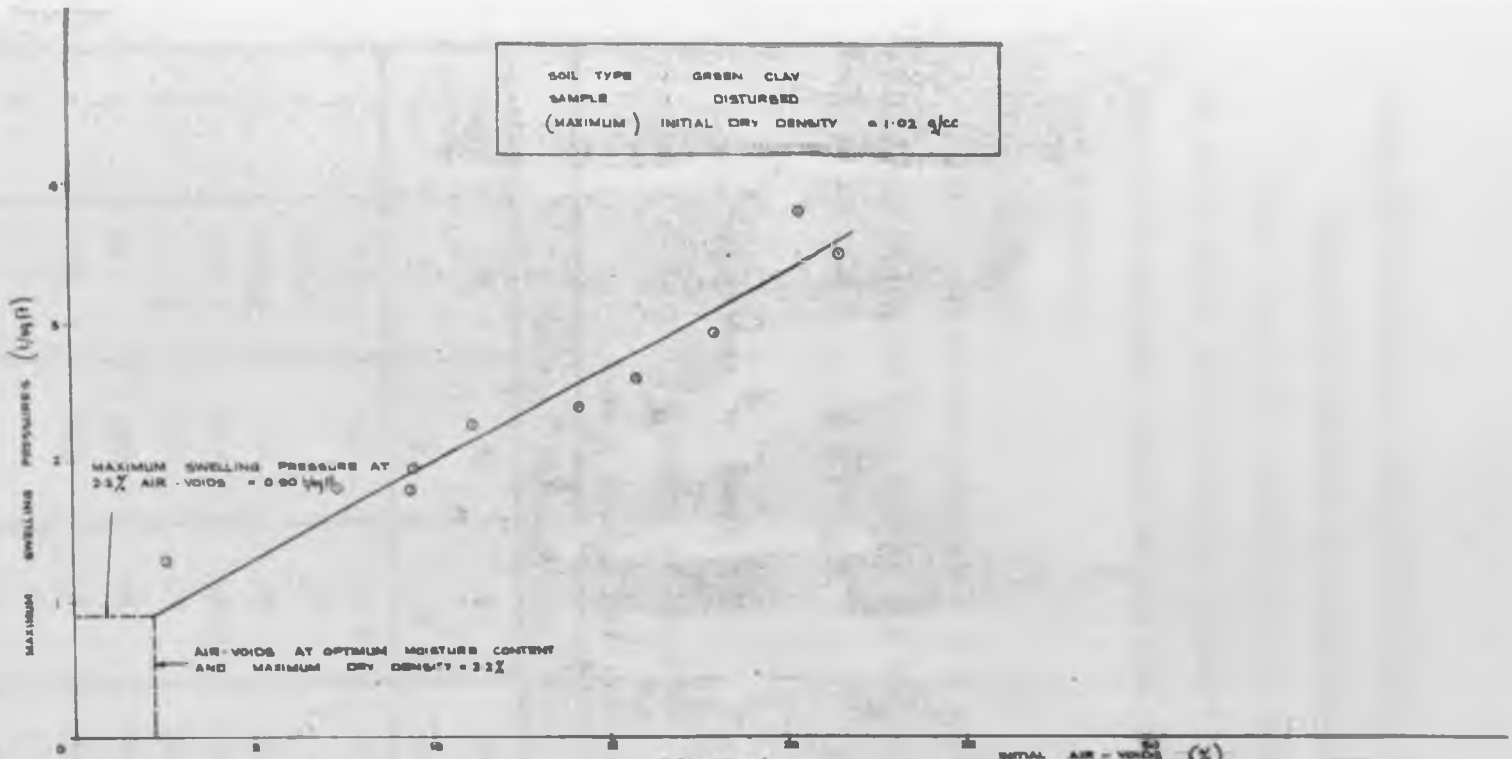

FIG. 5.103. MAXIMUM SWELLING PRESSURES (t/sqft) v. INITIAL AIR-VOIDS (%)
DIRECT METHOD TEST SERIES B-5 TEST No 1 TO 12

5.11 Direct Method. Test Series B-4. Tests on disturbed samples of sepiolite clay.

These tests were carried out at initial dry density of 0.476 g/cc. and at various moisture contents, using the specially designed apparatus and normal procedure for the Direct Method.

5.11a Results.

The results were evaluated in a similar manner as the Direct Metrod Test Series B-3 and are shown in Table 5.11 and Figs. 5.104 to 5.114.

Table 5.11

Summary of Results.

Direct Method Test Series B-4

Initial Dry Density = 0.476 g/cc. (Maximum)					
Test No	Wi %	Wf %	Si %	Sf %	SP max t/sq. ft.
1	90.5	161	50.5	90	1.11
2	103.7		58.0		0.96
3	107.5	158.2	60.0	89	0.62
4	108.0	167	60.5	93.5	1.02
5	117.5	167	66	93.5	0.60
6	122	158	68	89	0.45
7	128	163	72	91.5	0.40
8	131		73		0.35
9	139	161	76	90	0.24

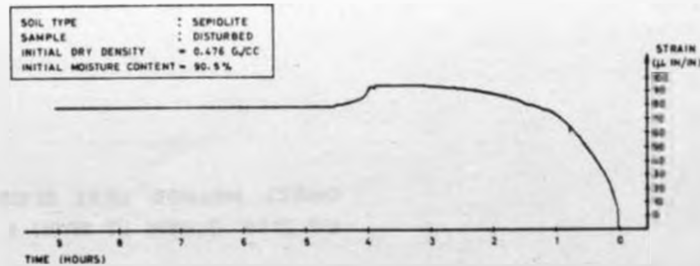

FIG. 5.104 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 1

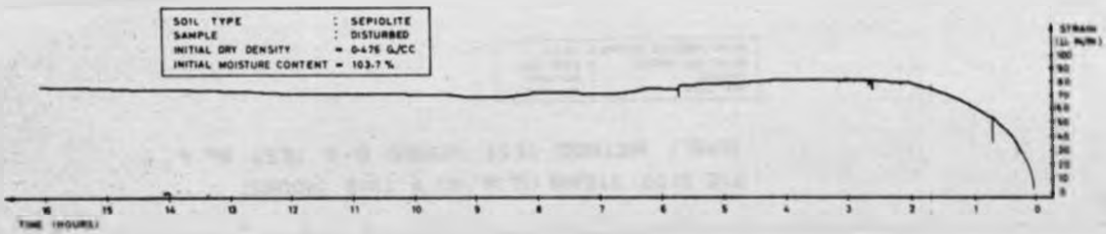

FIG. 5.105 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 2

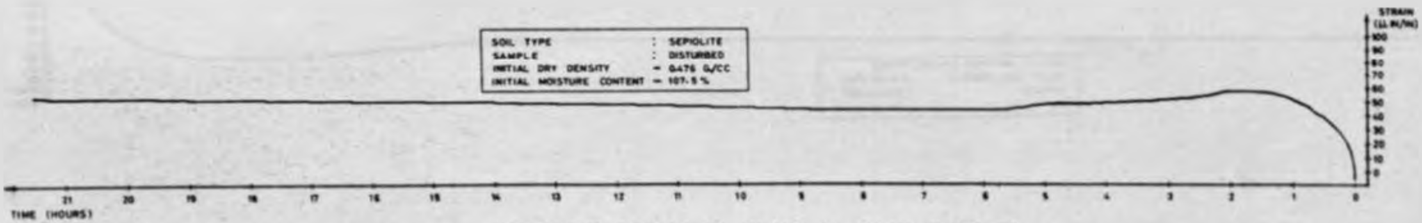

FIG. 5.106 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 3

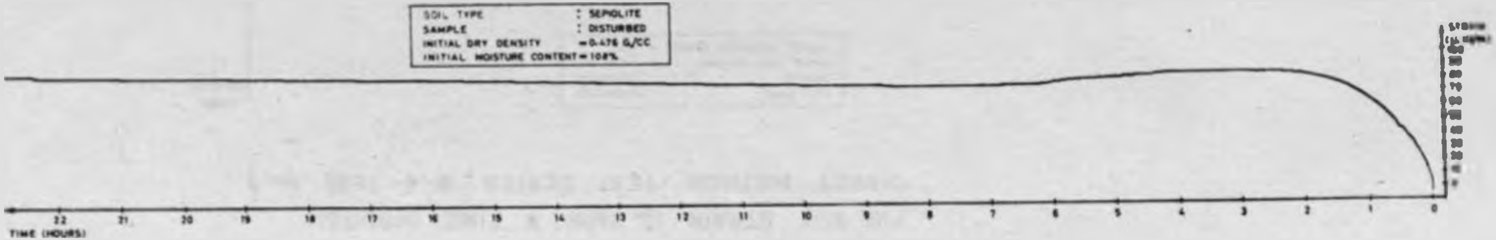

FIG. 5.107 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 4

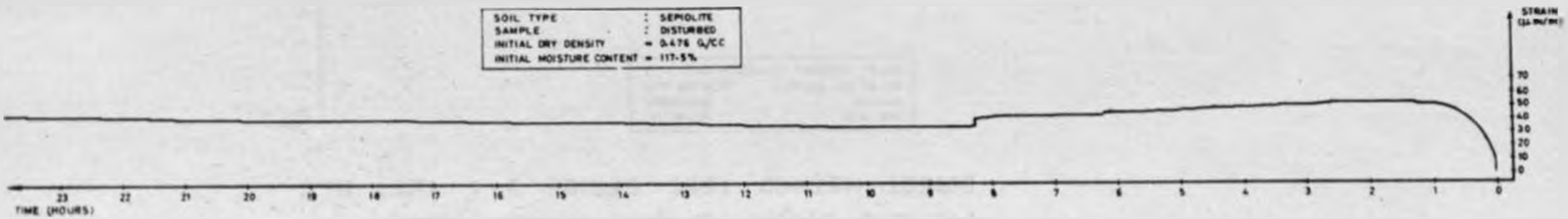

FIG. 5.108 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 5

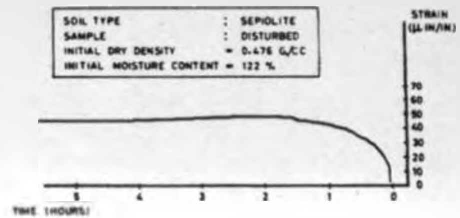

FIG. 5.109 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 6

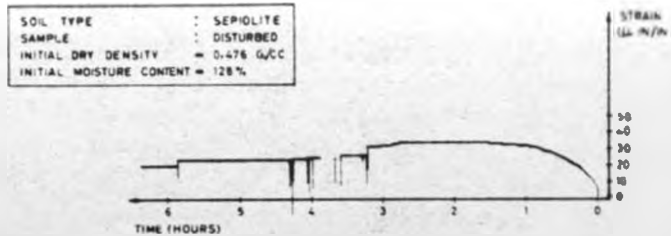

FIG. 5.110 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 7

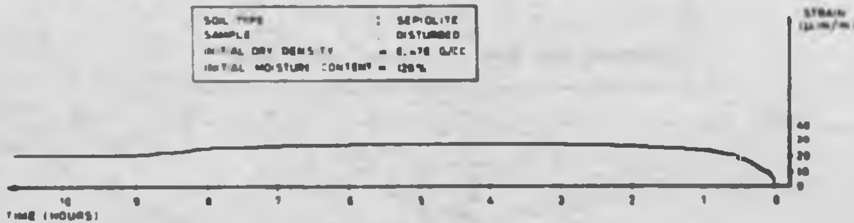

FIG. 5.111 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 8

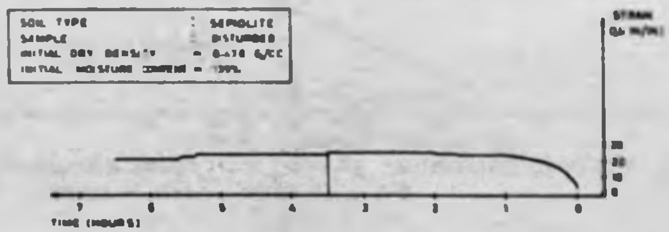

FIG. 5.112 STRAIN (μ IN/IN) V TIME (HOURS)
DIRECT METHOD TEST SERIES B-4 TEST No. 9

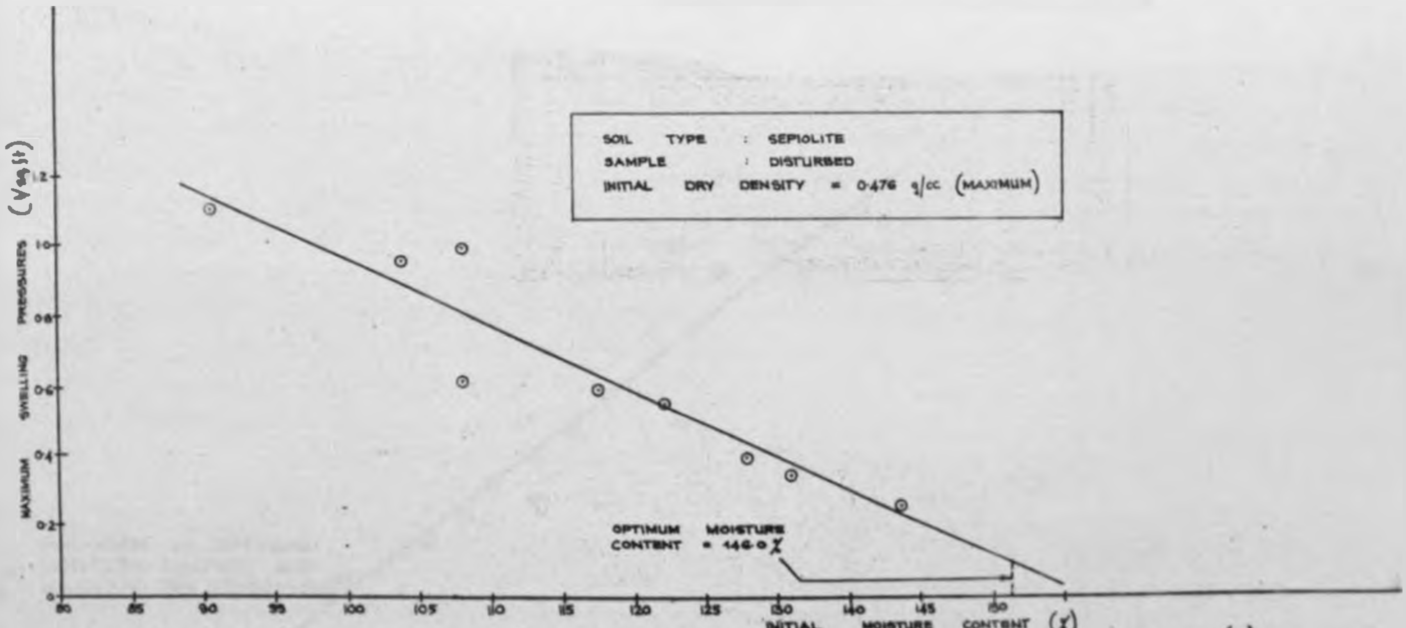

FIG. 5.113. MAXIMUM SWELLING PRESSURES (t/sqft) v. INITIAL MOISTURE CONTENT (%)
DIRECT METHOD TEST SERIES II-4 TEST No 1-9

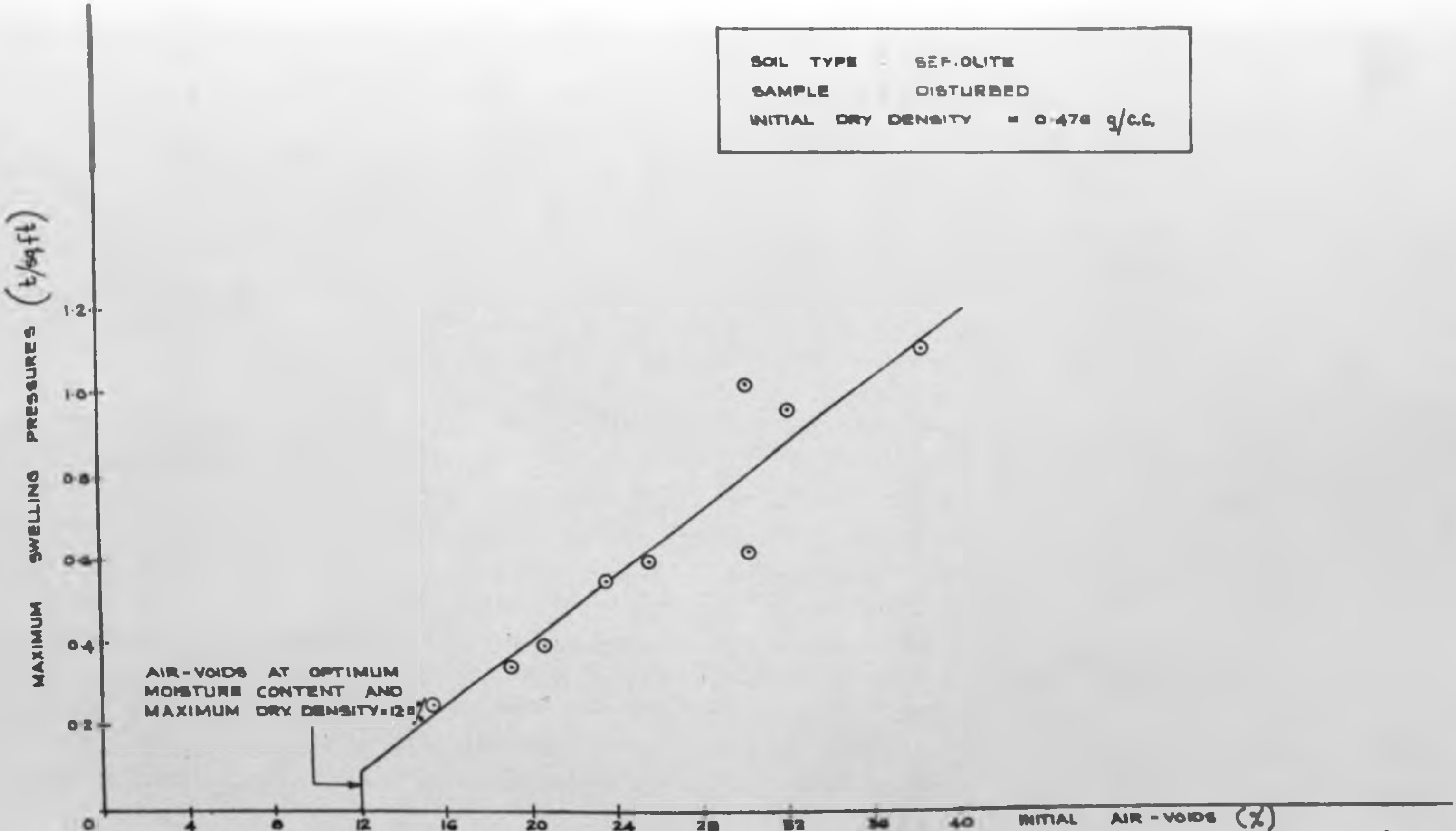

FIG. 5.14. MAXIMUM SWELLING PRESSURES (t/sq ft) v. INITIAL AIR - VOIDS (%)
DIRECT METHOD TEST SERIES B - 4 TEST No 1 TO 9

CHAPTER 6

DISCUSSION

6.1 Basic Tests:

The three types of soils tested could be classified, in accordance with the proposed classification procedure, as highly expansive, Sepiolite seemed the most expansive, followed by Black Cotton soil and Green Clay. The total volume change of these soils from the shrinkage limit to the liquid limit, as obtained from the shrinkage curve for remolded samples, is as follows:

	Sepiolite Clay	Green clay	Black Cotton Soil
Total Volume Change from shrinkage limit to liquid limit (%)	220	112	125
Figure No.	5.22	5.23	5.24

All three soils contain high percentage of clay fraction (Figs. 5.18, 5.19 and 5.20). Table 5.1 shows the very high value of the plasticity index for Sepiolite and Green Clay, indicating the high water adsorption capacity of these soils. It is interesting to note that the maximum dry density for the standard AASHO test for Sepiolite is only 0.476 g/cc, which is less than the density of water. Only the mineralogical composition of Black Cotton soil was examined and it showed a very high montmorillonite content (Table 5.3).

The value of the shrinkage limit determined by the mercury displacement method was higher than that determined from the shrinkage curve, as shown below. This may largely be due to the fact that in the former method some air is often trapped at the top of the glass disc and it is compressed.

Shrinkage limit (%)

	Black Cotton Soil	Green Clay	Sepiolite Clay
(1) Mercury Displacement Method	16.1	35.0	97.0
(2) From shrinkage curve for remolded sample.	13.5	33.0	79.8

It is seen that the shrinkage limit for Sepiolite is very high, indicating that the soil will begin volume change at high moisture content.

In the case of Black Cotton soil, the shrinkage limit, as obtained from the shrinkage curve of an undisturbed sample (Fig. 5.21) is higher than that of disturbed sample (Fig. 5.25). In case of the values shown in Fig. 5.22, there seems to be some experimental error. The result obtained is similar to that obtained by Wooltorton (40), who found that the shrinkage limit for an undisturbed sample of soil from a temperate area appears to be somewhat higher than that of a remolded sample and this is apparently because of the greater resistance to compression afforded by the natural soil structure of the undisturbed sample. The difference between the shrinkage limits of an undisturbed sample and a disturbed sample is, in a way, a measure of the structure of the undisturbed sample. It is also seen from Fig. 5.25, that the value of the shrinkage limit obtained at various densities seems to be fairly constant. The slope of the line in Fig. 5.21 is fairly constant at different densities and this is also true for the three undisturbed samples tested. The value of the maximum apparent dry density for the dry soil obtained from the value where the shrinkage curve cuts the volume (c.c.)/100 gm. dry soil axis varies for the undisturbed samples and for the disturbed samples. The maximum apparent dry density is an indication of the degree of compaction to which the sample has been subjected.

The above results have been obtained from limited data and they show only the basic trend. It has been found by Wooltorton (40) that spherical-shaped grains of equal size or non-uniform gradation will be expected to give lower apparent densities and higher shrinkage limits, than will well-graded particles or plate-shaped micelles. Also the shrinkage limit values have particular significance. It appears to be a function of the clay mineral or minerals present. In view of the physical concept, low values will be given by plate-shaped particles or those whose gradation leads to high densities and this has been found to be true for the soils tested.

6.2. Indirect Method. Test Series A-1: Tests on disturbed samples of Black Cotton Soil.

The swelling of Black Cotton soil under various conditions of initial dry density, initial moisture content and applied load was studied. At a particular initial dry density and initial moisture content, the swelling of sample was studied under various applied loads upto the maximum load under which the sample showed no increase in height. The results are shown in Figs. 5.31 through Fig. 5.56. The graphs of percentage increase in height $\frac{\Delta H}{H}$ v. time show that as the load applied on the sample is increased, $\frac{\Delta H}{H}$ decreases, until under a particular load the sample does not swell, but continues to consolidate. This shows that the pressure applied is greater than the internal pressure developed in a soil under the particular conditions. The amount of load on an expansive soil controls the amount of volume change that will take place under the particular conditions of moisture and density. If sufficient load is applied to balance the internal forces developed in a soil upon wetting, volume change could be avoided. Lesser loads than that required for zero volume change will allow some expansion to occur until the internal and external forces are in balance, with maximum expansion occuring under zero loading. The results have shown that the magnitude of swelling under the particular conditions of initial dry density and initial moisture content is inversely proportional to the restraining or confining pressure. The Figs. 5.31 to 5.56 also show that the rate of increase of $\frac{\Delta H}{H}$ decreases with time, although the rate of increase is rapid in the initial stages. For a particular initial dry density and initial moisture content, the time taken to reach maximum vertical expansion or equilibrium conditions decreases as the applied load is increased. The decrease in time is significant under heavier applied loads in which case the expansion is considerably reduced. It is also seen that for a particular initial moisture content and under particular applied load, the time to reach equilibrium conditions increases as the initial dry density is increased. For a particular initial dry density, and under particular applied load, the time to reach equilibrium decreases as the initial moisture content is increased.

From the graphs of percentage increase in height v. time, the total percentage vertical expansion after initial consolidation i.e. $\frac{\Delta H_T}{H}$ and the percentage vertical expansion above the initial height i.e. $\frac{\Delta Hi}{H}$ were deduced for each test. It is seen that when a particular load is applied on a sample, it first consolidates until there is enough build up of internal swelling pressure as the soil takes up moisture, to expand against the load.

The graphs of balancing swelling pressure v. total percentage vertical expansion after initial consolidation and balancing swelling pressure v. percentage vertical expansion above the initial height were deduced from

Fig. 5.31 through 5.34 for the value of initial moisture content equal to 23.0%. The graphs show distinctly for both cases:- (1) the inverse relationship between applied load and vertical expansion. (2) the effect of initial dry density on expansion - the greater the initial dry density, the greater is the vertical expansion under identical conditions. It is seen that there is significant increase in swelling pressure under particular conditions of initial moisture content and initial dry density, if the vertical expansion allowed is very small. Figs. 5.37 and 5.38, deduced from Figs. 5.35 and 5.36, show the relationship between balancing swelling pressures, initial dry density and vertical expansion for the initial moisture content of 23.0%. The curves could be used to predict the swelling pressures developed at a particular initial density and allowing certain vertical expansion, if the initial moisture content is at 23.0%. Figs. 5.43, 5.44, 5.45 and 5.46 confirm the above results for the value of initial moisture content equal to 27.3%; and Figs. 5.50, 5.51, 5.52 and 5.53 for initial moisture content of 29.5%, and Figs. 5.57, 5.58, 5.59 and 5.60 for initial moisture contents of 32.5%.

Interpolating and extrapolating the curves shown in Figs. 5.35, 5.36, 5.43, 5.44, 5.50, 5.51, 5.61, and 5.62, the maximum balancing swelling pressures for the two conditions were obtained i.e. balancing swelling pressures when $\frac{\Delta H_T}{H} = 0$ and when $\frac{\Delta Hi}{H} = 0$ and the results are summarised in Table 5.5. The swelling pressure obtained when $\frac{\Delta Hi}{H}$ is zero represents the pressure necessary to prevent swelling of the sample as it takes up water. Graphs of maximum swelling pressure v. initial moisture content at various values of initial dry density were drawn for the two conditions (Figs. 5.61 and 5.62). The graphs show clearly the effect of initial dry density and initial moisture content on swelling pressures. It is seen that the greater the initial dry density the greater is the swelling pressure developed. More clay particles are packed in a unit volume in a dense soil than in a loose soil. Therefore, when the soil is wetted greater movement will occur in a dense soil than in a loose soil. The inverse relationship between the swelling pressure and initial moisture content is clearly brought out in the Figs. 5.61 and 5.62. The form of curves in the Figs. 5.61 and 5.62 suggest that the relationship between maximum swelling pressure and initial moisture content may be expressed by the equations of the form:

$SPt = -2.3\ m+81....(10)$, for initial dry density = 1.5 g/cc

$SPt = -1.1\ m+40....(11)$, for initial dry density = 1.4 g/cc

$SPt = -0.42\ m+16.2.(12)$, for initial dry density = 1.33 g/cc

$SPt = -015\ m+5.3...(13)$, for initial dry density = 1.2 g/cc

$SPi = -0.54\ m+23...(14)$, for initial dry density = 1.5 g/cc

$SPi = -0.24\ m+11.1.(15)$, for initial dry density = 1.4 g/cc

$SPi = -0.12\ m+6.3..(16)$, for initial dry density = 1.3 g/cc

$SPi = -0.065\ m+2.6.(17)$, for initial dry density = 1.2 g/cc

In the above equations,

SPt = Maximum balancing swelling pressure in tons per sq.ft. when $\frac{\Delta Ht}{H} = 0$

SPi = Maximum balancing swelling pressure in tons per sq.ft. when $\frac{\Delta Hi}{H} = 0$

m = initial moisture content (%) and should be equal to or greater than the shrinkage limit.

The above equations are based on values of the moisture content between the range 23% to 32.5%. It is assumed that the maximum value of the balancing swelling pressure is at the initial moisture content equal to, or less than, the shrinkage limit. Below the value of the initial moisture content less than the shrinkage limit, there is no change in volume with increase in moisture content until the shrinkage limit is reached.

Wooltorton (40) has suggested that normally the swelling pressures would not be expected to give a strainght-line relationship when plotted against the moisture contents, but with the range of the moisture contents usually concerned, representing approximately the osmotic-range, it may perhaps be considered to decrease uniformly with increasing moisture content, becoming zero when the volume ceases to increase with moisture content.

The results obtained substantiate basically those obtained by Holtz and Gibbs (1). The Figs. 5.61 and 5.62 show that combination of placement densities which are lower than those obtained by standard compaction and high initial placement moisture content are required to ensure low amounts of expansion.

6.3. Indirect Method, Test Series A-2: Tests on undisturbed samples of Black Cotton Soil.

These tests were carried out on undisturbed samples. Owing to the difficulty of getting the undisturbed samples, tests carried out in this series were very few and these were divided into groups as shown in Table 5.6. In each group the values of initial dry density and initial moisture content were taken to be the average of all the tests in the particular group. The graphs of the percentage increase in height $\frac{(\Delta H)}{H}$ v. time (Figs. 5.66 and 5.67) are similar to those obtained for Test Series A-1. The graphs of balancing swelling pressure v. total percentage increase in height after initial consolidation, and balancing swelling pressure v. percentage increase in height above the initial height were obtained as before from the results (Figs. 5.68 and 5.69). The results confirm the influence of initial dry density and initial moisture content on swelling pressures. Table 5.7 was derived from Figs. 5.68 and 5.69. The values of balancing swelling pressure given for the undisturbed samples are very approximate. Hardly any definite conclusions could be drawn from the limited data. Curves (3) in Figs. 5.68 and 5.69 seem to be the most probable curves and these indicate that the undisturbed samples swelled more than the remolded samples under similar conditions (See Table 5.7).

Wooltorton (40) and Means (35) found that the undisturbed samples swelled more than the disturbed samples under similar conditions, but results obtained by Parcher and Liu (8) are contrary to this. Holtz and Gibbs (1) found that the remolded clays behaved similar to the undisturbed clays. More data is required to study the above aspect.

In view of the physical-chemical effects, one might anticipate that when the cementing effect was destroyed by remolding and recompaction, the clay micelles would be freer to absorb greater moisture and exert a greater swelling pressure than in the undisturbed state. Wooltorton (40) states that the reason for this is due to the structural differences. Remolded soils tend to be more permeable and water attractive than compacted soil. Undisturbed samples, therefore, tend to absorb less moisture than do remolded samples. The same, however, does not apply to the accompanying volume changes which tend to be greater for the undisturbed than for the remolded samples.

6.4. Direct Method; for the measurement of Swelling Pressure developed under condition of lateral and vertical confinement.

In the tests carried out by the Direct Method the maximum swelling pressures developed in a soil at various values of initial dry density and initial moisture content under complete confinement. According to Lambe (18), one indication of the nature of the electrical forces in a clay can be obtained by measuring the pressure required to prevent the swelling of clay when put into contact with water. The swell pressure seems to be a useful means of characterizing the force component of the structure. This aspect of the problem needs further study.

The swelling pressures developed in a soil and measured by the apparatus (Fig. 5.75) under conditions of complete vertical and lateral confinement have been classified as the maximum swelling pressures under particular conditions. Strictly speaking, this is not absolutely true. Very slight movement of the sample takes place owing to the deflection of the top steel plate and the deformation of the load cell. It was found that the maximum movement of the soil sample when it developed the maximum pressure was less than 0.001 in. The pressure dissipated due to this slight movement and due to the slight friction between the soil and the ring is not known (friction was reduced to the minimum by oiling the circumference of the ring). The apparatus gave consistent and reproducible results both during calibration, and testing of soil in the swelling studies.

The load cell used for these tests had the following specifications:

Accuracy: total accuracy 0.5% of full load, ambient temperature 5^{o} - 25^{o}C

Hysteresis: $\pm$0.05% of full nominal load.

Error due to side loading: at a side loading 10^{o} off axis $\pm$1.5% of full load.

6.5. Direct Method. Test Series B-1: Tests on disturbed samples of Black Cotton Soil.

The Figs. 5.78, 5.79, 5.80, 5.81 and 5.82 show the build up of pressure with time. The general shape of the curves in the Figs. is similar to those of percentage increase in height v. time obtained in Test Series A-1. The time required to develop the maximum pressure or to reach equilibrium is less in this series than in Test Series A-1. It seems from the Figs. that the time required to develop the maximum pressure is independent of both the initial dry density and initial moisture content, and is approximately 20 hours.

Fig. 5.83 shows the graph of maximum swelling pressure v. initial moisture content and Fig. 5.84 shows the graph of maximum swelling pressure v. initial air-voids, as deduced from the above figures. These results substantiate those obtained in Test Series A-1 as to the influence of initial moisture content (or initial air-voids) and initial dry density on swelling pressures developed in the soil. Comparing the results obtained in Test Series A-1 (Fig. 5.62) with those obtained in this series (Fig. 5.83), it is seen that for a particular initial dry density and initial moisture content and under conditions of no expansion, the swelling pressures obtained showed good agreement. The difference arising between them may be partly attributed to the variation in the structure of the soil under the two different conditions viz; (i) under complete lateral and vertical confinement. (ii) when the sample is allowed to expand vertically as it absorbs moisture. Parcher and Liu (8) pointed out that significant structural alterations may occur during swelling, if the magnitude of swelling is large. The variation of results may also be partly due to the fact that: (i) in Test Series A-1 the control of the initial moisture content and initial dry density was not very accurate. (ii) Fig.5.6 was obtained from the few values of the swelling pressures at a particular initial dry density and these also had to be indirectly extrapolated from other data.

The form of curves in Fig. 5.83 suggest that the relationship between swelling pressures and initial moisture content may be expressed by the equations of the form:

$SP = -0.63\ m+26.5$...(18), for initial dry density = 1.5g/cc

$SP = -0.29\ m+12.8$...(19), for initial dry density = 1.4g/cc

$SP = -0.18\ m+ 7.8$...(20), for initial dry density = 1.33g/cc

$SP = -0.084\ m+2.8$...(21), for initial dry density = 1.2 g/cc

In the above equations:

SP = Maximum swelling pressure in tons per sq.ft. under conditions of complete lateral and vertical confinement.

m = Initial moisture content (%)

The minimum value of the initial moisture content for which these equations are ture is, as mentioned in Test Series A-1, about 17%, which is the moisture content close to the shrinkage limit.

Similar equations as above could also be obtained for maximum swelling pressures and initial air-voids (Fig. 5.84).

6.6. Direct Method. Test Series B-2: Tests on disturbed samples of Black Cotton Soil to study the influence of the height of sample on the swelling pressure.

These tests were carried out to study the influence of the height of specimen on the swelling pressures developed in a soil. Fig. 5.87 shows that initially the swelling pressures increase with height, but the maximum swelling pressures developed are constant for a particular minimum height of the specimen. The energy possessed by clay particles is constant and the height of the specimen should not influence the maximum swelling pressures developed. However, boundary conditions and other factors which come into being during the experiment, if the height of the specimen is small, give rise to secondary effects.

6.7. Direct Method. Test Series B-3: Tests on disturbed samples of Green Clay.

These tests were carried out on Green clay and the results confirm the patterns already established earlier for Test Series B-1 (Figs. 5.102 and 5.103). The following equation could be deduced from the curve in

Fig. 5.102,

$$SP = -0.13m + 8.2 \ldots\ldots\ldots (22), \text{ for initial dry density} = 1.02 \text{ g/cc.}$$

In the equation,

SP = Maximum swelling pressure in tons per sq.ft. of complete lateral and vertical confinement.

m = Initial moisture content (%), the minimum value for which the equation is true, is 34%. This is the moisture content equal to the shrinkage limit.

Similar equation, as above, could be obtained fcr maximum swelling pressures and initial air-voids (Fig. 5.102)

6.8. Direct Method. Test Series B-4: Tests on disturbed samples of Sepiolite Clay.

These tests were carried out on Sepiolite Clay and the results again confirm the patterns already established (Figs. 5.113 and 5.114). The following equation could be deduced from the curve in Fig. 5.113,

$$SP = -0.019\,m + 2.9 \ldots (23), \text{ for initial dry density} = 0.476 \text{ g/cc.}$$

In the equation,

SP = Maximum swelling pressure in tons per sq. ft., under conditions of complete lateral and vertical confinement.

m = Initial moisture content (%), the minimum value for which the equation is true, is 80%. This is the value of moisture content equal to the shrinkage limit.

Similar equation, as above, could be obtained for maximum swelling pressures and initial air-voids (Fig. 5.114).

6.9. General.

The main results obtained for the soils investigated, are summarised in Table 6.1 and Table 6.2.

Table 6.1

Summary of the Results of Basic Tests.

TEST	Black Cotton Soil	Green Clay	Sepiolite Clay
Plasticity index(%)	55	114	167
Shrinkage index(%)	78	218	278
Clay fraction(%) (-.002mm)	75	67	61
Free Swell(%)	387	530	1045
Activity	0.73	1.70	2.74
Volume change from shrinkage limit to liquid limit (%)	125	112	220

Table 6.2

Comparison of some of the Results of the measurement of swelling pressures developed by Black Cotton soil by Indirect Method and Direct Method

Test	Placement conditions		Swelling pressure (tons/sq ft.)
	Initial Moisture Content (%)	Initial Dry Density (g/cc.)	
Indirect Method	27.3	1.5	[illegible]
	27.3	1.6	6.9
	32.5*	1.33**	[illegible]
	32.5	1.2	[illegible]
Direct Method	27.3	1.5	[illegible]
	27.3	1.6	[illegible]
	32.5*	1.33**	[illegible]
	32.5	1.2	[illegible]

* Optimum moisture content (standard AASHO)

** Maximum dry density (standard AASHO)

Table 6.2

Comparison of some of the Results of the measurement of swelling pressures developed by Black Cotton soil by Indirect Method and Direct Method

Test	Placement conditions		Swelling pressure (tons/sq.ft.)
	Initial Moisture Content (%)	Initial Dry Density (g/cc.)	
Indirect Method	27.3	1.5	8.4
	27.3	1.4	4.7
	32.5*	1.33**	2.4
	32.5	1.2	0.4
Direct Method	27.3	1.5	9.1
	27.3	1.4	5.0
	32.5*	1.33**	2.0
	32.5	1.2	0.1

* Optimum moisture content (standard AASHO)

** Maximum dry density (standard AASHO)

Table 6.3

Maximum Swelling Pressures developed in Black Cotton Soil as measured by the Direct Method.

Initial dry density (g/cc)	Maximum swelling Pressure developed under conditions of complete confinement (t/sq.ft.)
1.2	1.7
1.33	5.8
1.4	9.0
1.5	14.0

It was observed during the experiment that the swelling pressure falls a little, if the sample is allowed to take too much water at a time and/or if the experiment is carried out for longer period after the sample has developed the maximum pressure. This may be attributed to a deterioration of the structure called "slaking" by Wooltorton (40). Slaking implies moisture in excess of that required to fill voids and moisture sufficient to cause disruption of the initial structure.

It is believed that if a free water supply comes into contact with dry soil, a near-slaking condition may be reached very rapidly, virtually resulting in the destruction of cohesive bonds.

It was observed that the penetration of water through the Black Cotton soil was poor and was not uniform. This is due to the low permeability of the soil. It is suggested that with such soils experiments may be carried out by having a radial drainage through the sample as recommended by Shields and Rowe (48). This will also accelerate the rate of swell. The amount of swell that will occur in a given period for a given soil depends on the quantity of water that enters the soil. Thus the rate of swell is proportional to the coefficient of permeability and to the hydraulic gradient. For a given soil, these factors seem to be influenced by the soil structure or by the treatment during compaction.

Sepiolite Clay and Green clay seem to have higher permeability than Black Cotton Soil. The time taken to develop the maximum pressures or reach equilibrium conditions for the Black Cotton Soil is about 20 hours, for Green Clay about 10 hours, and for Sepiolite Clay about 4 hours.

Wooltorton (40) had found that when a naturally dense undisturbed sodium clay absorbs water from a free water surface, the colloidal surface energy available for adsorption is comparatively low. Osmotic swelling then comes into the picture and may be sufficient to block the capillary adsorption. This will have the effect of further reducing the amount of moisture absorbed together with the overall swelling. The lower the coefficient of permeability the less may be the water intake. Thus the intake will often be less for a sodium-saturated clay than for a calcium-saturated clay, although the affinity of the sodium-saturated clay unit for water may be considerably greater than that of calcium clay unit. This aspect of the problem needs detailed study.

The measurement of swelling pressures at a moisture content close to the shrinkage limit was difficult. It was found that at such low moisture contents, there was hardly any cohesion between the particles and it was rather difficult to compact the soil properly. Also, when a sample at such low moisture content is allowed to take water, slaking may in the initial stages cause disruption of the initial structure. The moisture content range close to the shrinkage limit may be considered as a region of instability.

Table 5.10 and 5.11 seem to indicate that the final degree of saturation after swelling has ceased is practically independent of the initial moisture content. More tests, however, are required to draw definite conclusions. Parcher and Liu (8) have also found that the degree of saturation after swelling has ceased is practically independent of the initial moisture content and initial degree of saturation.

From the limited data, it seems that there is not necessarily any direct relationship between the total swell of a soil under some standard conditions and the swelling pressures developed under conditions of complete confinement. The total volume change from the shrinkage limit to the liquid limit increases in the order of Green Clay, Black Cotton Soil and Sepiolite Clay. The maximum swelling pressures developed in the soils at initial moisture content and at initial maximum dry density increase in the order of Sepiolite, Green Clay and Black Cotton Soil (Table 6.2).

CHAPTER 7

CONCLUSIONS

The investigations carried out confirm some of the results obtained by other investigators. It is found that the initial dry density and the initial moisture content (or the initial air-voids) have a significant effect on the swelling pressures developed in a soil. In the light of all the investigations carried out, the following conclusions may be drawn:

(1) For any given system of compaction, the swelling pressure developed in a soil is inversely proportional to the initial moisture content (or directly proportional to the initial air-voids). This applies up to a critical moisture content when the swelling pressure becomes a minimum and may be zero. It is also found, that for the range of moisture contents concerned, representing approximately the osmotic range, a straight line relationship between swelling pressure and initial moisture content is obtained. The maximum swelling pressure is developed if the soil is at an initial moisture content close to the shrinkage limit.

(2) For any given initial moisture content, the swelling pressure increases with increase in initial dry density.

(3) It is seen that there is not necessarily any direct relationship between the swell of a soil under some standard conditions and the swelling pressures measured under conditions of no expansion. Further tests, however, are required to verify this aspect.

(4) The undisturbed samples swelled more than the remolded samples under similar conditions.

(5) The swelling pressures developed seem to be influenced by the height of the specimen only if the height of the specimen is small. If the height of the specimen is above a certain minimum value, the swelling pressures developed are independent of the height of the specimen.

(6) It is possible to control the amount of expansion in compacted clay soils by increasing the initial moisture content and/or decreasing the initial dry density. The favourable initial dry density and the initial moisture content conditions for expansion control can be ascertained from the tests.

(7) In the Indirect Method and the Direct Method an effort was made to evaluate the optimum swelling pressure. Under particular conditions of initial moisture content and initial dry density, this may be defined as the maximum pressure which is developed due to the increase in moisture

content when its volume change is prevented. In the Indirect Method, two types of balancing swelling pressure are considered. The balancing swelling pressure, when $\frac{\Delta H_i}{H}$ is zero, is considered to be the Optimum swelling pressure. The balancing swelling pressure when $\frac{\Delta H_T}{H}$ is zero, was evaluated for each condition of initial moisture content and initial dry density to distinguish the influence of the initial consolidation of the specimen, under heavy applied loads. The direct measurement of the swelling pressure developed in a soil, held under conditions of lateral and vertical confinement, was made possible by the Direct Method.

(8) The maximum swelling pressure developed in Black Cotton soil, as measured by the Direct Method was found to be in the region of 14 tons per sq.ft., at initial dry density of 1.5 g/c.c.

(9) The suggested system for the identification and classification of expansive soils, as shown in Fig. 2.9 and Table 2.3, based on the shrinkage index and the clay fraction, seems to give a better indication of the basic expansive characteristics of a soil.

CHAPTER 8

FUTURE WORK

It is suggested that tests should be carried out to measure the swelling pressures developed in undisturbed and disturbed soil samples at various values of initial dry density and initial moisture content, after allowing the sample to expand vertically to predetermined values. The apparatus designed for the Direct Method of the measurement of swelling pressure under controlled conditions of lateral and vertical confinement, may be modified for this purpose. The information received from the tests may be very useful in designing structures on expansive soils as the swelling pressures are considerably reduced, if the soils are allowed to expand a little.

An interesting aspect is the study of the ratio of horizontal to vertical pressure developed in a soil under complete confinement. Transducers could be used to measure separately the horizontal and vertical pressures developed.

Some research has been carried out at the Road Research Laboratory (49) to establish the soil-suction and moisture content relationship for various soils. It is suggested that the soil-suction/moisture content relationship be obtained for the soils at various values of initial dry density. Soil-suction may be considered to represent the total energy possessed by a soil to attract moisture. If the soil-suction v. moisture content relationship could be obtained at a particular density, the swelling pressures may be represented as the soil-suction between the initial and the final moisture contents. Assuming that the soil finally attains full saturation, then the swelling pressure is equal to the suction pressure at the particular moisture content. The results may be compared with the swelling pressures obtained by the Direct Method. From the suction/moisture content relationship, it is noticed that at the values of initial moisture contents below the shrinkage limit, the values of the suction pressures are very high. This aspect, however, is not evident from the swelling pressures measured by the apparatus. The maximum pressure found was at the value of the initial moisture content close to the shrinkage limit. The latter, however, seems to represent the swelling pressures normally encountered in the field.

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