LATERAL CONFINEMENT IN SUBGRADE UNDER VERTICAL LOADS AND ITS SIGNIFICANCE IN DESIGN

A Thesis
submitted in partial fulfilment
of the requirements for the Degree
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MASTER OF ENGINEERING
in
CIVIL ENGINEERING
(Highway Engineering)

By
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CERTIFICATE

Certified that the thesis entitled "LATERAL CONFINEMENT IN SUBGRADE UNDER VERTICAL LOADS AND ITS SIGNIFICANCE IN DESIGN" which is being submitted by Shri A.V. Joseph in partial fulfilment for the award of the Degree of Master of Engineering in Civil Engineering (HIGHWAY ENGINEERING) of the University of Roorkee is a record of student's own work carried out by him under our supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a period of over six months from December 21, 1966 to June 28,* 1967 at this University in order to prepare this thesis.

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CONTENTS

SYNOPSIS

The experimental study reported herein has been designed to define and evaluate the role of lateral confinement in the bearing capacity of the subgrade under vertical loads. The method of approach employed consists in studying the influence of lateral confinement on the modulus of elasticity E of three types of soils under conditions of modified AASHO compaction. This information is applied in the Kansas method of flexible pavement design to determine the effect of lateral confinement on design thicknesses of flexible pavement.

Undrained triaxial test has been used to determine the E of soils. Suitable values of lateral fluid pressures simulative of the lateral stresses developing in field are applied in the triaxial cell. These values of lateral pressures were determined by an experimental procedure suggested in the present study.

Kansas design method makes use of 20 psi lateral pressure in the triaxial test irrespective of soil types. The results of the present study indicate that realistic value of lateral pressure to be used in triaxial test varies over a relatively wide range depending on soil types. Consequently the value of E and the resultant design thickness of pavement are considerably affected.

It has been inferred also that the use of 20 psi lateral pressure is realistic only in case of soils having California Bearing Ratio around 2 percent or Group Index around 7.3. Other soils require lateral pressures varying directly as California Bearing Ratio and inversely as G.I. Based on these results a method of correcting the design thickness obtained by the Kansas design method is developed.

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CHAPTER - I

I N T R O D U C T I O N

CHAPTER - I.

INTRODUCTION

1.1 General

1.1.1 There are a great many methods of flexible pavement design, differing considerably in their methods of analysis. The design methods currently in practice are mostly empirical and semi-emprical. The basic draw back of empirical methods is that the safety factor actually being employed cannot be estimated (1)* .

1.1.2 Research work is in progress in many countries for modifying the present design methods to make them more realistic. Studies are also being made to evolve a rational design method. These efforts are justified for two major reasons: Firstly , success in this direction will greatly contribute towards standardization of uniform design practices under different field conditions. Secondly, there is immense academic interest centered around this topic of basic research. A design method is qualified as being rational when it is susceptible of complete mathematical analysis in terms of stress-displacement phenomena in the pavement structure under assumed design conditions. In France about half a dozen mathematicians have been working on this problem for the last many years (2) .

1.1.3 Burmister's analysis and design method for flexible pavements considered as layered systems have been regarded

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* Numbers in paranthesis refer to corresponding numbers in Bibliography listed in the end.

as the most rational approach to the problem made so far(1). Burmister's theory is based on the assumptions of a critical surface deflection and fully elastic behaviour of materials in each layer of the pavement composite (1,3). Consequently the most important property of the paving materials and subgrade as strength criterion is the modulus of elasticity E.

1.2 Scope of Investigation (Introduction)

1.2.1 The investigation reported herein is concerned with a realistic determination of the modulus of elasticity of the subgrade soil. This problem is analysed with special emphasis on the role of lateral confinement in subgrade under vertical loads. The present practice is to determine the value of modulus of elasticity of soil from the stress-strain plot developed from a triaxial test (undrained test) data. In the triaxial test a lateral fluid fpressure of 20 psi is applied irrespective of soil type to simulate the condition of lateral stress developing in subgrade under vertical loads (4). Whether the use of 20 psi lateral pressure for all soils is realistic is open to question. This problem is investigated in the present study by an experimental programme consisting of California Bearing Ratio Test, a Penetration Test with 1.5 inch plunger diameter, undrained Triaxial Test and a Tube Test. These tests are conducted on soil specimens under conditions of modified AASHO compaction and after 4 days soaking. Three soil types are studied.

1.2.2 The concept of lateral confinement in subgrade is explained in the following chapter. Chapter III outlines the characteristic features of the three types of soils used in the study and the procedures adopted for the various tests. Test results and their analysis are presented in Chapter V followed by the conclusions from the study in the final chapter.

CHAPTER - II

THE CONCEPT OF LATERAL CONFINEMENT IN SUBGRADE

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THE CONCEPT OF LATERAL CONFINEMENT IN SUBGRADE.

2.1 The strength measure of subgrade material is the measure of its resistance to deformation under wheel loads. The pattern of deformation in the subgrade needs to be defined for a complete description of the strength characteristics of the material.

2.2 Deformation Pattern In Subgrade.

The subgrade in service has only normal distributed load to support. Under a circular loaded area, the mass of soil that is stressed may undergo two forms of deformation , namely :

i) Downward displacement (Settlement) in vertical direction.

ii) Lateral displacement in radial direction.

Figure 1 illustrates this deformation pattern in subgrade under vertical loads. The soil mass under stress builds up resistance to both these forms of deformation.

2.3 Lateral Confinement.

2.3.1 Lateral (horizontal) stresses develop in a loaded soil mass to resist lateral displacement of soil. The term "Lateral Confinement" refers to the soil conditions making up the capacity of the soil to build up lateral stresses in subgrade to resist lateral displacement of the

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"Consider a normal subgrade upon which is laid a plate of 9 inches in diameter. If a load of 5000 pounds is applied to this plate resting upon the subgrade, some settlement is likely to occur. If the load is released, the material removed from the edge of the plate out for 6 inches and to a depth of at least 18 inches, and the 5000 pounds load is reapplied, the cylinder will settle to a much greater degree. The load has not been increased, but the horizontal resistance has been removed allowing the sides of the soil cylinder to move outward when unsupported. When the material surrounded the cylinder such horizontal movement was resisted. The lateral pressure applied to a specimen in a triaxial compression test is similar in effect to this horizontal resistance" (5).

2.3.2 To further illustrate the effect of lateral confinement in the stress-deformation pattern in subgrade, the following example may be employed :

A column of soil mass having density and moisture conditions as those specified for field use and filled in a cylindrical metal tube of unyielding walls is considered. The tube is open at both ends and is kept standing on one end on a firm platform. It is loaded axially (vertically) under a plunger area equal to the cross-sectional area of the tube. The soil deforms vertically only. No deformation is possible in the lateral direction. In other words, lateral stresses developed are such that the lateral deformation is fully resisted. It may be said that the lateral confinement is 100 percent or the maximum possible. The ultimate load carried by the soil may increase indefinitely. The phenomenon is represented in Fig. 2, Case I.

If the metal tube in the above case were replaced by a thin rubber sleeve of negligible strength, the lateral confinement present is practically zero. There is little resistance offered to lateral displacement. Under similar conditions of loading as in the previous case, the ultimate load carried by the soil will be a relatively small finite value. The phenomenon is represented in Fig. 2, Case II.

A normal subgrade in field has a degree of lateral confinement (or lateral support) in between the two extreme cases illustrated above. The phenomenon is represented in Fig. 2, Case III.

2.3.3 The experimental investigation reported herein was designed to evaluate the degree of lateral confinement existing in subgrade under conditions of modified AASHO compaction and after four days soaking. The degree of lateral confinement is estimated in relation to the two extreme cases of 100 percent and zero percent values of lateral confinement as discussed above.

2.4 Importance In Pavement Design.

2.4.1 Although the effect of lateral confinement in subgrade has been recognized, there is no standardized method for its evaluation so far. The Kansas method of flexible pavement design using triaxial test takes into account the effect of lateral confinement. The Kansas State Highway Commission began investigation on the use of triaxial testing for design of flexible pavements in 1941 (5,6). After five years of development, Kansas adopted a procedure for triaxial testing of subgrade soils, base course materials and bituminous mixtures. By the Kansas method the required thickness of pavement is based on the stress-strain relationship of each component part of the pavement as determined by the triaxial test. One confining or lateral pressure of 20 psi is used. It is considered by the State Highway Commission that 20 psi is comparable to the lateral support or horizontal resistance which is normally provided by the adjacent similar material under field conditions (4).

2.4.2 It has been recommended that the "Quick Undrained Test" should govern the design of pavements. These are tests in which the vertical load is applied with no drainage of the sample permitted during the test. It is suggested that the loads applied to a pavement in service being trancases of 100 percent and zero percent values of lateral confinement as discussed above.

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-sient, it is doubtful whether any drainage takes place during the loading cycle. The recommended rate of loading is 0.05 inch per minute (7,8).

2.4.3 It might appear that the use of 20 psi lateral pressure for all types of subgrade soil may not represent actual field conditions. The realistic value of lateral pressure simulative of lateral support in field might vary with soil types. This is investigated in the present project using three types of soils. The description of soils and the methods of tests employed are presented in the next chapter.

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CHAPTER - III

MATERIALS AND TESTING

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MATERIALS AND TESTING

3.1 Materials.

Three types of soils were selected for investigation. The criterion for the choice of the three types of soils was that, as could be judged from their texural and grain size features, they were expected to exhibit strength properties of considerably varying magnitudes, and provide a sufficient range for comparitive study of results. The relevant features of the three types of soils are outlined below :-

3.1.1 Soil I (C.B.R.I. Soil)

This soil was collected from some part of the Central Building Research Institute residential colony, Roorkee. The soil has a light shade. The grain size distribution as determined by sieve and hydrometer analysis is presented in Fig. 3. Other relevant properties of the soil are presented in Table 1.

This soil is classified as "Sandy Loam" as per U.S. Bureau of texural soil Classification System. According to the U.S. Public Roads Administrative System of soil classification, it falls under A-4 group. It has a Group Index of 1.6. Appendix I presents the calculations for Group Index.

3.1.2 <u>SOIL II (BLUE SOIL)</u>

This soil was procured from the bed of river Dhanauri, 8 miles away from Roorkee. It has a bluish shade. The soil is predominantly silty with 17 percent clay content. The grain size distribution as determined by Sieve and Hydrometer Analysis is presented in Fig. 3. Other relevant properties of the soil are presented in Table I.

This soil is classified as "Silty Loam" as per U.S. Bureau Textural Soil Classification System. According to the U.S. Public Roads Administration System of soil Classification, it falls under A - 7 Group. It has a group index of 11.8.

Appendix I presents the calculations for Group Index.

3.1.3 Soil III (Blend Soil)

This soil was a mixture of soil I and soil II. Soil fractions passing ASTM Sieve No.4 from the two soils in air dried condition were thoroughly mixed together in the ratio 1 : 1 by weight. The grain size distribution as determined by sieve and hydrometer analysis is presented in Fig. 3. Other relevant properties of the soil are presented in Table 1.

This soil is classified as "Silty Loam" as per U.S. Bureau Textural Soil Classification System. According to the U.S. Public Roads Administration System of soil classification, it falls under A-6 group. Its group index is 8.4. Appendix I presents the calculations for group index.

3.1.4 Thus soil I, Soil II and soil III have Group Index values as 1.6, 11.0 and 8.4 respectively. This indicates that they represent a sufficient range of strength variations to facilitate a comparitive study of test results in this investigation.

3.2 Testing

3.2.1 The following tests were performed :

1. California Bearing Ratio Test.
2. Penetration Test with 1.5 inch diameter plunger.
3. Quick Undrained Triaxial Compression Test.
4. Tube Test.

placed inside the C B R mould and the first of the five portions of soils prepared as above is poured in. After levelling its top surface, a 1 in. thick loosely fitting metal disc is placed over it. Static load was applied on to it from a 10 ton compression testing machine at a constant rate of 1 inch per minute. This rate was considered to be neither too rapid as to impair uniformity of compaction nor too slow as to cause undesirable delay. The soil layer was compacted to a thickness of 1 inch. After releasing the load and removing the metal disc, the surface of the compacted soil was scratched with a knife to secure firm bonding with the succeeding layer. The second portion of the soil was now poured into the mould and compaction was performed as before to obtain a total thickness of 2 in. The remaining portions of the soil were also similarly compacted and a 5 in. thick finished sample was prepared. Photo 1 shows the compaction process.

While weighing out the total weight of dry soil required to prepare one sample, 15 grams of excess soil is taken. This is intended to allow for the possible loss of soil during the preparation of sample. By this process, it was possible to obtain uniform density for all samples of a soil with a possible maximum error of $\pm$ 10 gms in the total bulk weight of sample in the mould which not substantial.

The sample with the mould is then kept immersed in water under a surcharge load of 10 lbs. After four days of soaking the sample was taken out and after allowing 15 minutes drainage, it was used for the various tests as described below. It was assumed that the sample after four days soaking represents the worst condition in the field.

3.2.2.2 Peculiar to Each Test :

3.2.2.2.1 1) California Bearing Ratio Test

2) Penetration Test with 1.5 inch diameter plunger.

The sample prepared as described before was tested directly for these two tests.

3.2.2.2.2 3) Triaxial Compression Test.

A steel core cutter of 1.5 inch inner diameter was used to extract a specimen of 3 in. length and 1.5 inch diameter from the sample prepared as described before. The inside of the core cutting was lightly oiled. Its cutting edge was driven down into the sample in the C.B.R. mould under light hammering, taking care to hold the core-cutter vertical always. After the core cutter was driven to a sufficient depth so as to get a sample longer than 3 inches, it was carefully rotated and slowly pulled out. The sample was extracted from the core cutter using the standard sample extractor device and was trimmed to exactly 3 inches length for the triaxial compression test.

3.2.2.2.2 b) Tube Test.

The equipment for this test was specially designed for the purpose of this investigation. It consists of a metal tube open at both ends and having unyielding walls. It is provided with a slot around it on its exterior side and near one end. This slot is used to firmly connect the tube to the standard sample extractor device. The tube has an inner diameter of exactly 1.5 inch.

The sample from the core-cutter obtained as explained before was slowly extracted into the above metal tube after firmly connecting its slotted end to the sample extractor device. When the sample emerged out at the other end of the tube, the soil contained in the tube was finished flush with its ends and is used for testing as described later.

3.2.3 Test Procedures.

The procedures adopted for the various tests are briefly outlined below :

3.2.3.1 1) California Bearing Ratio Test.

The California Bearing Ratio Test was run in the standard manner with a surcharge of 10 lbs and at a rate of strain of 0.05 inches per minute. A 5 ton compression testing machine was used. Load readings were recorded at every 0.025 inch deflection upto a total deflection of 0.50 inch.

The tests were run for each soil type. Figure 4 shows the schematic diagram of test procedure.

3.2.3.2 2) Penetration Test Using 1.5 Inch Diameter Plunger.

This test was also run in the same manner as the California Bearing Ratio Test, except that initial 10 lbs load for seating the plunger was not applied. When the plunger touched the soil surface, the corresponding load is recorded as zero. This procedure was adopted to simulate the procedure in triaxial test as it was intended to compare the penetration test resuts with triaxial test results.

Load readings were recorded at every 0.025 inch deflection upto a total deflection of 0.4 inch. Three tests were run for each soil type. Photograph No. 2 shows the test set up.

3.2.3.3 3) Triaxial Compression Test (Quick Undrained)

The sample 3 inch long and 1.5 inch diameter was placed on one of its face on a perspex disc about 3/4 in. thick on the central pedestal of the triaxial cell. A perspex cap was placed on top. A rubber membrane was placed inside a 3.5 inch. length of thin-walled brass tubing of about 1.75 inch internal diameter, and was turned back over the ends. Suction was applied to the space between the membrane and the brass tube (with mouth) which expanded the membrane to a convenient size to slip over the sample without touching it. The suction was released so that the membrane contracts on to the sample. The ends of the membrane was moistened and slipped off the membrane stretcher.

Two rubber O rings were then stretched over the end of the brass tube which was held over the sample again while the ring was rolled off with the fingers to grip the membrane against the lower porous disc and the loading cap, successively. A stainless steel ball was placed in the seating on the cap and the axial alignment was checked (9,10).

The upper part of the cell was carefully placed over the sample, the ram being lifted to the upper limit of its travel while this was being done. The three wing nuts were tightened evenly.

At this point all valves were closed except the one connected to the pressure gauge to let in water into the cell and the top valve for air escape. Water was admitted to the chamber to bring the water level up until the cap was just covered and then the top valve was closed air tight. The pressure in the chamber was raised to the desired value by means of a foot pump.

The loading platform of the testing machine was then raised to bring the ram close to the loading cap. With the ram about 1/4 inch above the sample the water drive was started at the rate of 0.05 inch per minute which is the recommended rate for quick undrained tests for flexible pavement design purposes. The dial gauge on the proving ring was set to zero. This automatically compensated for the upward thrust on the ram due to the fluid pressure in the cell and for any frictional drag. The motor drive was then stopped and the hand adjustment was used to bring the ram in contact with the top of the sample. Contact was indicated by the dial gauge on the proving ring (9).

The strain dial gauge was set to zero by adjusting the movable arm on the pillar, and the test was commenced using the motor drive. Load readings were recorded at every 0.025 inch deflection. The test is continued upto a total deflection of 0.4 inch.

Three tests were run for each soil type for each of the following values of lateral pressure in psi. 0, 5, 10, 15, 20, 30, 40, 50 and 60. The test set up is represented in Fig. 5 and photograph 3.

3.2.3.4 4) Tube Test.

The metal tube filled with soil sample as described before was placed on its face on a thick metal plate as shown in Fig. 6. A metal disc 1/2 inch thk was placed over the top just covering the soil core. The seating was carefully checked to ensure that the metal disc did not overlap the inside edges of the metal tube.

Axial load from a 5 ton compression testing machine was applied on the metal disc at the rate of 0.05 inch per minute. Load readings were noted at every 0.025 inch deflection till a total deflection of 0.3 inch was reached. Three such tests were run for each soil type. Photograph No. 4 shows the test set up.

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CHAPTER - IV

TEST RESULTS AND ANALYSIS

CHAPTER - IV

TEST RESULTS AND ANALYSIS

4.1 Test Results

The results of the California Bearing Ratio Test, Penetration Test with 1.5 inch diameter plunger, Triaxial Tests, and Tube Test conducted on the three soils are presented below.

4.1.1 1) California Bearing Ratio Test.

Three tests are run for each soil and the load-deformation plots are drawn for each test. Necessary correction is applied in cases where the initial portion of the cur ve showed downward concavity. The average of the corrected load values of the three tests are used to develop the final plot for the determination of the California Bearing Ratio of each soil. The data are presented in Tables 2,3 and 4, and corresponding plots in Figures 7, 8 and 9.

The C.B.R. values of the three soils are :-

C.B.R. of Soil I	(CBRI Soil)	= 3.64 percent
C.B.R. of Soil II	(Blue Soil)	= 1.103 percent
C.B.R. of Soil III	(Blended Soil)	= 1.82 percent.

4.1.2 2) Penetration Test Using a 1.5 inch Diameter Plunger.

As in the case of C.B.R. tests, load-deformation values for each of the three tests on every soil are plotted and correction applied wherever required. The average of the corrected load values of the three tests are used to develop the final load-deformation plot for each soil. The results are presented in Tables 5,6 and 7 and corresponding plots in Figures 10, 11 and 12.

4.1.3 3) Triaxial Test

Average of load values from three tests for each lateral pressure are taken for drawing the load-deformation plot. Tables 8,9 and 10 and corresponding plots in Figures 13, 14 and 15 show the results of tri-axial tests on the three soils investigated.

4.1.4 4) Tube Test .

Average of the load values from three tests for each soil are used for drawing the load-deformation plot. Tables 11, 12 and 13 and corresponding plots in Figures 16, 17, 18 show the results of tube tests on the three soils investigated.

4.2 Analysis of Results :

4.2.1 For the purpose of the analysis, following two basic assumptions are made. Those are :

i) As far as the effect of lateral confinement is concerned, the penetration test in a C.B.R. mould of inner diameter 6 inches with a plunger

1.5 inches in diameter may be considered to be identical to the same penetration test on sub-grade in field.

ii) There may be a triaxial test with a certain value of lateral stress whose load-deformation curve will be comparable to the load-deformation curve of the penetration test, for a given deflection level.

The first assumption is based on the fact that the pressure bulb developing under the 1.5 inch loaded area at the centre of the C.B.R. mould does not extend radially more than about $\frac{1.5}{2} \times 2.5 = 1.87$ inches from the centre (11). This means that the walls of the mould, being 3" away from the centre, can have little or no influence on the lateral confinement for the test conditions. In other words, the test is as good as one on field.

It has been noted before that the triaxial test with an all-round fluid pressure is simulative of the loaded soil mass in the field. It, therefore, follows that the load-deformation curves for the penetration test and the triaxial tests have a basis for comparison.

4.2.2 'Equivalent Lateral Stress'

As a means of defining the effect of lateral confinement, the term "Equivalent Lateral Stress" is introduced. As assumed above, if it is possible to identify a triaxial test with a certain definite value of lateral fluid pressure whose load-deformation curve is comparable to the load deformation curve of the penetration test for a given deflection level, that particular value of fluid pressure in triaxial test may be designated as "Equivalent Lateral Stress". In other words, whatever be the precise measure and distribution of the lateral stresses that develop within the loaded soil mass, the above value of lateral fluid pressure in the triaxial test bears an equivalence to it.

For the purpose of this analysis by comparison and interpolation of load-deformation curves, the range of deflection upto 0.2 inch only is considered. It may be noted that in flexible pavement design practice the usual deflection criteria are 0.1 inch and 0.2 inch.

4.2.3 Determination of the "Equivalent Lateral Stress" For the Three Soils (From Penetration Tests and Triaxial Tests).

By comparison and interpolation between the load-deformation curves of the penetration tests and tri-axial tests with various values of lateral pressures, the "Equivalent Lateral Stress" for each soil is derived as follows:

4.2.3.1 Equivalent Lateral Stress for Soil I (CBRI Soil).

The load-deformation curve for the penetration test shown in Figure 10 is interposed among the set of curves for the triaxial tests for various values of lateral fluid pressures shown in Figure 13. This is presented in Figure 19.

It is found from Figure 19 that there is no single triaxial test with a definite value of lateral fluid pressure whose load-deformation curve is identical to that of the penetration test. This is for the obvious reason that as in any of the triaxial test the lateral fluid pressure is held constant, the lateral stresses in the loaded soil mass in a penetration test goes on increasing as the vertical load increases. The objective of the study, therefore, is limited to the determination of the approximate value of Equivalent Lateral Stress for the range of deflections used in design practice, namely 0.1 to 0.2 inch. For this range, the load-deformation curve falls clearly between the load-deformation curves for lateral fluid pressures 20 psi and 30 psi in triaxial test. It is nearer to 20 psi at 0.1 inch deflection range and to 30 psi at 0.2 inch deflection range.

The value of 30 psi is suggested as the Equivalent Lateral Stress for Soil I (CBRI Soil).

4.2.3.2 Equivalent Lateral Stress for Soil II (Blue Soil).

The load deformation curve for the penetration test shown in Figure 11 is interposed among the set of curves for the triaxial tests for various values of lateral fluid pressures shown in Figure 14. This is presented in Figure 20.

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It is found from Figure 19 that there is no single triaxial test with a definite value of lateral fluid pressure whose load-deformation curve is identical to that of the penetration test. This is for the obvious reason that as in any of the triaxial test the lateral fluid pressure is held constant, the lateral stresses in the loaded soil mass in a penetration test goes on increasing as the vertical load increases. The objective of the study, therefore, is limited to the determination of the approximate value of Equivalent Lateral Stress for the range of deflections used in design practice, namely 0.1 to 0.2 inch. For this range, the load-deformation curve falls clearly between the load-deformation curves for lateral fluid pressures 20 psi and 30 psi in triaxial test. It is nearer to 20 psi at 0.1 inch deflection range and to 30 psi at 0.2 inch deflection range.

The value of 30 psi is suggested as the Equivalent Lateral Stress for Soil I (CBRI Soil).

4.2.3.2 Equivalent Lateral Stress for Soil II (Blue Soil).

The load deformation curve for the penetration test shown in Figure 11 is interposed among the set of curves for the triaxial tests for various values of lateral fluid pressures shown in Figure 14. This is presented in Figure 20.

For the deflection ranges of 0.1 inch and 0.2 inch the load-deformation curve of the penetration test lies between the load-deformation curves for lateral fluid pressures of 0 psi and 5 psi in the triaxial test. It lies very close to the 5 psi curve in the 0.2 inch deflection range.

The value of 5 psi is suggested as the Equivalent Lateral Stress for Soil II (Blue Soil).

4.2.3.3 Equivalent Lateral Stress for Soil III(Blended Soil)

The load-deformation curve for the penetration test shown in Figure 12 is interposed among the set of curves for the triaxial tests for the various values of lateral fluid pressures shown in Figure 15 . This is presented in Figure 21.

For the deflection ranges of 0.1 inch and 0.2 inch the load deformation curve of the penetration test lies between the load-deformation curves for the lateral fluid pressure of 0 psi and 5 psi in triaxial test. It is very close to the 5 psi curve in the 0.2 inch deflection range.

The vaho of 5 psi is suggested as the Equivalent Lateral Stress for Soil III (Blended Soil).

4.2.3.4 According to the analysis developed in the preceding pages , it is clear from Figures 19, 20 and 21 that the Equivalent Lateral Stress developed in subgrade under vertical loads is a function of the deflection level. The trend of the above plots seems to indicate that the Equivalent Lateral Stress will go on increasing with deflection until its possible maximum value may be attained when failure occurs under the penetration test. As mentioned before, it may be observed that the present study is limited to an analysis of the effect of lateral confinement within the deflection level used in flexible pavement design, namely , 0.1 inch and 0.2 inch.

4.2.4 Tube Test

For any given value of deflection, the Equivalent Lateral Stress will depend upon the factors that contribute to lateral confinement in subgrade. As has been explained in Chapter II, the tube test, for given soil conditions, represent the maximum possible degree of lateral confinement, although unattainable in field. Load-deformation curves of the tube tests of all the three soils illustrate a common pattern. They register a steep rise of load till a deflection level of about 0.025 inch. Then the rate of load increase slows down for a small range upto a deflection range of about 0.1 inch, beyond which the load rise is steep and steady. This common pattern of load-deformation curves of the three soils may be explained as follows :

4.2.3.4 According to the analysis developed in the preceding pages , it is clear from Figures 19, 20 and 21 that the Equivalent Lateral Stress developed in subgrade under vertical loads is a function of the deflection level. The trend of the above plots seems to indicate that the Equivalent Lateral Stress will go on increasing with deflection until its possible maximum value may be attained when failure occurs under the penetration test. As mentioned before, it may be observed that the present study is limited to an analysis of the effect of lateral confinement within the deflection level used in flexible pavement design, namely , 0.1 inch and 0.2 inch.

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At the start of the loading process, the load is resisted both by inter-grannular pressure as well as the pore water pressure, the drainage being little at this stage. This stage is represented by the initial steep rise of the curve. But soon the process of drainage sets in, and is accelerated as the soil grains get compressed. This accelerated release of pore-water effects a reduction in rate load increase. This process quickly ceases to have dominating effect as soon as the inter grannular contacts of soil grains becomes greater. The continued load increase depends for the most part on the resistance offered by the inter-grannular pressure and very little on pore-water pressure, which continues to drain out all the time. This inter-grannular pressure is contributed by the resistance to vertical settlement and also the lateral confinement of the soil.

4.2.5 "Percent Lateral Confinement".

For any given deflection, say 0.2 inch, the corresponding load value of the tube test is a measure of the maximum possible mobilisation of the effect of lateral confinement, under given soil conditions. This load value may be said to correspond to 100 percent Lateral confinement which can be expressed in terms of an Equivalent Lateral Stress.

Similarly the load value corresponding to 0.2 inch deflection in a triaxial test with zero lateral pressure may be said to correspond to zero percent lateral confinement whose "Equivalent Lateral Stress" value is zero.

In actual field, the subgrade will have an Equivalent Lateral Stress in between zero and that corresponding to the 100 percent lateral confinement. It is possible to determine the above three values of Equivalent Lateral Stress. Having defined the scale of zero to 100 percent of lateral confinement in terms of Equivalent Lateral Stresses and knowing the value of the Equivalent Lateral Stress for the subgrade, it is possible to express the Equivalent Lateral Stress of the subgrade as a Percent Lateral Confinement.

4.2.5.1 Determination of Percent Lateral Confinement of a Soil for Given Deflection.

A plot is developed with load values vs lateral pressures for the given deflection, say 0.2 inch. For soil I (CBRI Soil) the values are read from Table 14, and the plot is presented in Figure 22. In developing this plot, lateral pressures which give values of corresponding loads higher than the load at 0.2 inch deflection in tube test are neglected, since they are not to be considered in the scale of zero to 100 percent, lateral confinement explained above. A nearly straight line plot is obtained.

The load value corresponding to 0.2 inch deflection(read from Table 11)in tube test is marked on the

Y-axis. It is projected on the extended straight line plot. This meeting point is projected down on the X-axis. This point, as read on the X-axis (stress scale), gives the value of Equivalent Lateral Stress in psi corresponding to 100 percent lateral confinement. The Percent Lateral Confinement Scale is now marked for 0 to 100 percent. Having determined the Equivalent Lateral Stress of the subgrade, it is entered on the stress axis and projected on the Percent Lateral Confinement scale. This value gives the Percent Lateral Confinement of the subgrade for the given deflection.

The above procedure may be illustrated as follows :-

Figure 22 represents the load vs lateral stress for deflection 0.2 inch for Soil I (CBRI Soil).

The Load corresponding to 0.2 inch deflection in the tube test	=	155.6 lbs
Equivalent lateral stress corresponding to 100 percent lateral confinement.	=	42.5 psi
Equivalent Lateral Stress of Soil I (CBRI Soil)	=	30 psi

Percent Lateral Confinement of Soil I (CBRI Soil) for 0.2 inch deflection level $= \frac{30}{42.5} \times 100$

$= 70.60$ percent

Y-axis. It is projected on the extended straight line plot. This meeting point is projected down on the X-axis. This point, as read on the X-axis (stress scale), gives the value of Equivalent Lateral Stress in psi corresponding to 100 percent lateral confinement. The Percent Lateral Confinement Scale is now marked for 0 to 100 percent. Having determined the Equivalent Lateral Stress of the subgrade, it is entered on the stress axis and projected on the Percent Lateral Confinement scale. This value gives the Percent Lateral Confinement of the subgrade for the given deflection.

The above procedure may be illustrated as follows :-

Figure 22 represents the load vs lateral stress for deflection 0.2 inch for Soil I (CBRI Soil).

The Load corresponding to 0.2 inch deflection in the tube test	=	155.6 lbs
Equivalent lateral stress corresponding to 100 percent lateral confinement.	=	42.5 psi
Equivalent Lateral Stress of Soil I (CBRI Soil)	=	30 psi

Percent Lateral Confinement of Soil I (CBRI Soil) for 0.2 inch deflection level $= \frac{30}{42.5} \times 100$

$= 70.60$ percent

Similalrly from Tables 15 and 16 and corresponding plots in Figures 23 and 24 for Soil I and Soil II respectively.

Percent Lateral Confinement of Soil II (Blue Soil) for 0.2 inch deflection level = 6.13 percent

Percent Lateral Confinement of Soil III (Blended Soil) for 0.2 inch deflection level. = 6.85 percent

-:O:-

CHAPTER - V

APPLICATION OF TEST RESULTS

CHAPTER - V

APPLICATION OF TEST RESULTS

5.1 The results of the present study have certain direct applications in flexiblepavement design. These are explained below :

5.1.1 1) Determination of the Modulus of Elasticity (E) of Subgrade Soil :

The value E is made use of in the Kansas design method for flexible pavements to account for the strength of the subgrade. To determine E, an undrained triaxial test with a lateral fluid pressure of 20 psi and rate of loading 0.05 is run. The value of E is obtained from the stress-strain plot.

The value of lateral pressure used in the triaxial test is 20 psi for all types of soils. In other words, the Equivalent Lateral Stress due to the effect of lateral confinement under vertical loads is assumed to be the same (20 psi) for all soil types (8).

The results of the present study, however, show that the value of Equivalent Lateral Stress is significantly affected by the type of soil. It has a value of as high as 30 psi for Soil I (CBRI Soil), whereas it is as low as 5 psi for Soil II (Blue Soil) and Soil III (Blended Soil). It may, therefore, be suggested that in a triaxial test for determination of E of soil I, Soil II, and soil III, the lateral fluid pressures to be used should be respectively 30 psi, 5 psi and 5 psi.

The method of calculation of E is indicated in Appendix III, part A. The values of E corresponding to the standard lateral pressure (20 psi) and the suggested value for each soil are calculated in Appendix III, part B, and presented in Table 17. For each pressure, E values are calculated at deflection levels 0.1 inch and 0.2 inch, assuming the soil to be elastic over this range. The stresses are calculated for the effective cross-section area of the sample at the respective deflection level. Thus there are four E values for each soil.

As may be seen from Table 17 for any given deflection, the values of E for the standard value of lateral pressure (20 psi) and the suggested value of lateral pressure show a marked range of variation. For instance, for soil II (Blue Soil) at deflection 0.1 inch, the value of E corresponding to the standard lateral pressure (20 psi) is 435 psi where-as that corresponding to the actual Equivalent Lateral Stress (5 psi) is only 261 psi.

This variation in E value has a marked influence in the design thickness of flexible pavement as explained below.

5.1.2 2) Design Thickness of Flexible Pavement According to Kansas Design Method.

The Kansas design method using triaxial test accounts for the effect of lateral confinement in subgrade by applying a lateral fluid pressure of 20 psi in the triaxial testing of subgrade soil. Therefore, this design method is used in the present project to determine the pavement thicknesses for the standard value of lateral pressure for the three soils. The results are used for comparitive study.

The method of determining the design thickness is explained in Appendix IV Part A.

Appendix IV, part B presents calculations for four values of design thicknesses for each soil type corresponding to the four values of E. The results are presented in Table 17. It is seen that for any deflection level, the variation in thicknesses between those corresponding to the standard value of lateral pressure and suggested value of lateral pressure is remarkable. This is particularly so in the case of Soil I (CBRI soil) and Soil II (Blue oil Soil). A diagrammatic representation of the variation in thicknesses is presented in bar charts in Figures 25, 26 and 27.

5.1.9 C.B.R. of Subgrade Soil Related to Variation of Design Thickness.

The percentage variation in thickness corresponding to Equivalent Lateral Stress over the thickness corresponding to the standard value of lateral pressure (20 psi) is calculated as shown in Appendix V and

and presented in Table 17.

The percentage variation in thickness as plotted against C.B.R. value of soil as shown in Figure 23. The plots for both deflections of 0.1 inch and 0.2 inch are nearly similar and follow close ranges. They cross the zero line (X-axis) at a C.B.R. range of about 2. The nature of these plots indicate that ;

i) For soils having C.B.R. values less than about 2 percent the required thickness of the pavement is greater than that obtained by using the standard value of lateral pressure (20 psi) in triaxial test. In other words, the Equivalent Lateral Stress for soils having C.B.R. value less than about 2 percent, is less than 20 psi.

ii) For soils having C.B.R. values greater than about 2 percent, the required thickness of the pavement is less than that obtained by using the standard value of lateral pressure (20 psi) in triaxial test. In other words, the Equivalent Lateral Stress for soils having C.B.R. value greater than about 2 percent is greater than 20 psi.

iii) For soils having C.B.R. value around 2 percent, required thickness of pavement is the same as that obtained by using the standard value of Lateral Pressure (20 psi) in the triaxial test. In other words, the Equivalent Lateral Stress for and presented in Table 17.

The percentage variation in thickness as plotted against C.B.R. value of soil as shown in Figure 23. The plots for both deflections of 0.1 inch and 0.2 inch are nearly similar and follow close ranges. They cross the zero line (X-axis) at a C.B.R. range of about 2. The nature of these plots indicate that ;

i) For soils having C.B.R. values less than about 2 percent the required thickness of the pavement is greater than that obtained by using the standard value of lateral pressure (20 psi) in triaxial test. In other words, the Equivalent Lateral Stress for soils having C.B.R. value less than about 2 percent, is less than 20 psi.

ii) For soils having C.B.R. values greater than about 2 percent, the required thickness of the pavement is less than that obtained by using the standard value of lateral pressure (20 psi) in triaxial test. In other words, the Equivalent Lateral Stress for soils having C.B.R. value greater than about 2 percent is greater than 20 psi.

iii) For soils having C.B.R. value around 2 percent, required thickness of pavement is the same as that obtained by using the standard value of Lateral Pressure (20 psi) in the triaxial test. In other words, the Equivalent Lateral Stress for soils having C.B.R. values around 2 percent is 20 psi.

5.1.4 Group Index of Subgrade Related to Variation of Design Thickness.

The percentage variation in thickness is plotted against G.I. value of soils as shown in Fig. 29. The plots for both deflections of 0.1 inch and 0.2 inch are nearly similar and follow close ranges. They cross the zero line (x-axis) at a G.I. value of about 7.30. The nature of these plots indicate that ;

i) For soils having G.I. greater than about 7.30 the required thickness of the pavement is greater than that obtained by using the standard value of lateral stress (20 psi) in triaxial test. In other words, the Equivalent Lateral Stress for soils having G.I. greater than about 7.30 is less than 20 psi.

ii) For soils having G.I. less than about 7.30 the required thickness of the pavement is less than that obtained by using the standard value of lateral pressure (20 psi) in tri-axial test. In other words, Equivalent Lateral Stress for soils having G.I. less than 7.30 is greater than 20 psi.

iii) For soils having G.I.around 7.30, the required thickness of pavement is the same as that obtained by using the standard value of lateral pressure (20 psi) in the triaxial test . In other words, the Equivalent Lateral Stress for soils having G.I. around 7.30 is 20 psi.

5.1.5 5) Classification of Soils Based on C.B.R. and G.I. Values

The preceeding analysis would show that soils having C.B.R. values of the order of 2 percent and G.I. values of the order of 7.30 are of basic significance in this approach. It may go observed that the plot C.B.R. vs G.I. presented in Figure 30 shows C.B.R. 2 percent as equivalent to a G.I. with little over 7.3 namely, 8.

This provides us with a basis to classify soils into three basic categories for the purpose of application of Kansas Design method. These three categories are :

Group I.

Soils having a C.B.R. of around 2 percent or G.I. of around 7.30 percent, for those soils the Kansas' Design method with a lateral fluid pressure of 20 psi for triaxial test is applicable.

Group II.

Soils having C.B.R. values below 2 percent or G.I. above 7.30. For these soils the Kansas design method with lateral pressure of 20 psi in triaxial test will result in under-design. The lateral pressure to be used is less than 20 psi. If the lateral pressure used is 20 psi, the design thickness obtained needs to be corrected as indicated in articles 5.1. 6.1 and 5.1.6.2.

Group III.

Soils having C.B.R. values greater than 2 percent or G.I. less than 7.30. For these soils the Kansas design method with lateral pressure of 20 psi in triaxial test will result in over-design. The lateral pressure to be used is greater than 20 psi. If the lateral pressure used is 20 psi, the design thickness obtained needs to be corrected as indicated in articles 5.1.6.1 and 5.1.6.2 .

5. 1.6 6) Correction to Design Thickness :

The experimental processes involved in the determination of Equivalent Lateral Stress of a soil are too delicate and time consuming to be applied in practical design work. Instead the present practice of using 20 psi as lateral pressure (now defined as Equivalent Lateral Stress) may be retained, but a correction to the thickness so obtained may be applied. This is intended to make the design thickness realistic to the actual Equivalent Lateral Stress of the soil which might be different from 20 psi.

The plots giving percentage variation in thickness versus C.B.R. and G.I. of soils presented in Figure 28 and 29 provide a simple basis for the correction. The thickness of the flexible pavement (T inches) is determined from Kansas design method using a lateral pressure of 20 psi.

5.1.6 (a) Correction Based on C.B.R.

The C.B.R. of the soil is determined. If this value falls around 2 percent, T needs no correction. If the C.B.R. value falls below 2 the corresponding percent increase in thickness to be added to T is directly read from plot for the given deflection in Figure 28. If this is t percent, the corrected thickness of pavement is T + T x t/100 = T (1 + t/100) inches. Similarly, the thickness = T (1 - t/100) inches if the C.B.R. value is greater than 2 percent.

5.1.6.2 b) Correction Based on G.I.

The G.I. of the soil is determined. If this value falls around 7.30, T needs no correction. If the G.I. falls above 7.30 the corresponding percent increase in thickness to be added to T is directly read from plot for the given deflection in Figure 29. If this is t percent the corrected thickness of pavement is T + T x t/100 = T (1 + t/100) inches. Similarly, the thickness = T (1 - t/100) inches if the G.I. of soil is less than 7.30.

5.1.7 7) Design Deflection Criteria

It has been shown that the percent lateral confinement mobilised to resist vertical loads increases with deflection level. For economic considerations it will be necessary to define the deflection designs criteria in such a way that the highest permissible deflection level within the range of elastic (or nearly elastic) behaviour of soil be chosen. This will ensure the mobilisation of a considerable amount of Percent Lateral Confinement and thus result in economic design thickness. The deflection levels of 0.1 inch and 0.2 inch which are the present deflection criteria for design may now be examined in this context.

5.1.7.1 a) Deflection Criterion of 0.1 Inch.

From plots presented in Figures 19, 20 and 21, it may be seen that the load deformation curve for tube test has only a very small rise above the load-deformation curve of the triaxial test with zero lateral pressure in the deflection range upto 0.1 inch. This indicates that at 0.1 inch deflection level only a very small Percent Lateral Confinement is mobilised to resist vertical loads. It may be said that the load is mostly carried by the resistance to vertical settlement offered by the subgrade. This implies that 0.1 inch deflection is a conservative design criterion which might result in over design.

5.1.7.2 b) Deflection Criterion of 0.2 inch.

From plots presented in Figure 19, 20 and 21, it may be seen that the load deformation curve for tube test has a marked rise above the load-deformation curve of the triaxial test with zero psi lateral pressure at the 0.2 inch deflection level. This indicates that at 0.2 inch deflection level a considerable Percent Lateral Confinement is mobilized to resist vertical loads. It may be said that the resistance to lateral deformation of subgrade plays a considerable role along with resistance to vertical deformation in resisting the vertical load. This implies that 0.2 inch deflection is a more economic design criterion than 0.1 inch and results in a smaller factor of safety.

In the process of design the role of percent Lateral Confinement is incorporated by making realistic choice of deflections criteria and the value of Equivalent Lateral Stress to be used in triaxial test for determination of the value of E. For a given deflection level, the higher the value of Equivalent Lateral Stress used, the higher the percent Lateral Confinement expected to be available. This emphasizes the need for realistic evaluation of Equivalent Lateral Stress for the subgrade soil for design.

Plots in Figures 22, 23 and 24 and Table 17 present the Percent Lateral Confinement at 0.2 inch deflection for the standard values and suggested values of lateral pressures for the three soils. For Soil I Percent Lateral Confinement is only 47 percent for the standard value of lateral pressure (20 psi) whereas it is 70.6 percent for the suggested value of Lateral Pressure

(30 psi). Corresponding design thicknesses are 15.00 inches and 12.13 inches, on the other hand for the other two soils in whose case the suggested value of lateral pressure is only 5 psi and the corresponding percent Lateral Confinement for Soil II and Soil III are 6.13 percent and 6.85 percent . Corresponding values for the standard value of lateral pressure (20 psi) are 24.51 percent and 27.4 percent. The design thickness shows marked increase with decreasing Percent Lateral Confinement in the case of Soil I. It is 25.10 inches and 14.72 inches respectively for the suggested values of lateral pressure for two soils, as against 19.00 and 13.67 inches for the standard value.

With a higher value of Equivalent Lateral Stress the Percent Lateral Confinement available is higher. Since Equivalent Lateral Stress increases with the deflection level, the 0.2 inch deflection criteria results in greater utilization of lateral confinement and thus results in economy in design thickness as shown in Figures 25, 26 and 27.

5.2 Thus this discussion demonstrates the role of lateral confinement in subgrade under vertical loads and illustrates its application in the design of flexible pavements.

CHAPTER - VI

CONCLUSIONS AND SCOPE FOR FURTHER WORK

CHAPTER - VI

CONCLUSIONS AND SCOPE FOR FURTHER WORK

6.1 Based on the experimental investigation carried out and the various assumptions made in the course of developing the analysis as reported herein, the following conclusions are drawn :

6.1.1 1) The effect of lateral confinement in subgrade under vertical loads is a significant factor to be taken into account in the design of flexible pavements.

6.1.2 2) Although the exact distribution pattern of the lateral stress developing in a subgrade under vertical loads is not known, it is possible to evaluate an Equivalent lateral Stress for known conditions of load and deflection.

6.1.3 3) The value of Equivalent Lateral Stress varies with soil types.

6.1.4 4) The value of Equivalent Lateral Stress of 20 psi used in triaxial test for the determination of the modulus of elasticity is not applicable to all soil types.

6.1.5 5) For soils having a C.B.R. around 2 percent or Group Index around 7.30, the Equivalent Lateral Stress of 20 psi is applicable. Hence for those soils the design thicknesses as obtained by the current practice of Kansas design method are satisfactory.

6.1.6 6) For soils having a C.B.R. below 2 Group Index above 7.30, the Equivalent Lateral Stress to be used in triaxial test for Kansas design method is lower than 20 psi. If 20 psi is used, it results in under design. The variations in the design thicknesses were 41.30 percent and 32.10 percent for design ~~xxxxxx~~ deflection criteria 0.1 inch and 0.2 inch respectively, in the case of Soil II (Blue Soil), as given in Table 17. However, a method is indicated to effect a correction for the thickness so obtained.

6.1.7 7) For soils having a C.B.R. above 2 or Group Index below 7.30, the Equivalent Lateral Stress to be used in triaxial test for Kansas design method is greater than 20 psi. If 20 psi is used, it results in over-design. The variations in design thicknesses were 27.82 percent and 19.14 percent for design deflection criteria 0.1 inch and 0.2 inch respectively, in the case of Soil I (CBRI Soil), as given in Table 17. However, a method is indicated to effect a correction for the thickness so obtained.

6.1.8 8) With a deflection criteria of 0.1 inch in design the role of lateral resistance to displacement in soils in carrying vertical loads is insignificant. The mobilization of the effect of lateral confinement is more pronounced with a deflection level of 0.2 inch as design criterion.

6.2 The following problems are suggested for further experimental study :-

6.2.1 In the present investigation triaxial tests were performed with lateral pressures of 0, 5, 10, 15, 20, 30, 40, 50, and 60 psi. It is seen from Figures 19, 20, 21 that for a more accurate evaluation of Equivalent Lateral Stress by comparison and interpolation between load-deformation curves of penetration test and triaxial tests, it is necessary to have triaxial tests data for closer intervals of lateral pressures than the 5 psi and 10 psi intervals used in the present investigation. It may be useful to conduct this study for typical subgrade soils and lay down specifications for the lateral pressures to be used for each.

6.2.2 Plots as shown in Figures 28 and 29 relating percentage variation in thicknesses to the C.B.R. and G.I. values may be developed using standard subgrade soil types. Such standardised plots may be of immediate use to apply necessary corrections to thicknesses obtained by the standard design procedure. It may be of interest also to examine how corrections for the same soil based on C.B.R. and G.I. compare with each other.

TABLES

TABLE - I

SOIL PROPERTIES

Soil Designation	Per cent passing No.40 sieve.	Per-cent passing No.200 sieve	Liqduid Limit Percent	Plasticity index	Textural classification	USPRA Soil Group	Group index	OMC per cent	Max. D.D. in pcf.
Soil I (CDRI Soil)	84	42.7	30	10	Sandy Loam	A-4	1.6	10	110
Soil II (Blue Soil)	100	99	51	14	Silty loam	A-7	11.8	18	96
Soil III (Blended Soil)	92	72	35	11	Silty Loam	A-6	8.4	14	102

TABLE - 2

CORRECTED LOAD-DEFLECTION DATA OF CBR TEST FOR SOIL I (CBRI SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	22.10
0.050	43.25
0.075	64.80
0.100	86.75
0.150	129.80
0.200	164.00
0.300	221.50
0.400	270.00
0.500	316.50

TABLE - 3

CORRECTED LOAD-DEFLECTION DATA OF CBR TEST FOR SOIL II (BLUE SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	9.07
0.050	17.14
0.075	23.05
0.100	28.50
0.150	39.15
0.200	49.70
0.300	71.00
0.400	92.60
0.500	117.00

TABLE - 4

CORRECTED LOAD-DEFLECTION DATA OF CBR TEST

FOR SOIL III (BLENDED SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	10.09
0.050	20.40
0.075	30.61
0.100	41.75
0.150	60.75
0.200	82.00
0.300	124.10
0.400	159.20
0.500	188.00

TABLE - 5

CORRECTED LOAD-DEFORMATION DATA OF PENETRATION TEST (1.5 INCH DIA. PLUNGER) FOR SOIL I (CBRI SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	4.53
0.050	5.97
0.075	6.91
0.100	12.67
0.125	22.55
0.150	32.10
0.175	47.80
0.200	63.70
0.225	82.75
0.250	97.20
0.275	118.00
0.300	130.60
0.325	143.00
0.350	151.30
0.375	160.00
0.400	169.40

TABLE - 6

CORRECTED LOAD-DEFLECTION DATA OF PENETRATION TEST (1.5 INCH DIA. PLUNGER) FOR SOIL II (BLUE SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	4.53
0.050	7.42
0.075	9.78
0.100	13.74
0.125	16.57
0.150	20.15
0.175	24.30
0.200	29.15
0.225	32.10
0.250	36.20
0.275	40.25
0.300	43.15
0.325	49.70
0.350	54.00
0.375	58.20
0.400	61.80

TABLE - 7

CORRECTED LOAD-DEFLECTION DATA OF PENETRATION TEST (1.5 INCH PLUNGER) FOR SOIL III (BLENDED SOIL)

Deflection in inches	Load in pounds
0.000	0.00
0.025	3.24
0.050	4.39
0.075	6.19
0.100	8.21
0.125	12.90
0.150	18.23
0.175	23.05
0.200	28.85
0.225	34.60
0.250	41.00
0.275	49.80
0.300	58.15
0.325	66.20
0.350	72.00
0.375	79.50
0.400	87.50

TABLE - 8

LOAD-DEFLECTION DATA OF TRIAXIAL TEST FOR VARIOUS LATERAL PRESSURES FOR SOIL I (CBRI SOIL)

Deflections in inches	Load in lbs								
	Lateral Pressures in psi								
	0	5	10	15 *	20	30	45	50 *	60
0.000	0.00	0.00	0.00		0.00	0.000	0.00		0.00
0.025	8.14	17.44	28.10		42.25	64.40	88.83		143.40
0.050	13.75	24.72	33.30		49.10	82.70	105.60		188.30
0.075	19.96	33.65	38.90		56.65	88.00	114.25		219.10
0.100	27.40	41.50	45.20		62.90	95.30	130.40		24.2.50
0.125	33.80	49.10	52.10		68.65	106.30	143.55		257.60
0.150	37.70	57.50	59.30		73.80	115.10	159.00		282.10
0.175	38.40	61.60	64.65		81.40	123.30	174.10		289.40
0.200	42.15	65.95	73.40		93.10	131.50	188.60		305.30
0.225		69.30	77.95		101.40	138.50	202.50		318.50
0.250		71.10	80.85		107.90	144.30	212.60		328.10
0.275		72.55	86.75		113.20	150.50	222.70		336.60
0.300		73.30	90.60		117.50	156.80	229.30		348.60
0.325			92.20		121.20	161.10	230.90		354.90
0.350			94.95		125.60	164.00	243.40		359.60
0.375			96.00		129.10	167.60	244.90		362.30
0.400			96.70		133.00	170.90	246.40		366.60

* Reading not recorded.

TABLE - 9

LOAD-DEFLECTION DATA OF TRIAXIAL TEST FOR VARIOUS LATERAL PRESSURES FOR SOIL II (BLUE SOIL)

Deflection in inches	Load in lbs.								
	Lateral Pressures in psi								
	0	5	10	15	20	30	40	50	60
0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.025	1.80	18.80	26.96	29.35	47.08	55.32	79.18	96.35	123.30
0.050	5.04	19.50	29.85	30.36	50.60	55.73	85.89	98.90	125.05
0.075	7.56	22.12	32.44	39.35	57.50	71.88	86.60	113.40	127.55
0.100	9.36	25.00	35.46	47.65	63.10	74.25	92.15	122.80	129.35
0.125	13.75	27.16	37.25	50.75	66.80	82.70	106.40	127.10	122.95
0.150	18.73	31.50	39.90	56.30	70.50	97.10	111.90	132.30	144.85
0.175	22.10	31.05	43.22	62.60	73.80	103.90	118.65	137.00	154.60
0.200	25.90	35.10	47.50	64.10	76.90	110.45	123.00	141.80	168.60
0.225	28.80	37.90	50.60	67.15	79.90	116.50	127.75	147.00	181.30
0.250	36.00	44.10	54.20	71.10	82.00	121.80	131.90	151.30	190.20
0.275	39.60	45.50	60.10	74.30	85.65	127.30	136.76	155.45	199.20
0.300	40.30	49.50	60.90	77.70	89.20	131.50	141.80	161.70	206.17
0.325	41.45	49.90	66.80	80.65	93.00	138.40	146.85	167.20	208.90
0.350		52.40	70.85	83.00	96.60	140.30	150.55	171.20	215.40
0.375		54.10	73.90	86.20	98.20	145.00	154.40	175.75	222.20
0.400		55.60	75.40	89.80	100.60	148.50	158.20	176.70	225.90

TABLE - 10

LOAD DEFLECTION DATA OF TRIAXIAL TEST FOR VARIOUS LATERAL PRESSURES FOR SOIL XXI (BLENDED SOIL)

Deflection in inches	Load in lbs — Lateral Pressures in psi								
	0	5	10	15	20	30	40	50	60
0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.025	6.48	16.78	32.94	42.60	43.95	60.30	84.65	93.77	112.8
0.050	9.15	25.00	43.90	51.30	52.85	64.95	95.25	97.50	130.80
0.075	12.04	35.20	44.50	56.00	62.70	71.24	104.60	108.45	137.30
0.100	14.55	43.00	59.35	62.80	71.00	79.90	114.20	118.90	147.35
0.125	17.43	48.80	66.60	69.60	79.80	89.60	122.80	137.70	170.40
0.150	21.40	54.95	74.10	76.50	85.96	97.95	131.50	155.20	202.10
0.175	24.60	60.30	81.35	84.60	94.35	109.90	140.20	164.00	227.00
0.200	29.40	67.00	87.60	91.20	100.90	118.60	150.80	175.60	248.60
0.225	32.60	69.20	98.30	99.00	108.00	126.80	160.00	188.15	265.65
0.250	36.20	71.50	90.10	106.15	114.90	134.80	169.15	197.20	274.10
0.275	40.80	74.30	103.00	112.60	121.00	144.30	176.40	207.20	289.30
0.300	43.20	78.40	106.30	118.20	126.20	150.05	183.40	214.70	300.10
0.325		79.60	109.60	122.50	130.80	155.80	196.70	221.00	307.60
0.350		81.80	111.60	126.15	134.30	163.10	201.10	228.50	314.60
0.375		83.60	114.40	130.40	137.80	166.30	204.90	234.50	325.00
0.400		85.30	115.10	132.60	140.30	177.80	209.60	239.00	326.60

TABLE - 11

LOAD -DEFLECTION DATA OF TUBE TEST FOR SOIL I

(CBRI SOIL)

Deflections in inches	Load in pounds
0.000	0.00
0.025	9).43
0.050	10.56
0.075	14.15
0.100	25.00
0.125	43.20
0.150	70.70
0.175	106.10
0.200	155.60
0.225	214.70
0.250	302.00
0.275	415.00
0.300	514.00

TABLE - 12

LOAD-DEFLECTION DATA OF TUBE TEST FOR SOIL II

(BLUE SOIL)

Deflections in inches	Load in pounds
0.000	0.00
0.025	10.37
0.050	10.93
0.075	13.20
0.100	26.00
0.125	54.70
0.150	113.20
0.175	130.20
0.200	228.50
0.225	415.00
0.250	777.50
0.275	1161.00
0.300	1594.00

TABLE - 13

LOAD-DEFLECTION DATA OF TUBE TEST FOR SOIL III

(BLENDED SOIL)

Deflections in inches	Load in pounds
0.000	0.00
0.025	18.68
0.050	22.65
0.075	28.30
0.100	37.70
0.125	56.60
0.150	103.70
0.175	151.00
0.200	230.00
0.225	330.00
0.250	491.00
0.275	707.00
0.300	981.00

TABLE - 14

LOAD-LATERAL PRESSURE DATA FOR 0.2 INCH DEFLECTION FOR SOIL I (CBRI SOIL)

(From Table 8)

Load in pounds	Lateral pressure in psi
42.15	0
65.95	5
73.40	10
93.10	20
131.50	30

TABLE - 15

LOAD-LATERAL PRESSURE DATA FOR 0.2 INCH DEFLECTION FOR SOIL II (BLUE SOIL)

(From Table 9)

Load in lbs	Lateral Pressure in psi
25.90	0
35.10	51
47.50	10
64.10	15
76.90	20
110.45	30
123.00	40
141.80	50
168.60	60

TABLE - 16

LOAD -LATERAL PRESSURE DATA FOR 0.2 INCH DEFLECTION FOR SOIL III (BLENDED SOIL)

(From Table 10)

Load in pounds	Lateral Pressure in psi
28.40	0
67.00	5
87.00	10
91.20	15
100.90	20
118.60	30
150.80	40
175.60	50

TABLE - 17

Soil Designation	Soil I(CBRI Soil)				Soil II(Blue Soil)				Soil III(Blended soil)			
Lateral pressure in psi	20 standard		30 Suggested		20 Standard		5 Suggested		20 Standard		5 Suggested	
Allowable deflection in inches	0.1	0.2	0.1	0.2	0.1	0.2	0.1	0.2	0.1	0.2	0.1	0.2
Modulus of Elasticity in psi	432	437.50	665	592.5	435	310	261	203.5	566	500	588	457
Flexible pavement thickness T in inches	30.35	15.00	21.90	12.13	30.15	19	42.60	25.10	25.35	13.67	26	14.72
100 % lateral confinement in equivalent psi (for 0.2 inch deflection)		42.50				81.60				73.00		
% Lateral confinement utilized in design (for 0.2 inch deflection)		47.00		70.60		24.51		6.13		27.40		6.86
% variation in thickness for suggested lateral pressure over thickness for standard lateral pressure			-27.82	-19.14			+41.30	+ 32.10			+2.56	+ 7.68
C.B.R. in percent		3.64				1.10				1.82		
Group Index		1.60				11.80				8.40		

TABLE - 18

STRESS-STRAIN DATA FOR SOIL I (CERA SOIL)

Deflections in inches	Percent Strain	Lateral Pressure 20 psi Deviator stress in psi	Lateral Pressure 30 psi Deviator Stress in psi
0.000	0.00	0.00	0.00
0.025	0.83	3.92	6.50
0.050	1.66	7.75	16.75
0.075	2.50	12.10	19.75
0.100	3.33	15.60	24.00
0.125	4.16	18.80	30.25
0.150	5.00	21.78	35.20
0.175	5.83	26.15	39.00
0.200	6.66	32.70	44.50
0.225	7.50	37.50	49.50
0.250	8.33	41.00	51.80
0.275	9.16	44.20	55.50
0.300	10.00	46.50	59.80
0.325	10.83	48.70	61.30
0.350	11.66	51.15	63.40
0.375	12.50	53.15	65.00
0.400	13.33	55.30	66.75

TABLE - 19

STRESS STRAIN DATA FOR SOIL II (BLUE SOIL)

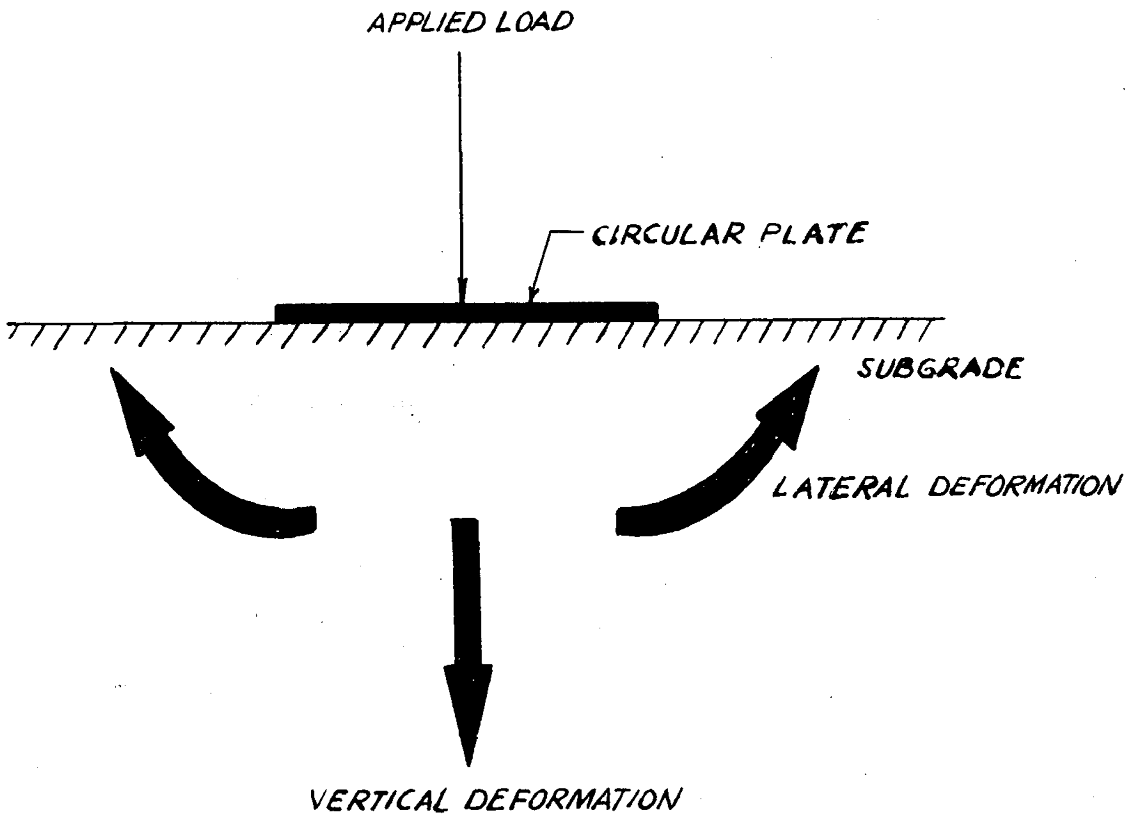
Deflections in inches	Percent strain	Lateral Pressure 20 psi	Lateral Pressure 5 psi
		Deviator stress in psi	Deviator stress in psi
0.000	0.00	0.00	0.00
0.025	0.83	6.60	5.64
0.050	1.66	8.60	6.04
0.075	2.50	12.50	7.52
0.100	3.33	15.70	9.30
0.125	4.16	17.90	10.95
0.150	5.00	19.90	12.74
0.175	5.83	21.75	13.05
0.200	6.66	23.50	14.85
0.225	7.50	25.20	15.95
0.250	8.33	26.40	19.90
0.275	9.16	28.50	20.75
0.300	10.00	30.50	23.00
0.325	10.83	32.65	23.25
0.350	11.66	34.70	24.70
0.375	12.50	35.55	25.60
0.400	13.33	36.90	26.45

TABLE - 20

STRESS-STRAIN DATA FOR SOIL III (BLENDED SOIL)

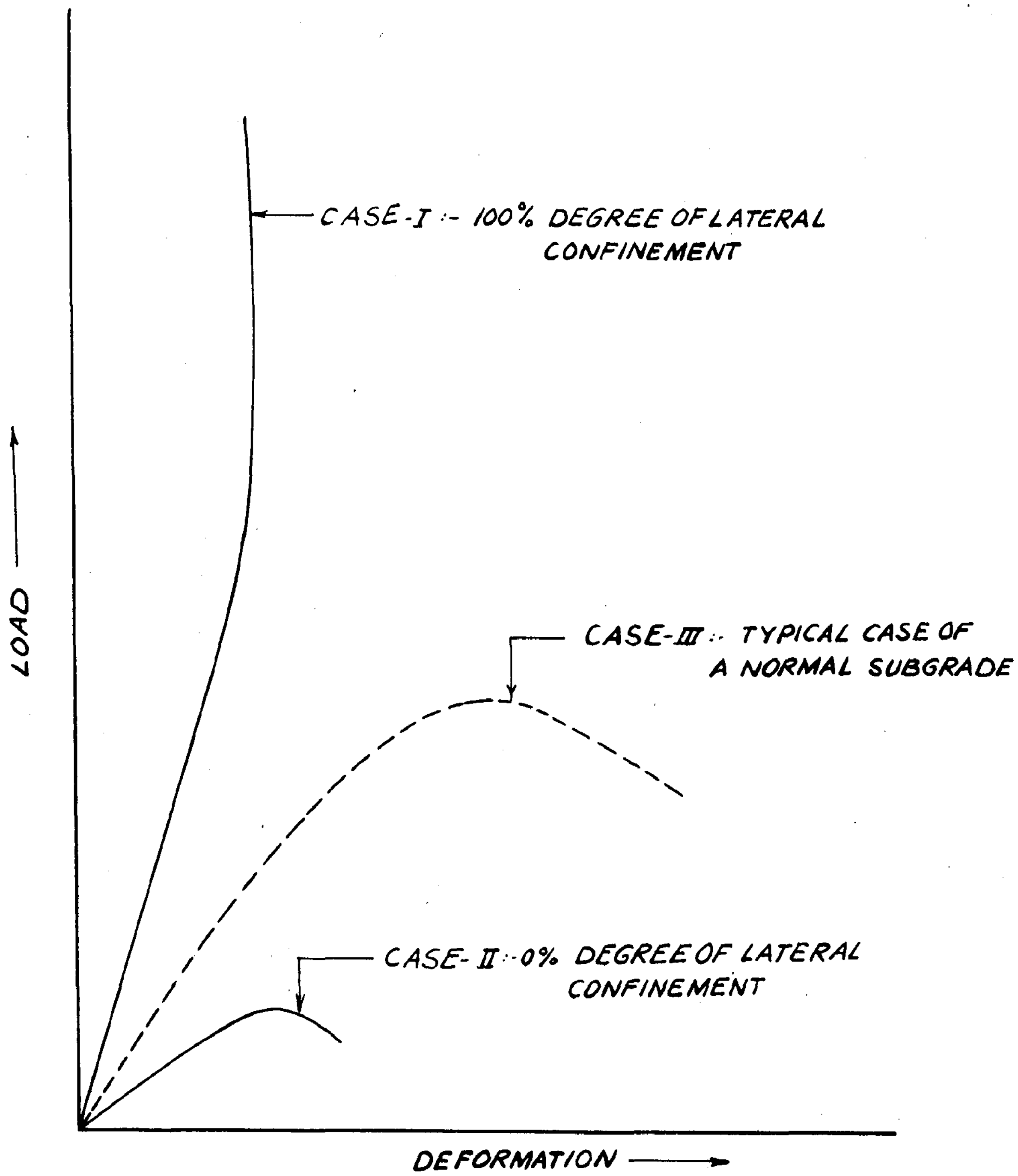
Deflections in inches	Percent strain	Lateral Pressure ~~is~~ 20 psi	Lateral Pressure 5 psi
		Deviator stress in psi	Deviator stress in psi
0.000	0.00	0.00	0.00
0.025	0.83	4.90	4.50
0.050	1.66	9.95	9.15
0.075	2.50	15.50	14.90
0.100	3.33	20.20	19.35
0.125	4.16	25.20	22.70
0.150	5.00	28.65	26.10
0.175	5.83	33.35	29.15
0.200	6.66	37.20	32.90
0.225	7.50	41.20	34.20
0.250	8.33	45.00	35.50
0.275	9.16	48.50	37.15
0.300	10.00	51.50	39.35
0.325	10.83	54.15	40.00
0.350	11.66	56.00	41.35
0.375	12.50	58.00	42.30
0.400	13.33	59.50	43.30

FIGURES



SCHEMATIC REPRESENTATION OF DISPLACEMENTS.

FIG. 1



EFFECT OF LATERAL CONFINEMENT IN STRESS-DEFORMATION PATTERN OF SUBGRADE

FIG. 2

GRAIN SIZE DISTRIBUTION OF SOILS

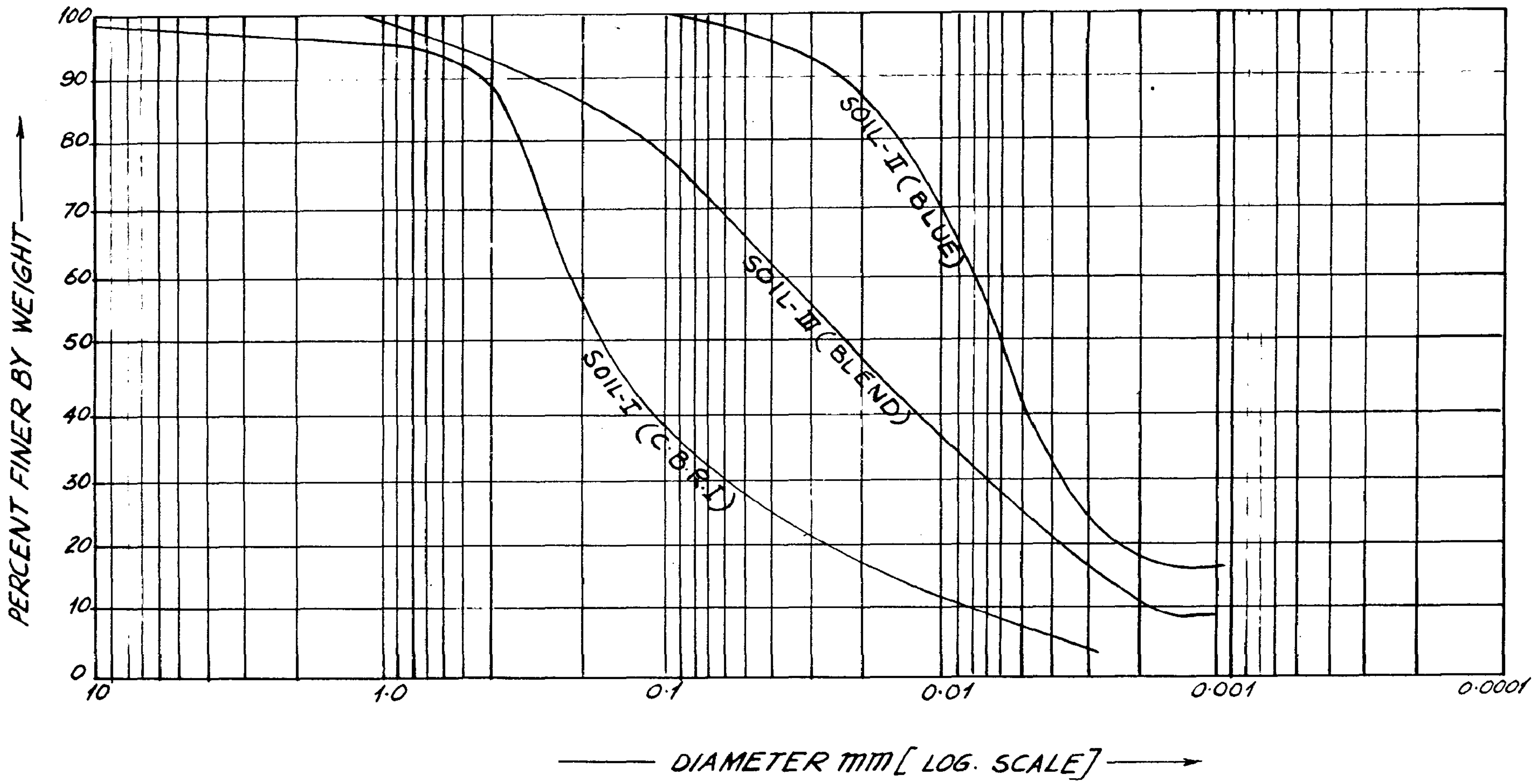
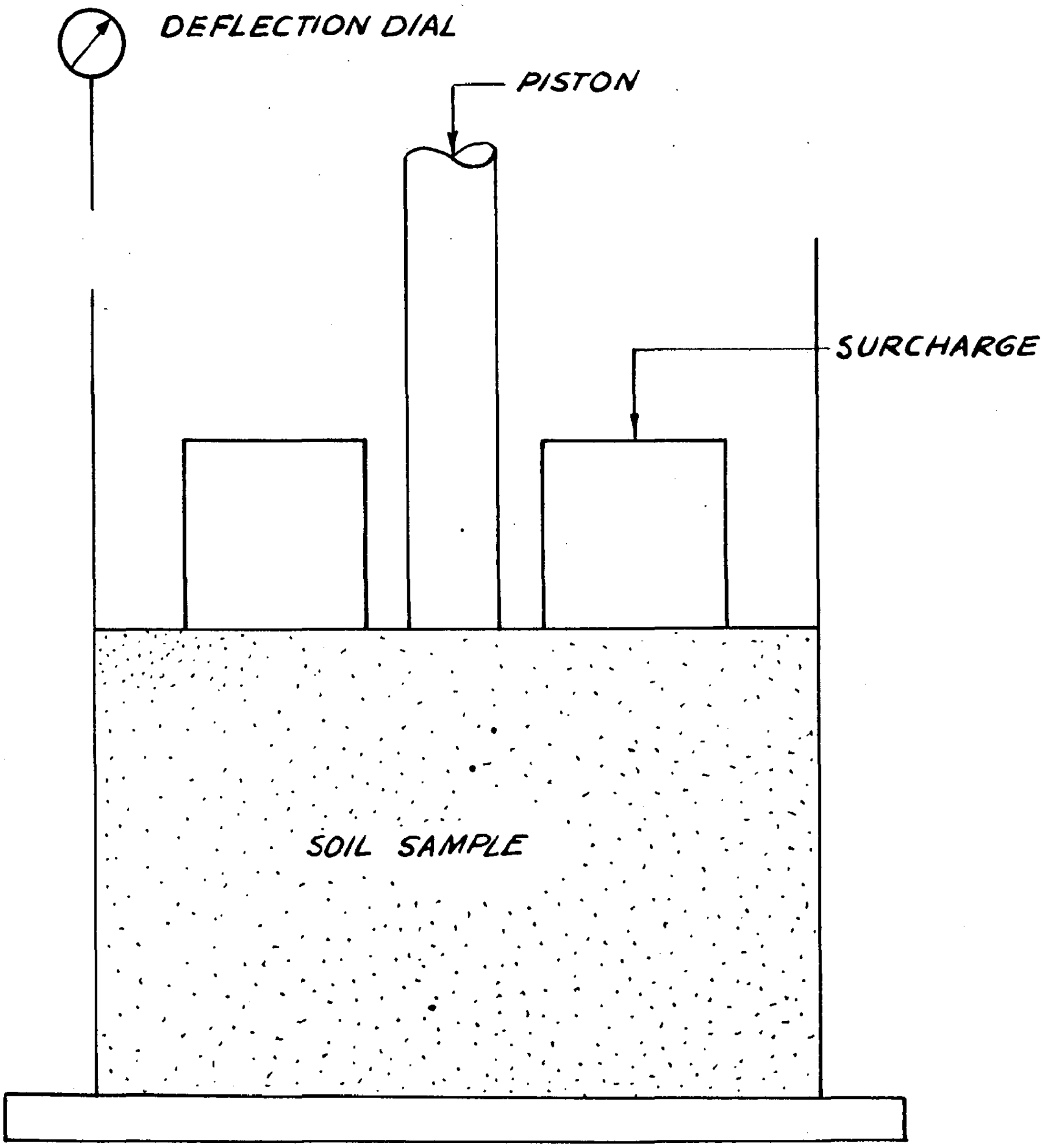
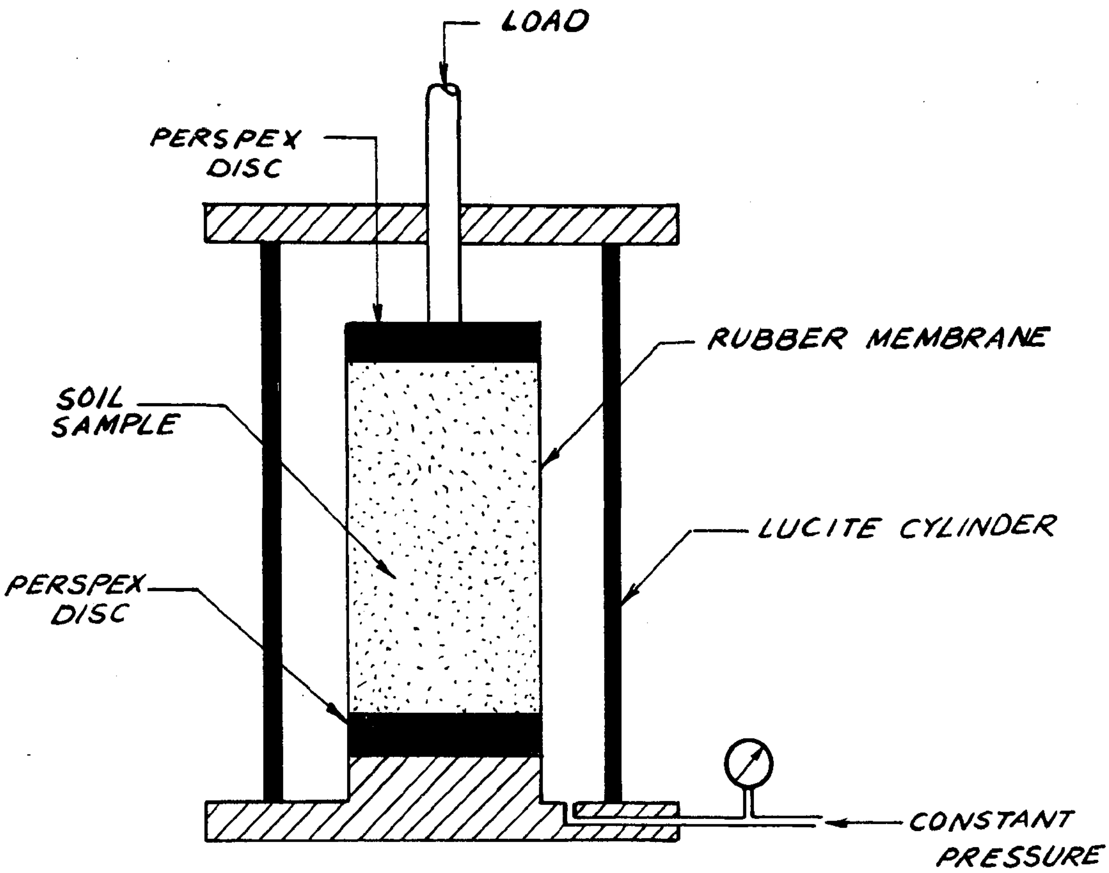


FIGURE. 3



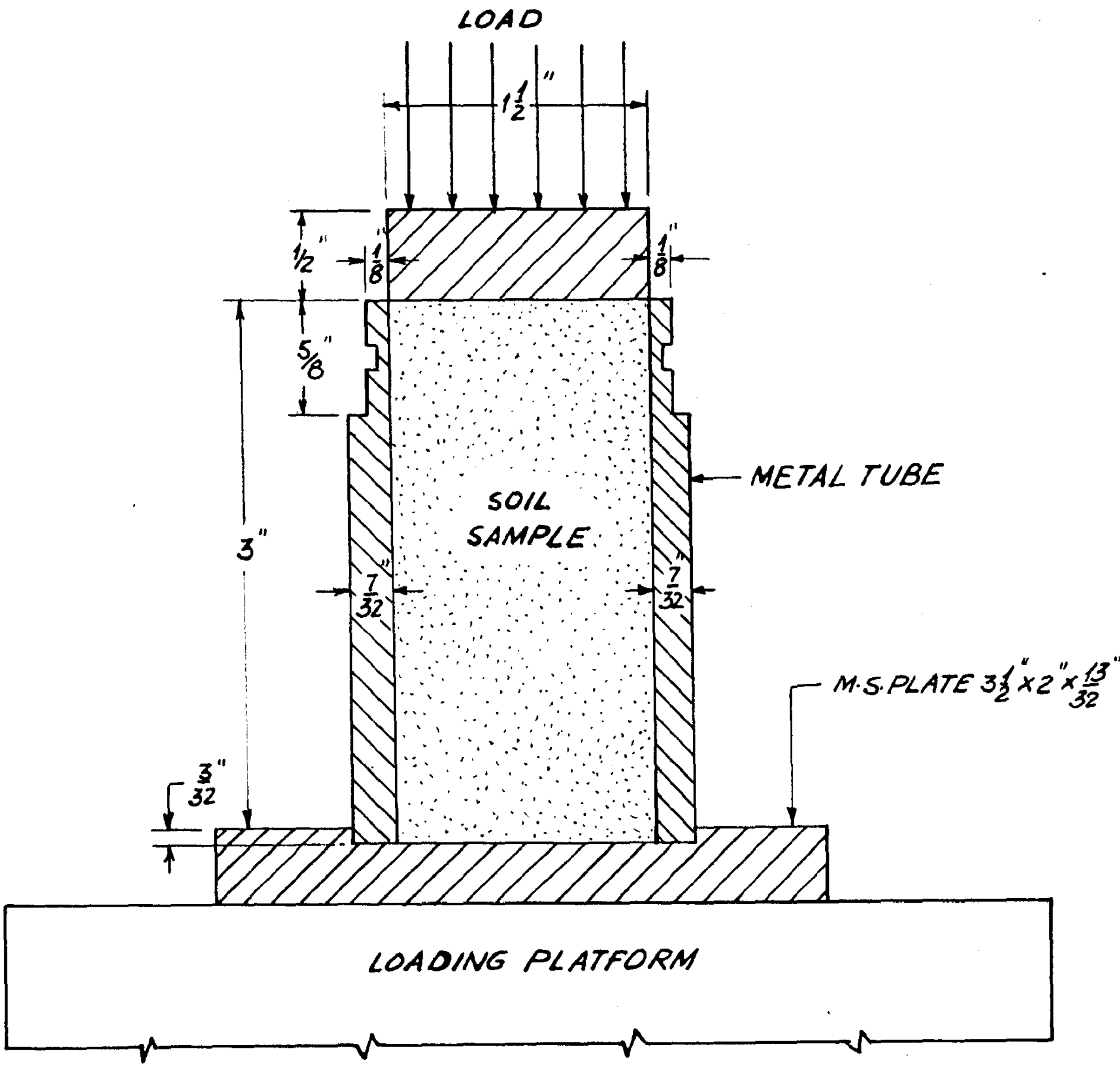
CALIFORNIA BEARING RATIO TEST-SCHEMATIC DIAGRAM

FIG. 4



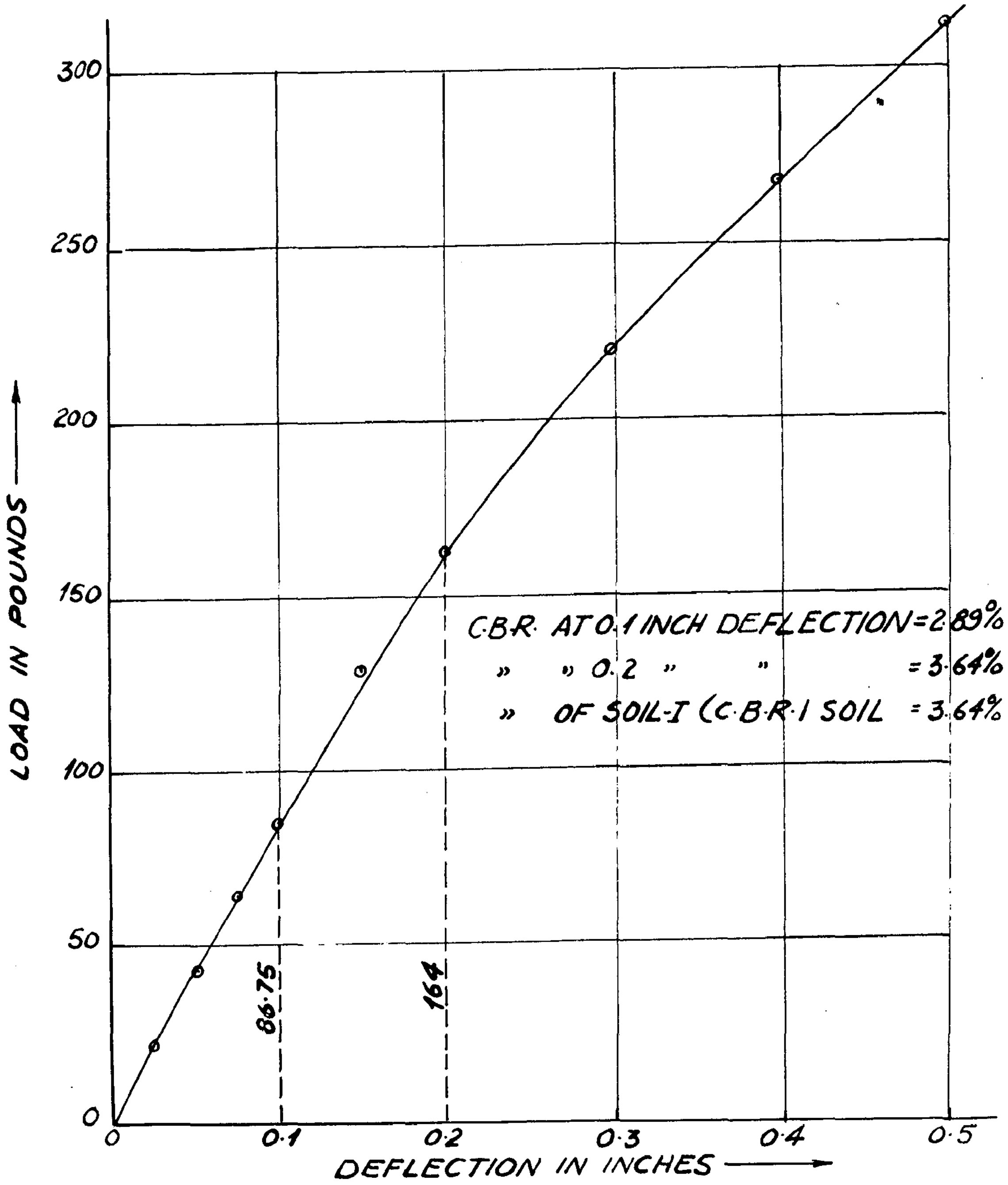
TRIAXIAL TEST [UNDRAINED] SET-UP-SCHEMATIC DIAGRAM

FIG.5



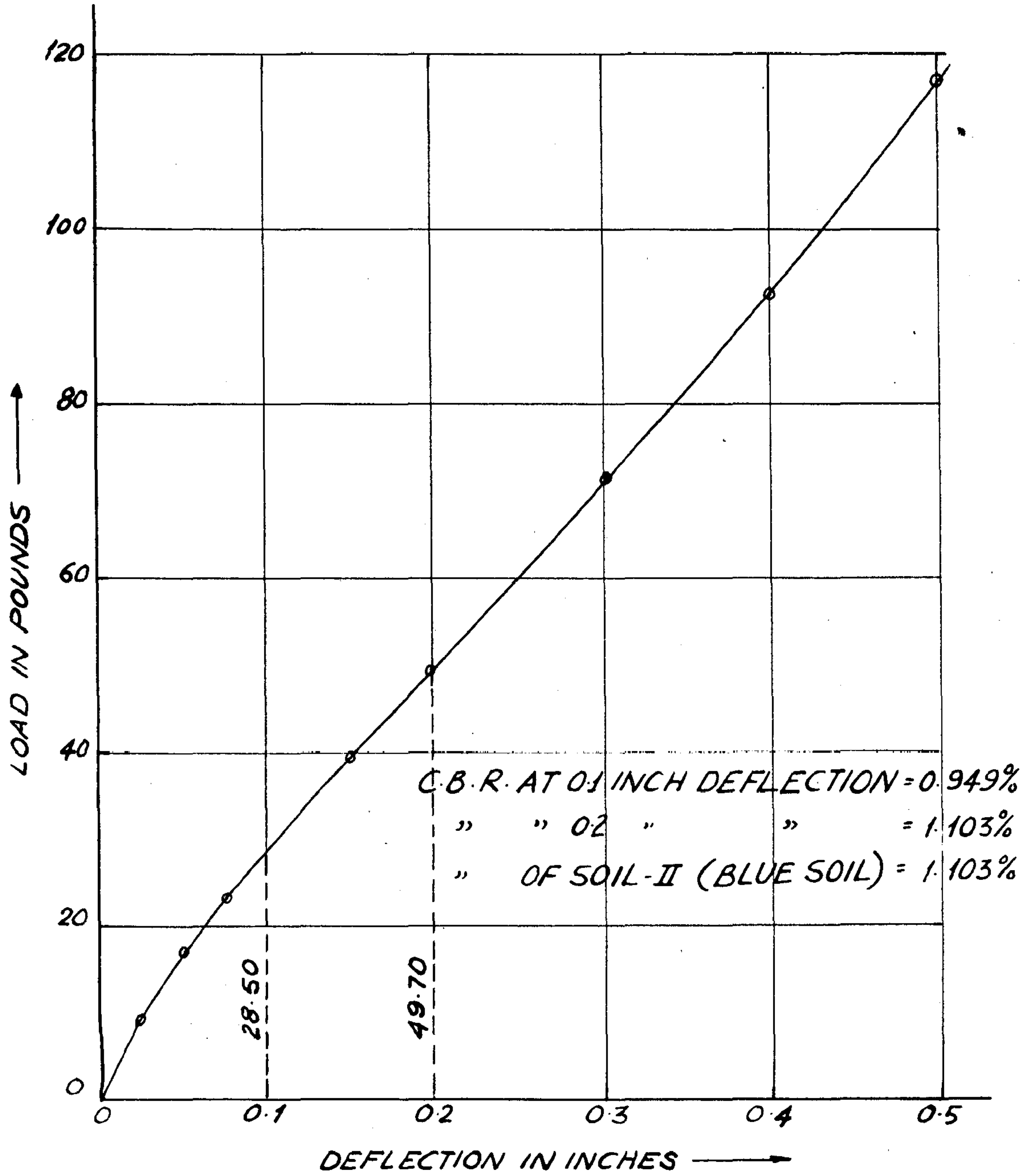
TUBE TEST SET-UP - SCHEMATIC DIAGRAM

FIG. 6



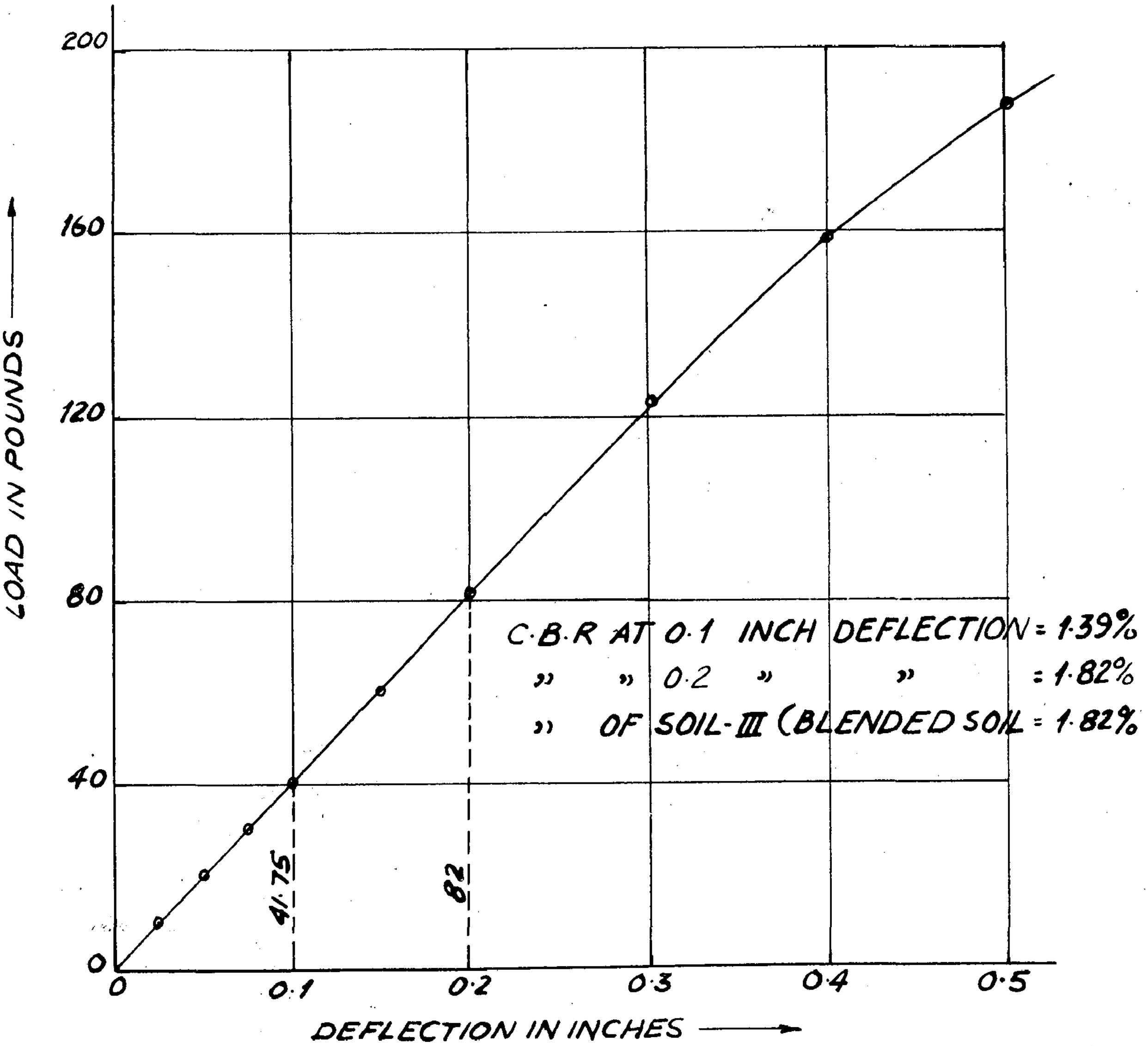
C·B·R· TEST PLOT FOR SOIL-I
(C·B·R·1 SOIL)

FIG. 7



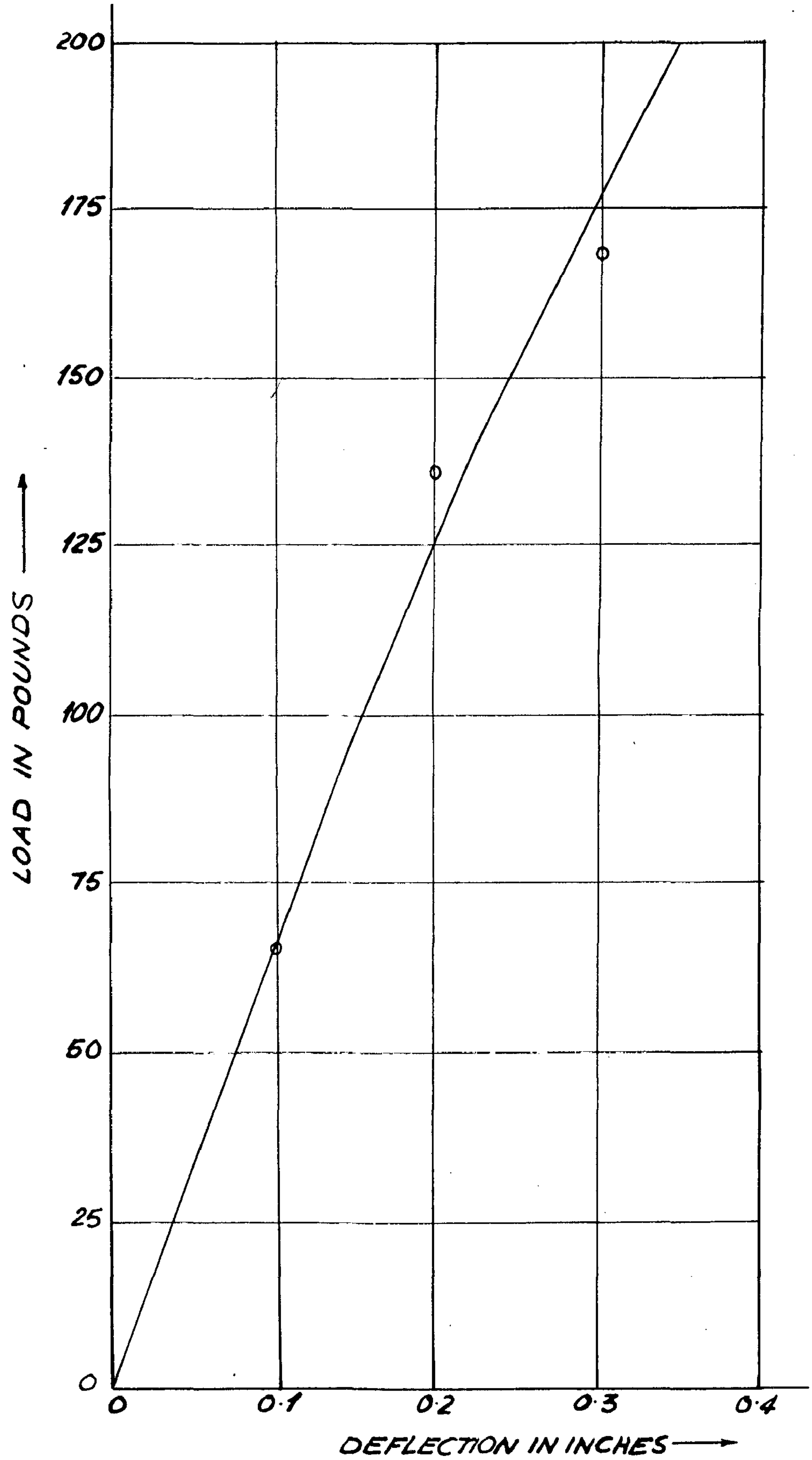
C·B·R· TEST PLOT FOR SOIL·II
(BLUE SOIL)

FIG.8



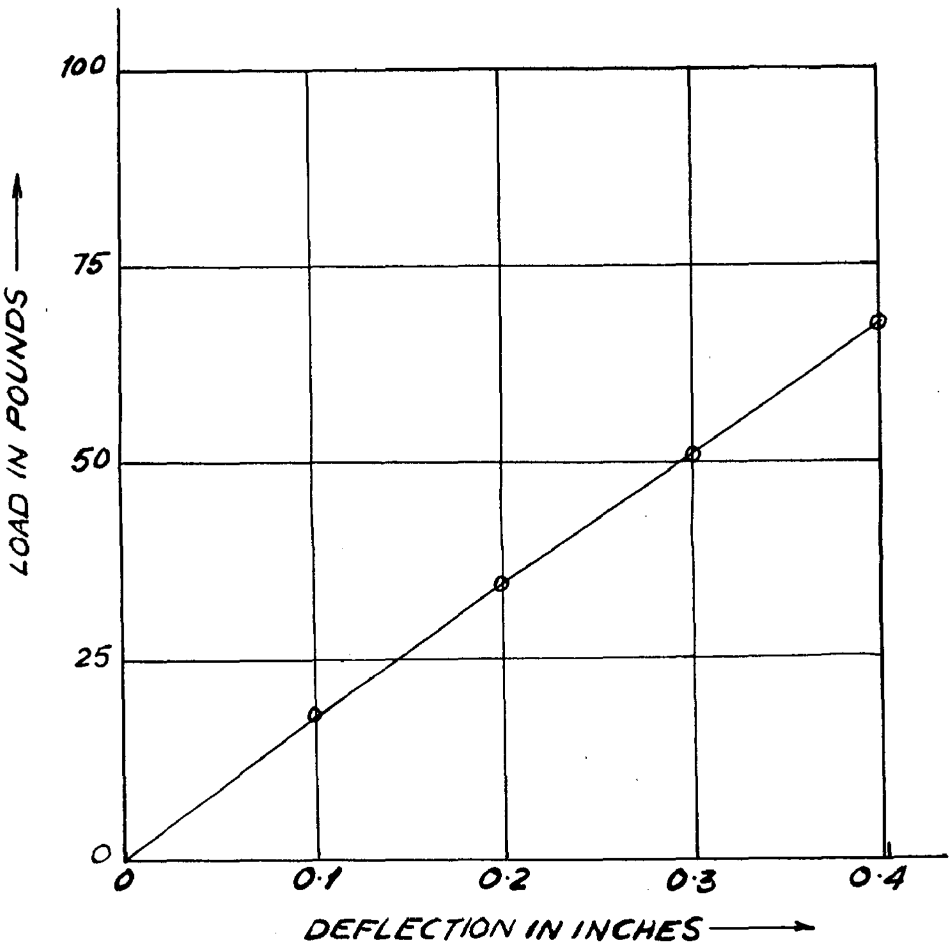
C·B·R· TEST PLOT FOR SOIL·III
(BLENDED SOIL)

FIG. 9



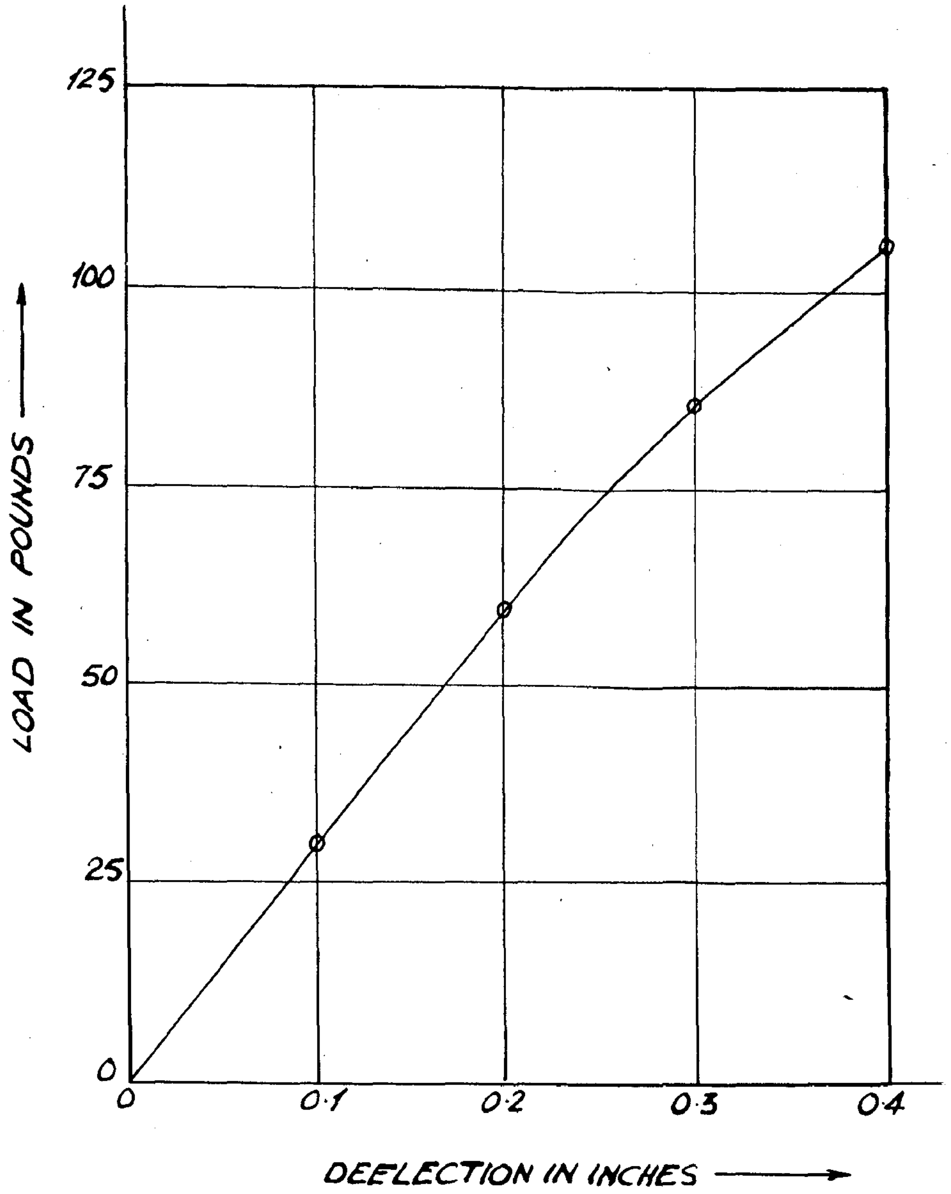
PENETRATION TEST WITH 1·5 INCH DIA. PLUNGER FOR SOIL-I (C·B·R·I· SOIL)

FIG. 10



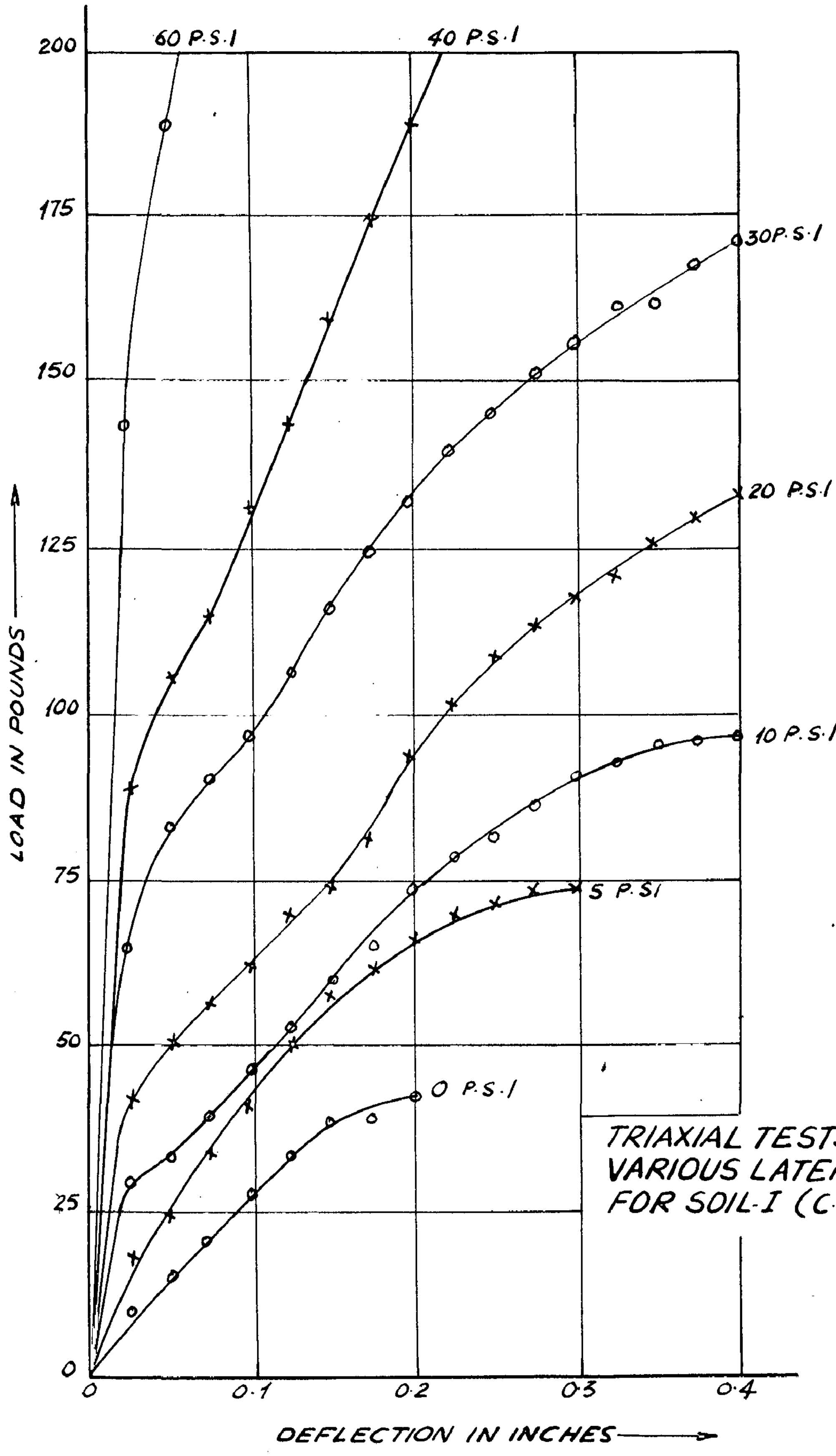
PENETRATION TEST WITH 1·5 INCH DIA. PLUNGER FOR SOIL-II (BLUE SOIL)

FIG. 11



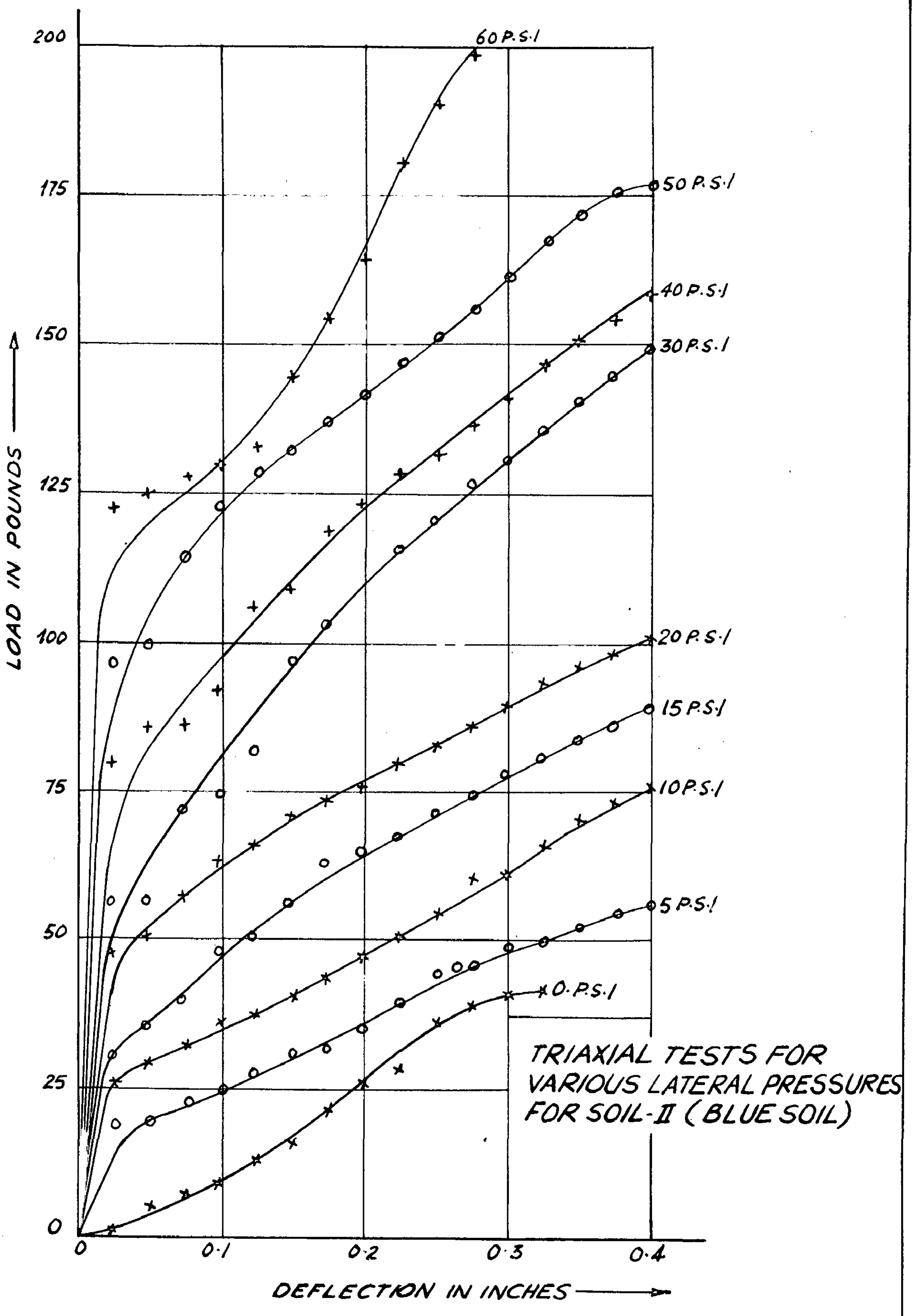
PENETRATION TEST WITH 1·5 INCH DIA PLUNGER FOR SOIL-III (BLENDED SOIL)

FIG. 12



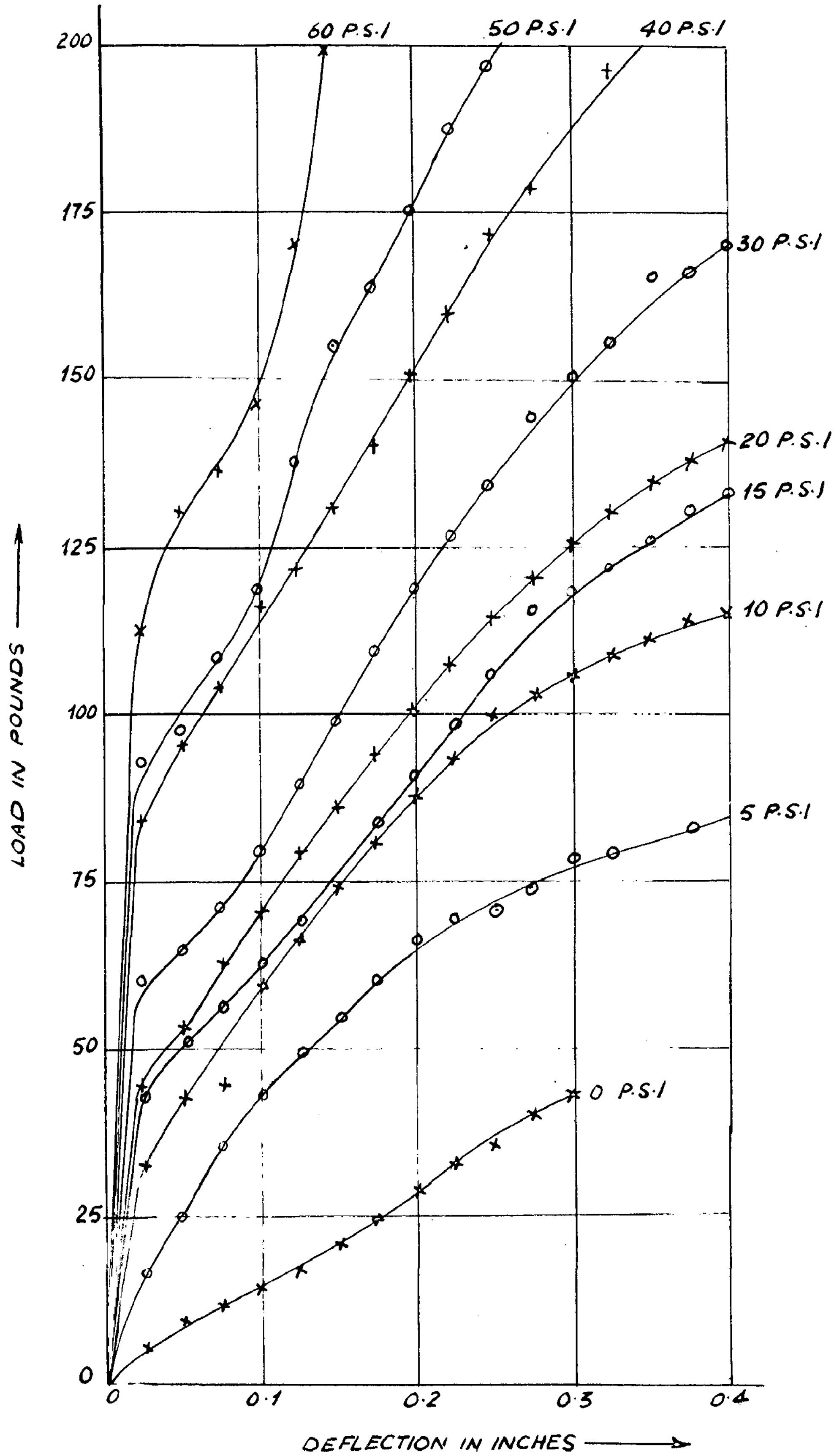
TRIAXIAL TESTS FOR VARIOUS LATERAL PRESSURE FOR SOIL-I (C.B.R.I. SOIL)

FIG. 13



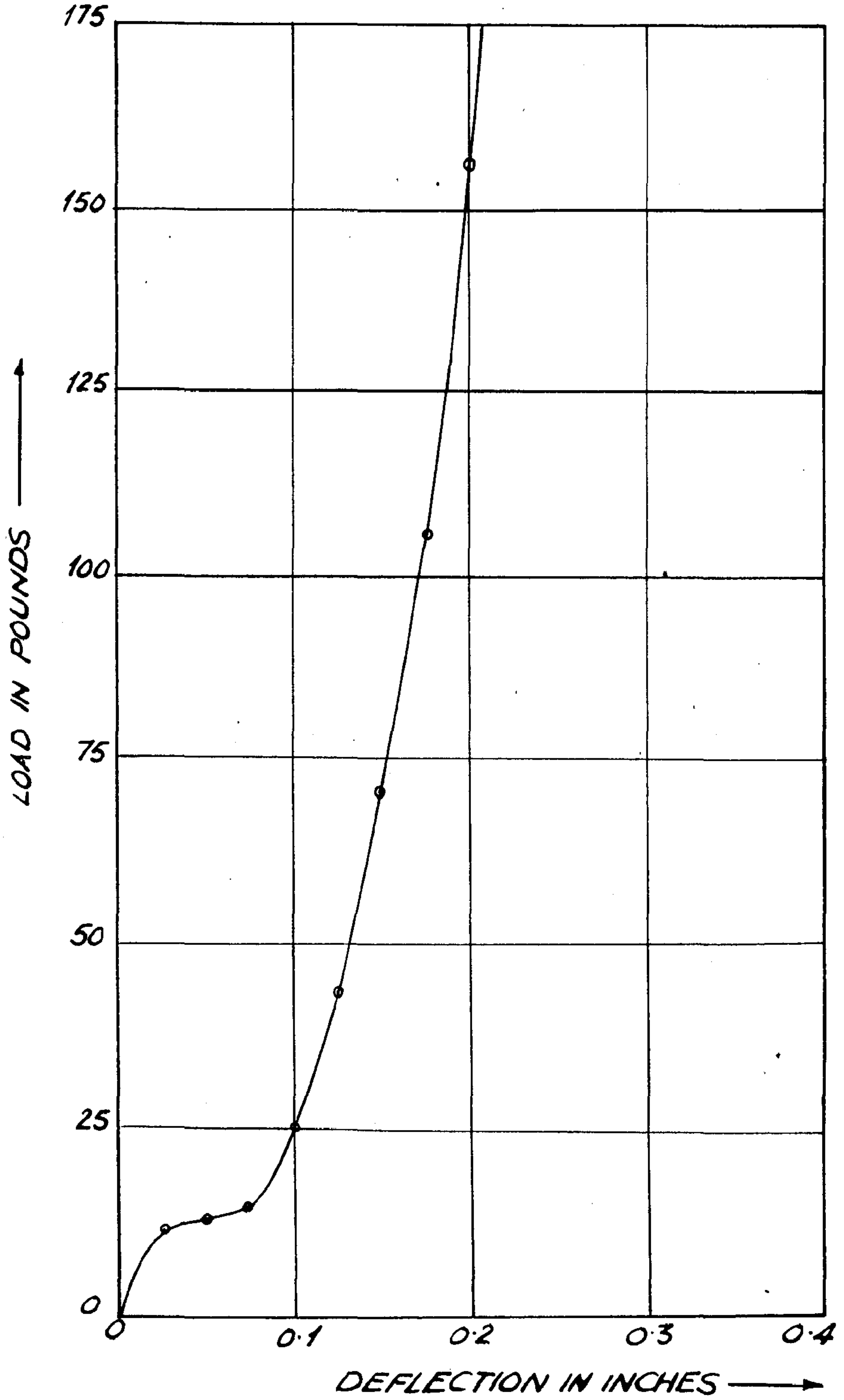
LOAD IN POUNDS
DEFLECTION IN INCHES
60 P.S.I
50 P.S.I
40 P.S.I
30 P.S.I
20 P.S.I
15 P.S.I
10 P.S.I
5 P.S.I
0 P.S.I
TRIAXIAL TESTS FOR VARIOUS LATERAL PRESSURES FOR SOIL-II (BLUE SOIL)

FIG. 14



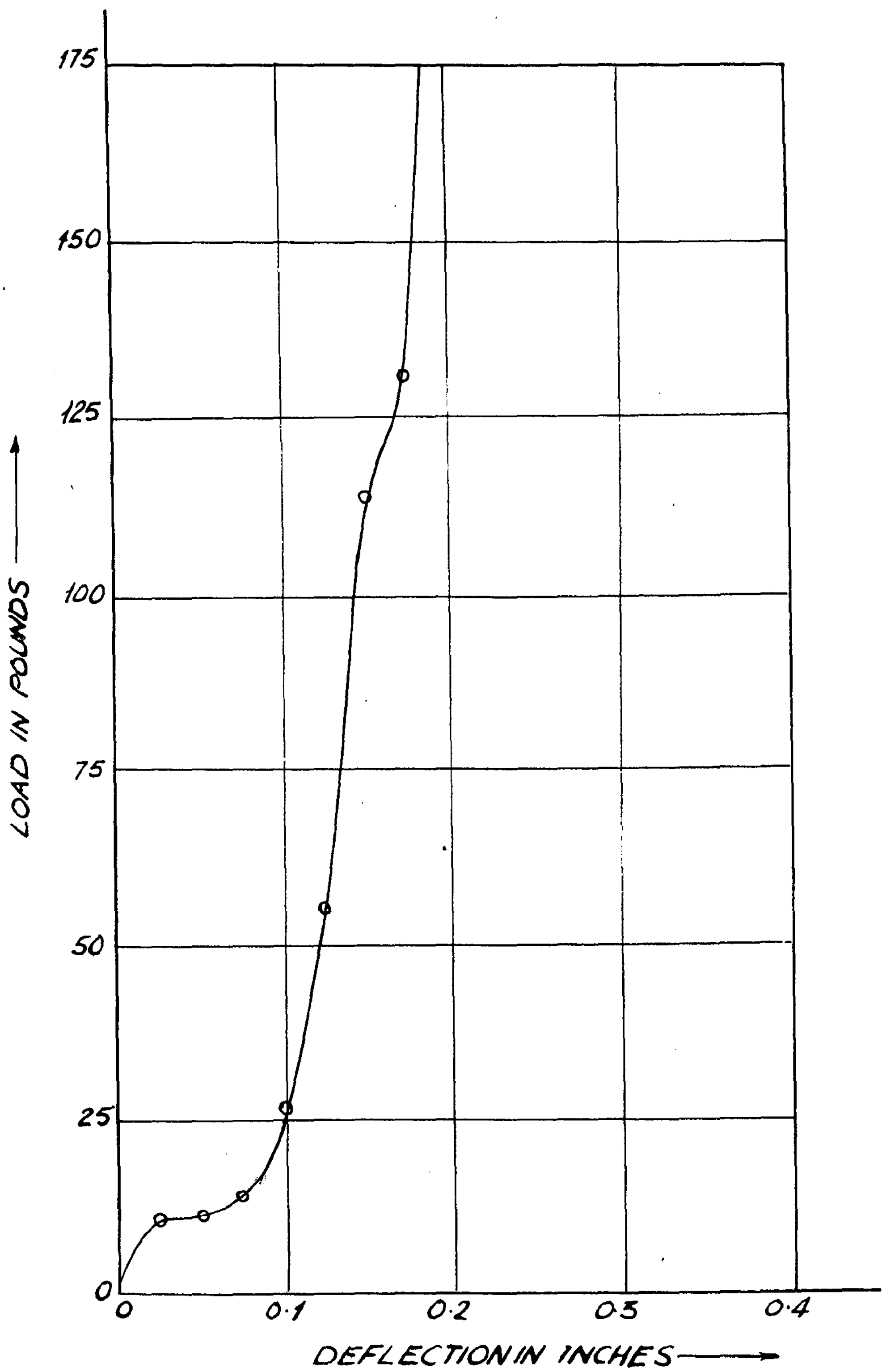
TRIAXIAL TESTS FOR VARIOUS LATERAL PRESSURES
FOR SOIL-III (BLENDED SOIL)

FIG 15



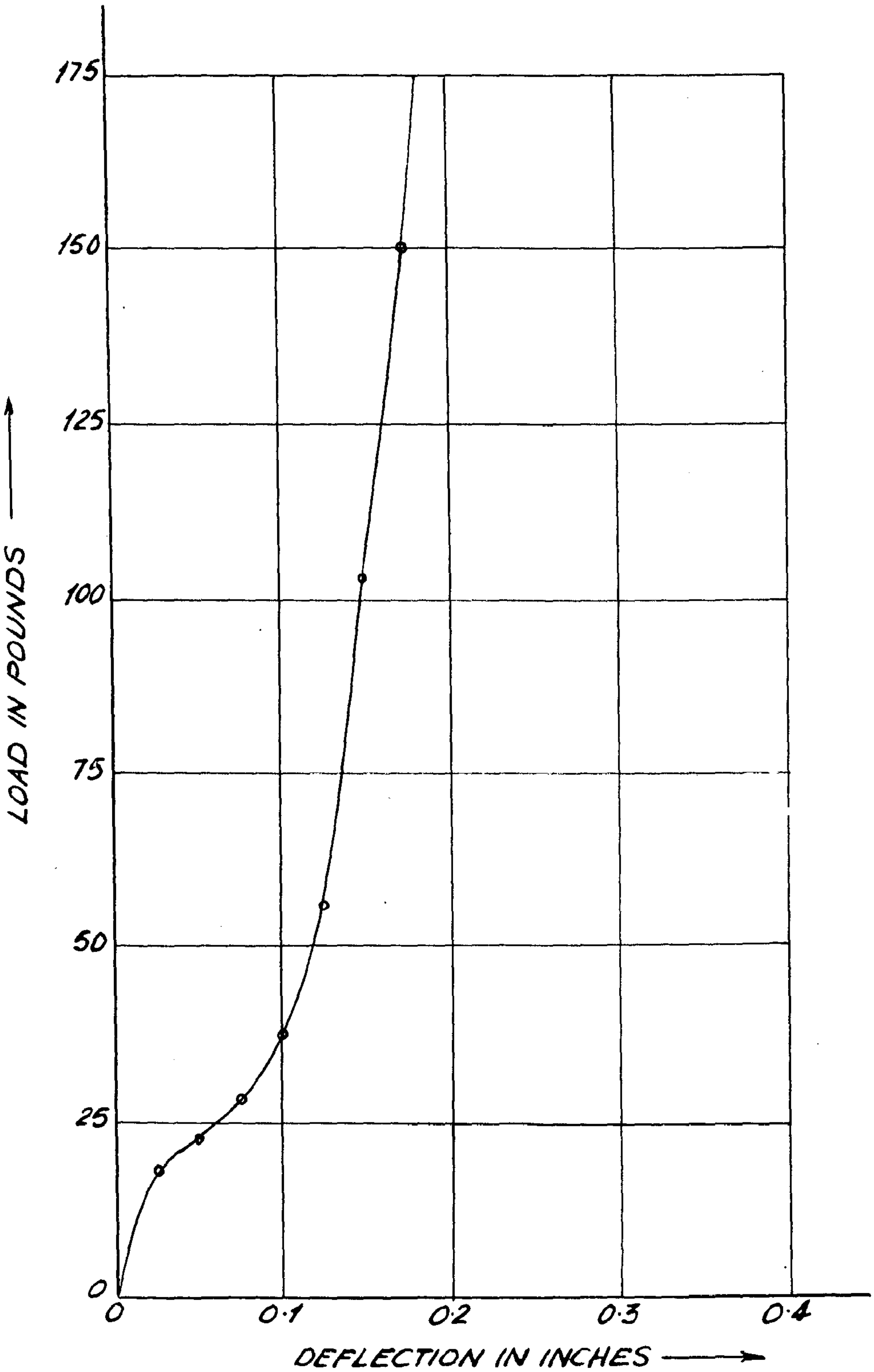
TUBE TEST FOR SOIL-I (C·B·R·I· SOIL)

FIG. 16



TUBE TEST FOR SOIL-II (BLUE SOIL)

FIG. 17



TUBE TEST FOR SOIL-III (BLENDED SOIL)

FIG. 18

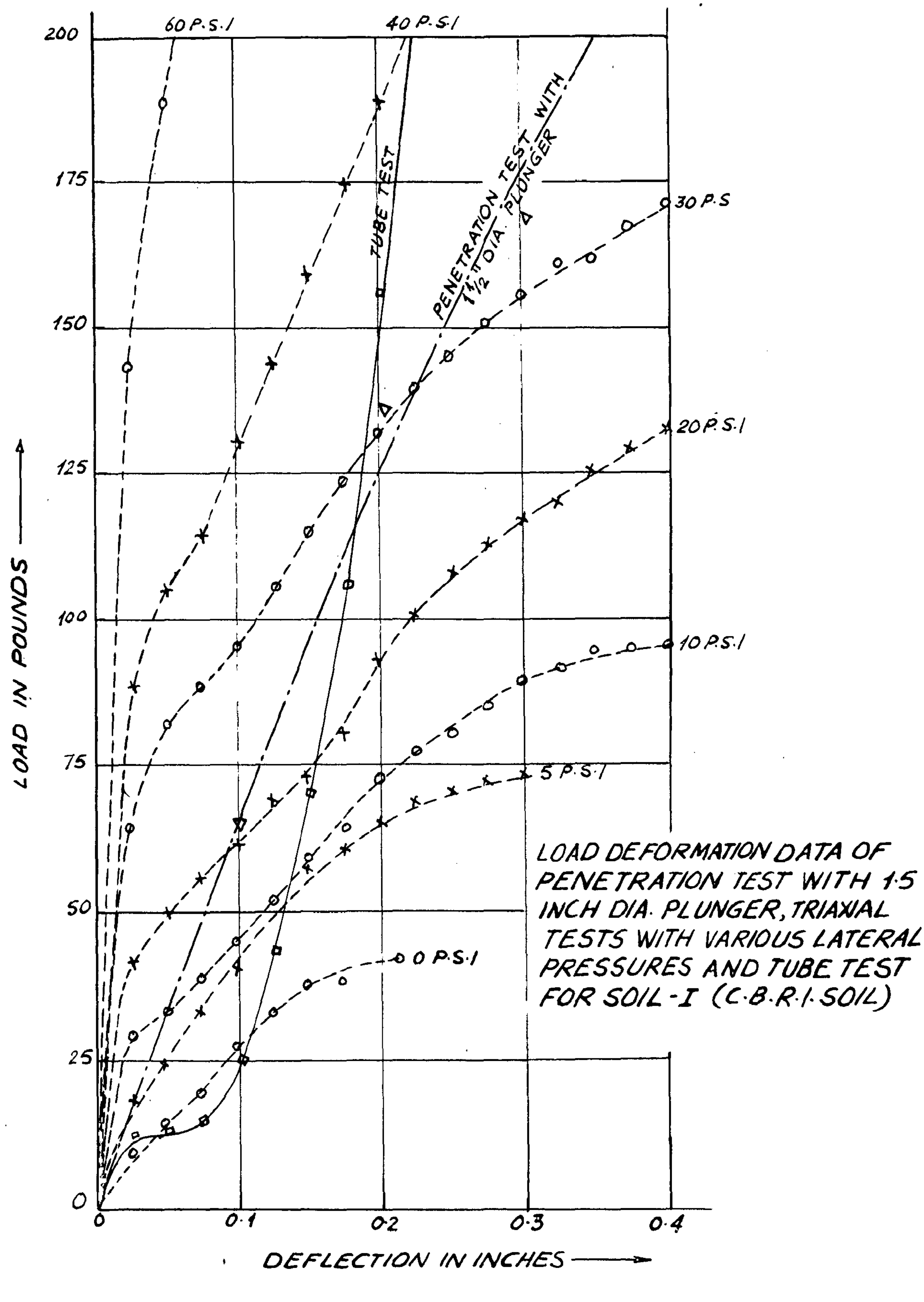
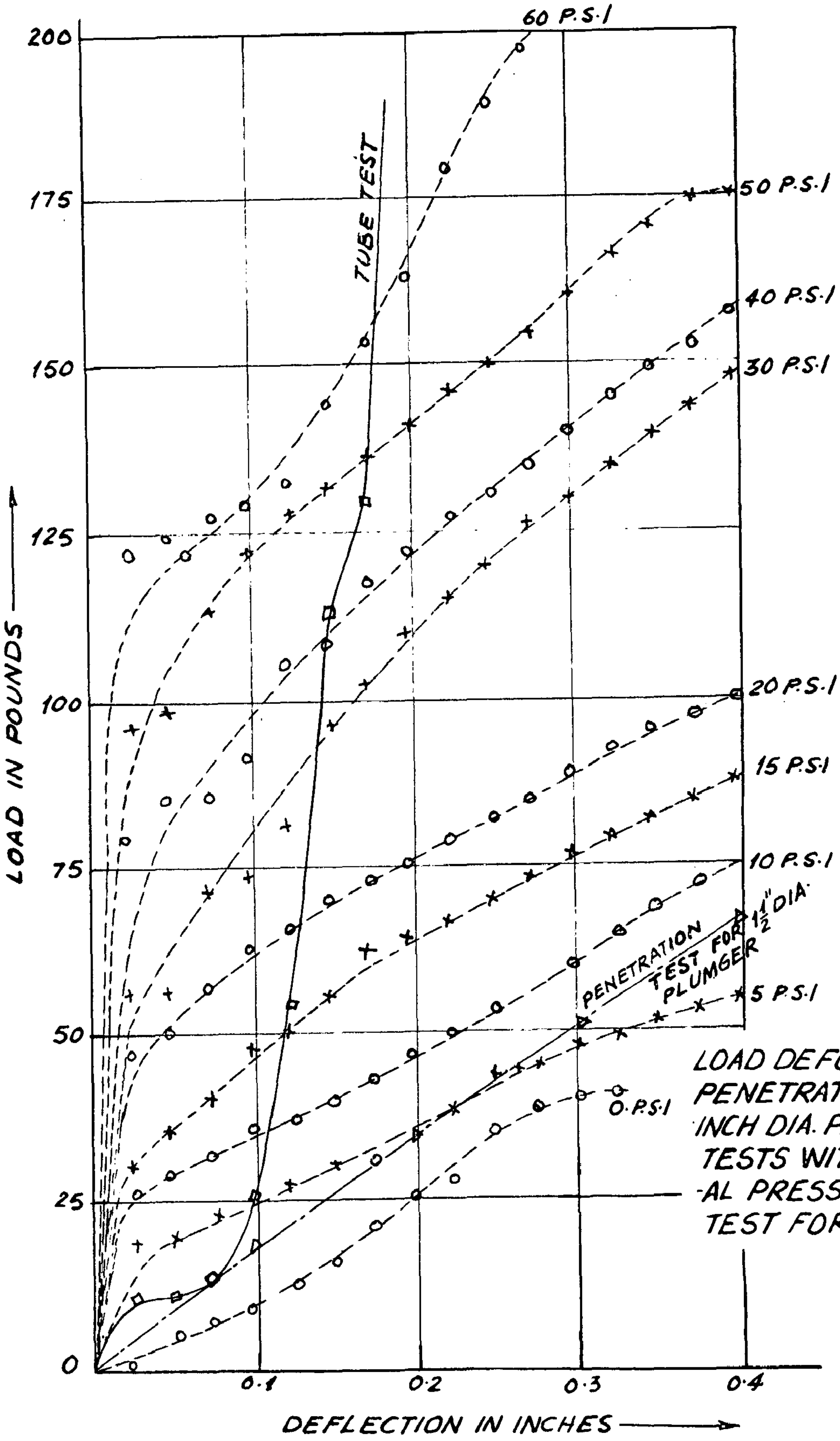
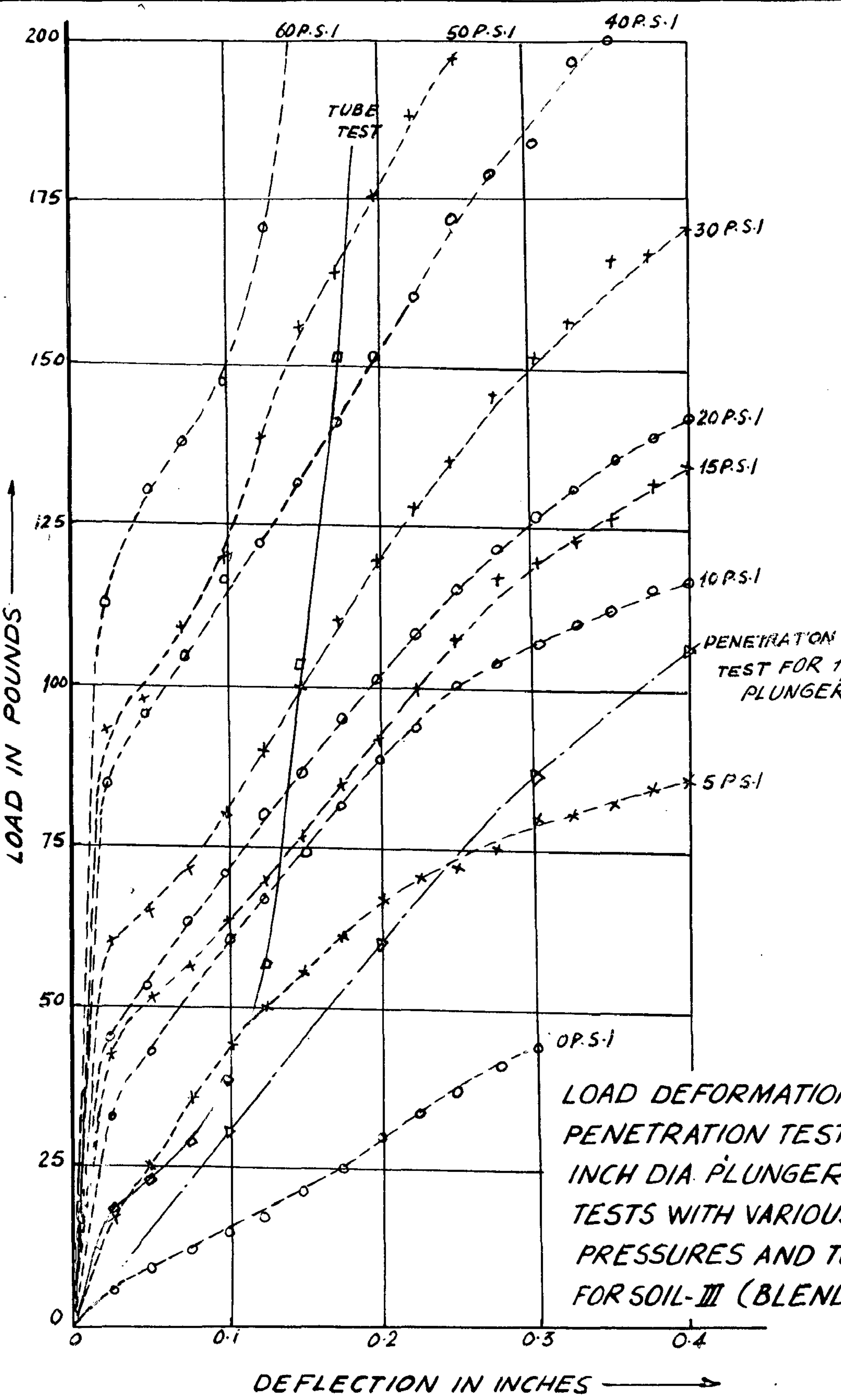

60 P.S.I
40 P.S.I
TUBE TEST
PENETRATION TEST WITH 1½" DIA. PLUNGER
30 P.S
20 P.S.I
10 P.S.I
5 P.S.I
0 P.S.I
LOAD IN POUNDS
DEFLECTION IN INCHES
200
175
150
125
100
75
50
25
0
0
0·1
0·2
0·3
0·4
LOAD DEFORMATION DATA OF PENETRATION TEST WITH 1·5 INCH DIA. PLUNGER, TRIAXIAL TESTS WITH VARIOUS LATERAL PRESSURES AND TUBE TEST FOR SOIL - I (C·B·R·I· SOIL)

FIG. 19



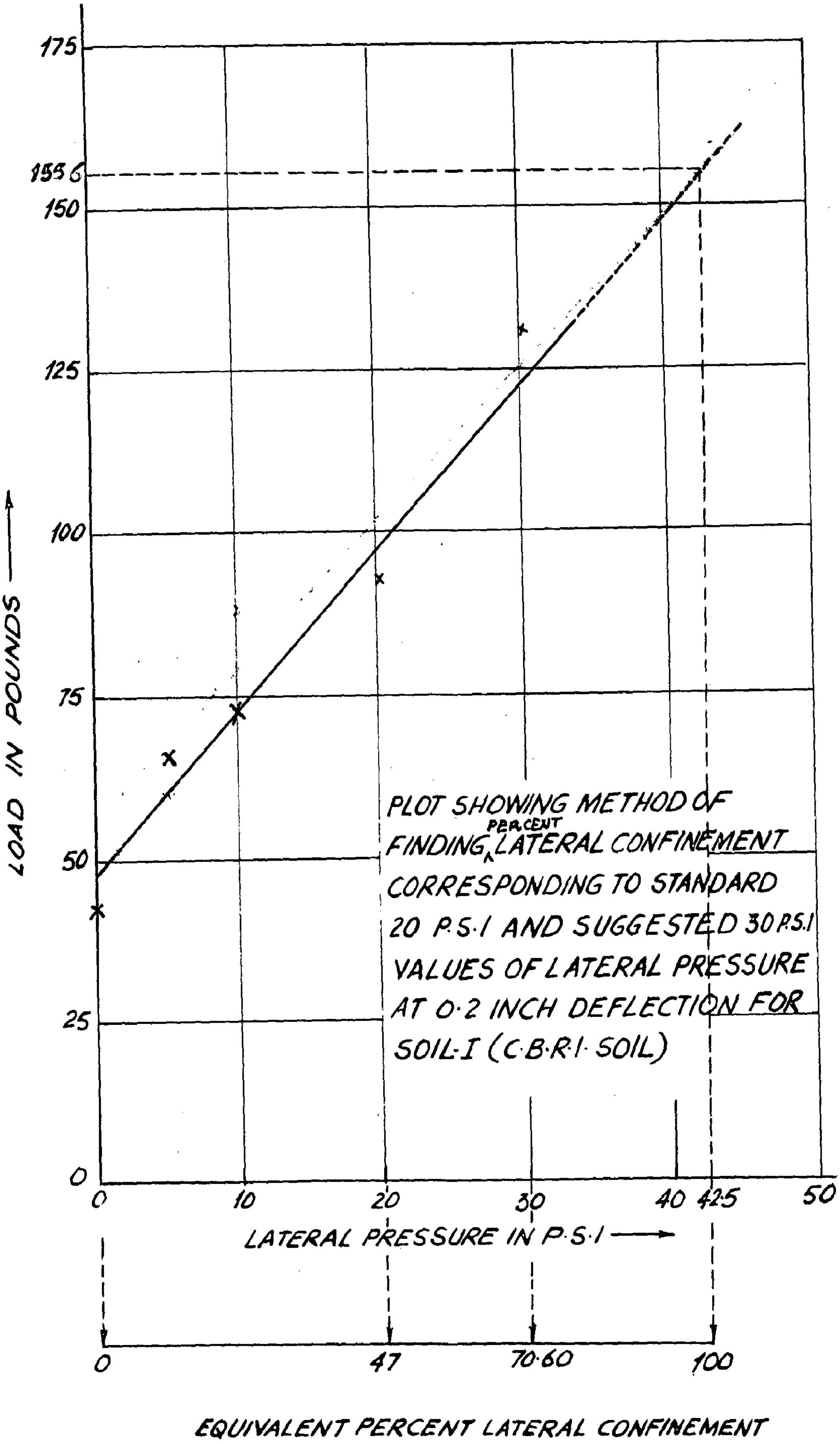
LOAD DEFORMATION DATA OF PENETRATION TEST WITH 1·5 INCH DIA. PLUNGER, TRIAXIAL TESTS WITH VARIOUS LATERAL PRESSURES AND TUBE TEST FOR SOIL-II (BLUE SOIL)

FIG. 20



LOAD DEFORMATION DATA OF PENETRATION TEST WITH 1·5 INCH DIA. PLUNGER, TRIAXIAL TESTS WITH VARIOUS LATERAL PRESSURES AND TUBE TEST FOR SOIL-III (BLENDED SOIL)

FIG. 21



175
155.6
150
125
100
75
50
25
0
LOAD IN POUNDS
0
10
20
30
40
42.5
50
LATERAL PRESSURE IN P.S.I
PLOT SHOWING METHOD OF FINDING PERCENT LATERAL CONFINEMENT CORRESPONDING TO STANDARD 20 P.S.I AND SUGGESTED 30 P.S.I VALUES OF LATERAL PRESSURE AT 0.2 INCH DEFLECTION FOR SOIL-I (C.B.R.I. SOIL)
0
47
70.60
100
EQUIVALENT PERCENT LATERAL CONFINEMENT

FIG. 22

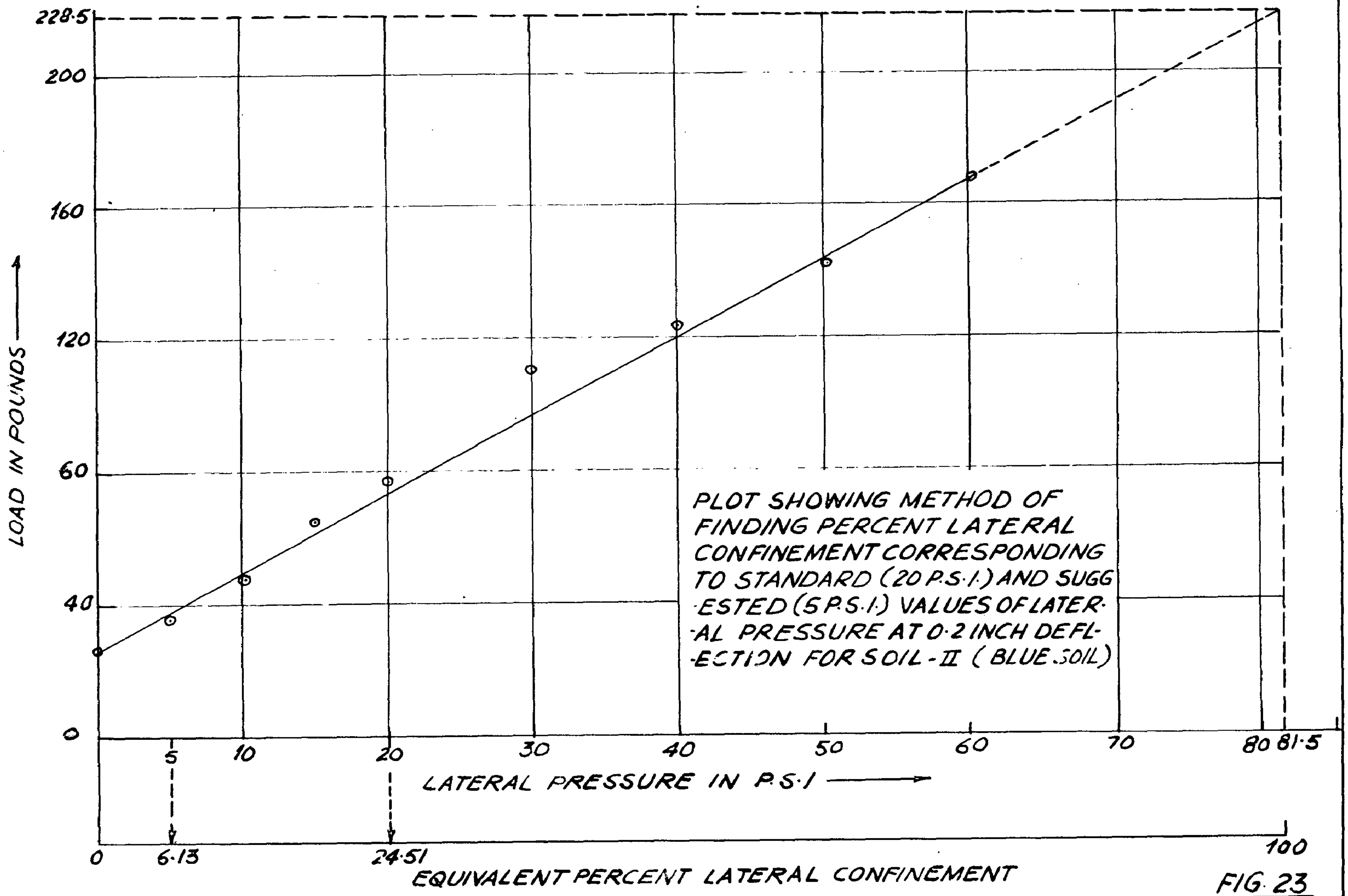

228·5
200
160
120
60
40
0
LOAD IN POUNDS
0
5
10
20
30
40
50
60
70
80
81·5
LATERAL PRESSURE IN P.S.I
0
6·13
24·51
100
EQUIVALENT PERCENT LATERAL CONFINEMENT
PLOT SHOWING METHOD OF FINDING PERCENT LATERAL CONFINEMENT CORRESPONDING TO STANDARD (20 P.S.I.) AND SUGG-ESTED (5 P.S.I.) VALUES OF LATER-AL PRESSURE AT 0·2 INCH DEFL-ECTION FOR SOIL - II (BLUE SOIL)

FIG. 23

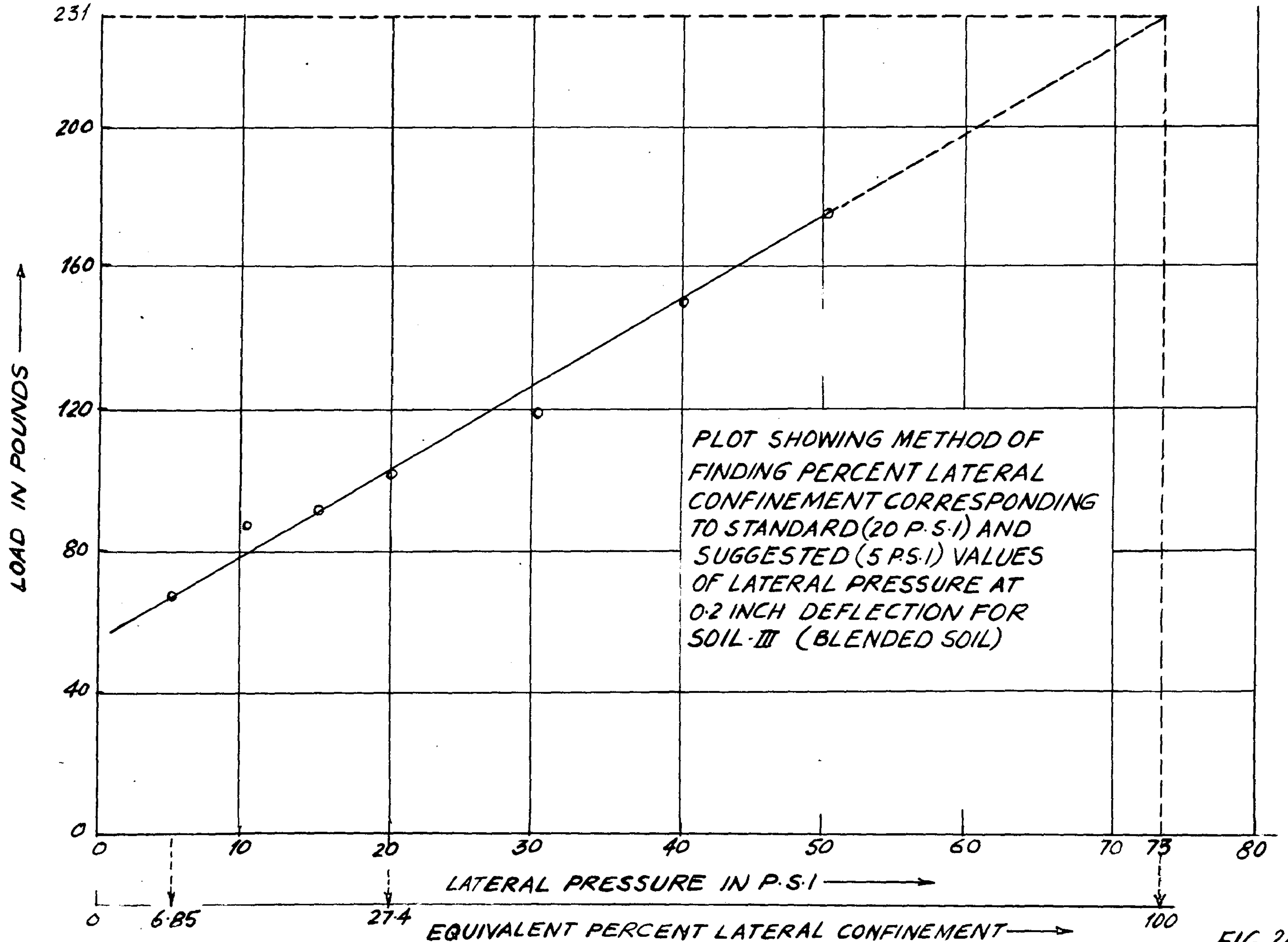

231
200
160
120
80
40
0
LOAD IN POUNDS
0
10
20
30
40
50
60
70
73
80
LATERAL PRESSURE IN P.S.I
0
6.85
27.4
100
EQUIVALENT PERCENT LATERAL CONFINEMENT
PLOT SHOWING METHOD OF FINDING PERCENT LATERAL CONFINEMENT CORRESPONDING TO STANDARD (20 P.S.I) AND SUGGESTED (5 P.S.I) VALUES OF LATERAL PRESSURE AT 0.2 INCH DEFLECTION FOR SOIL-III (BLENDED SOIL)

FIG. 24

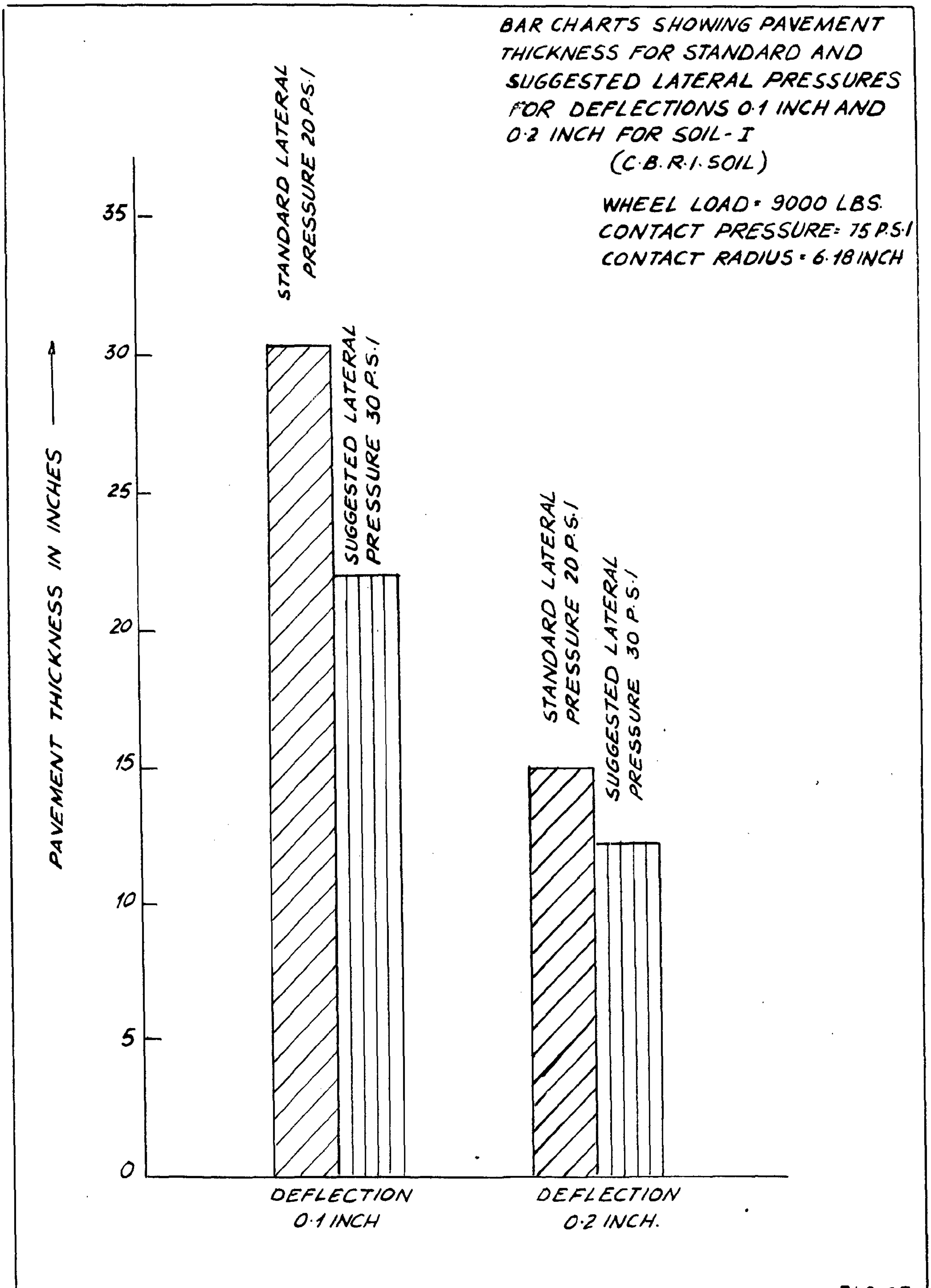

BAR CHARTS SHOWING PAVEMENT THICKNESS FOR STANDARD AND SUGGESTED LATERAL PRESSURES FOR DEFLECTIONS 0·1 INCH AND 0·2 INCH FOR SOIL - I
(C·B·R·I· SOIL)
WHEEL LOAD = 9000 LBS.
CONTACT PRESSURE = 75 P.S.I
CONTACT RADIUS = 6·18 INCH
PAVEMENT THICKNESS IN INCHES
0
5
10
15
20
25
30
35
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 30 P.S.I
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 30 P.S.I
DEFLECTION 0·1 INCH
DEFLECTION 0·2 INCH.

FIG. 25

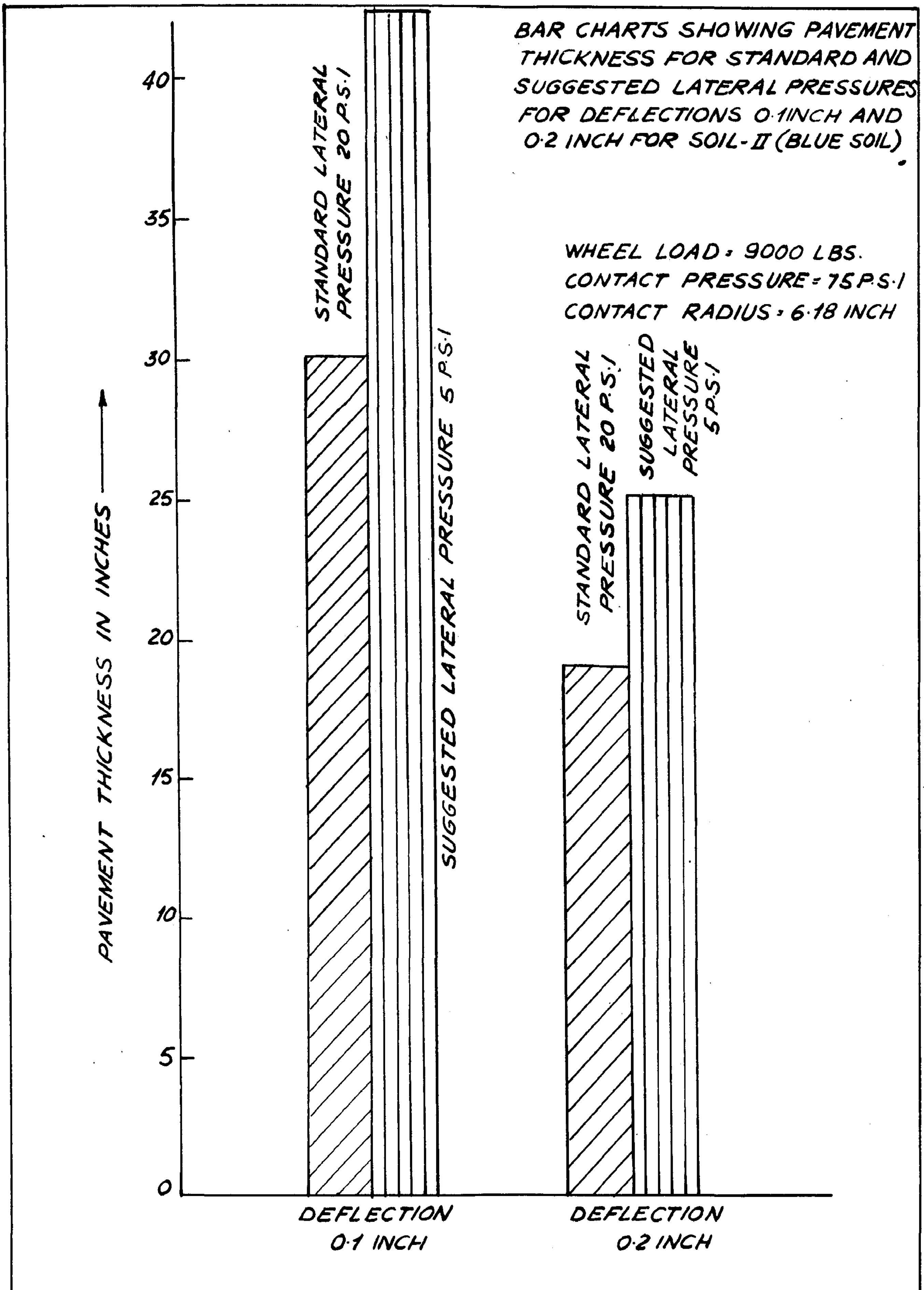

BAR CHARTS SHOWING PAVEMENT THICKNESS FOR STANDARD AND SUGGESTED LATERAL PRESSURES FOR DEFLECTIONS 0·1INCH AND 0·2 INCH FOR SOIL-II (BLUE SOIL)
WHEEL LOAD = 9000 LBS.
CONTACT PRESSURE = 75 P.S.I
CONTACT RADIUS = 6·18 INCH
PAVEMENT THICKNESS IN INCHES
0
5
10
15
20
25
30
35
40
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 5 P.S.I
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 5 P.S.I
DEFLECTION 0·1 INCH
DEFLECTION 0·2 INCH

FIG. 26

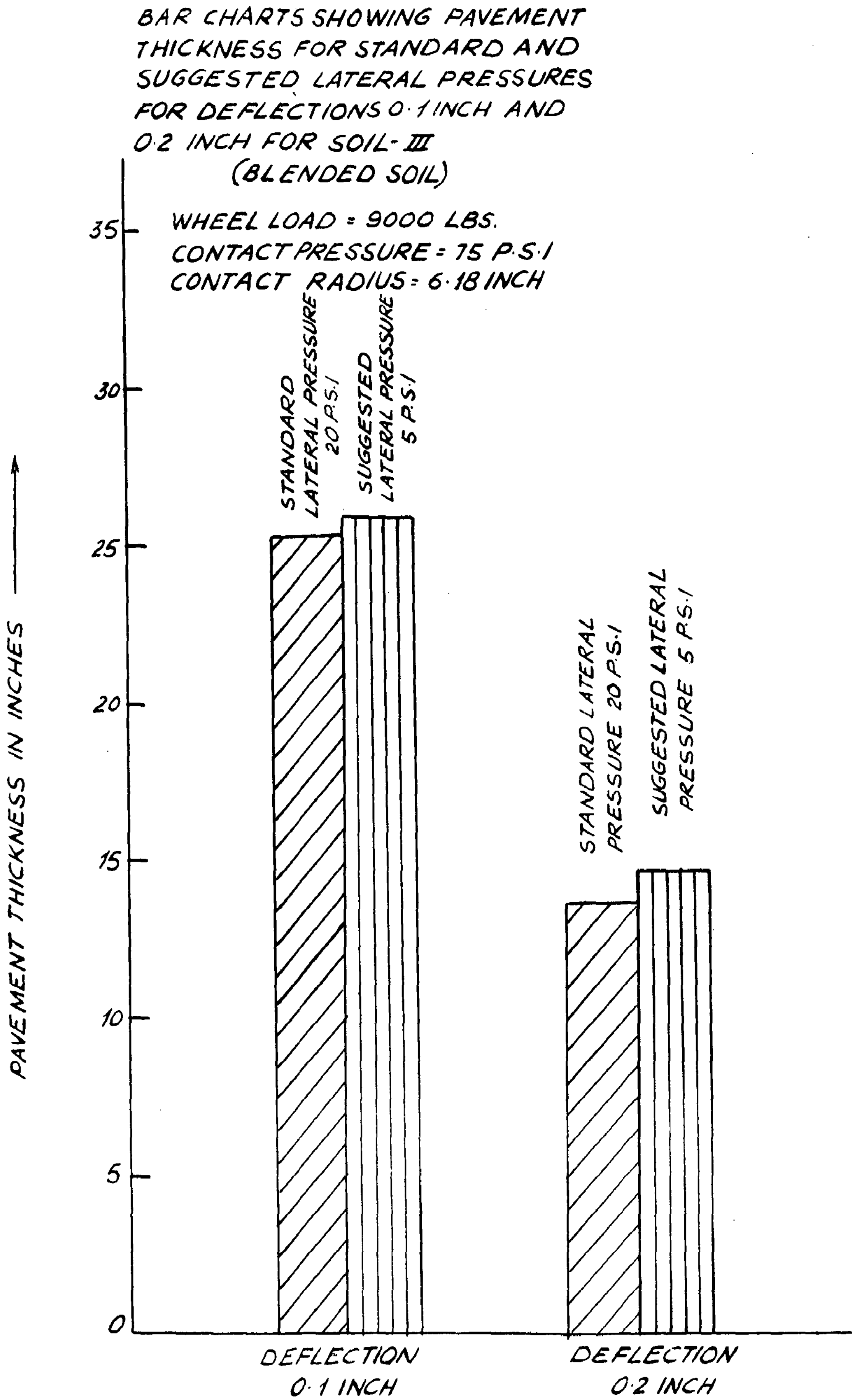
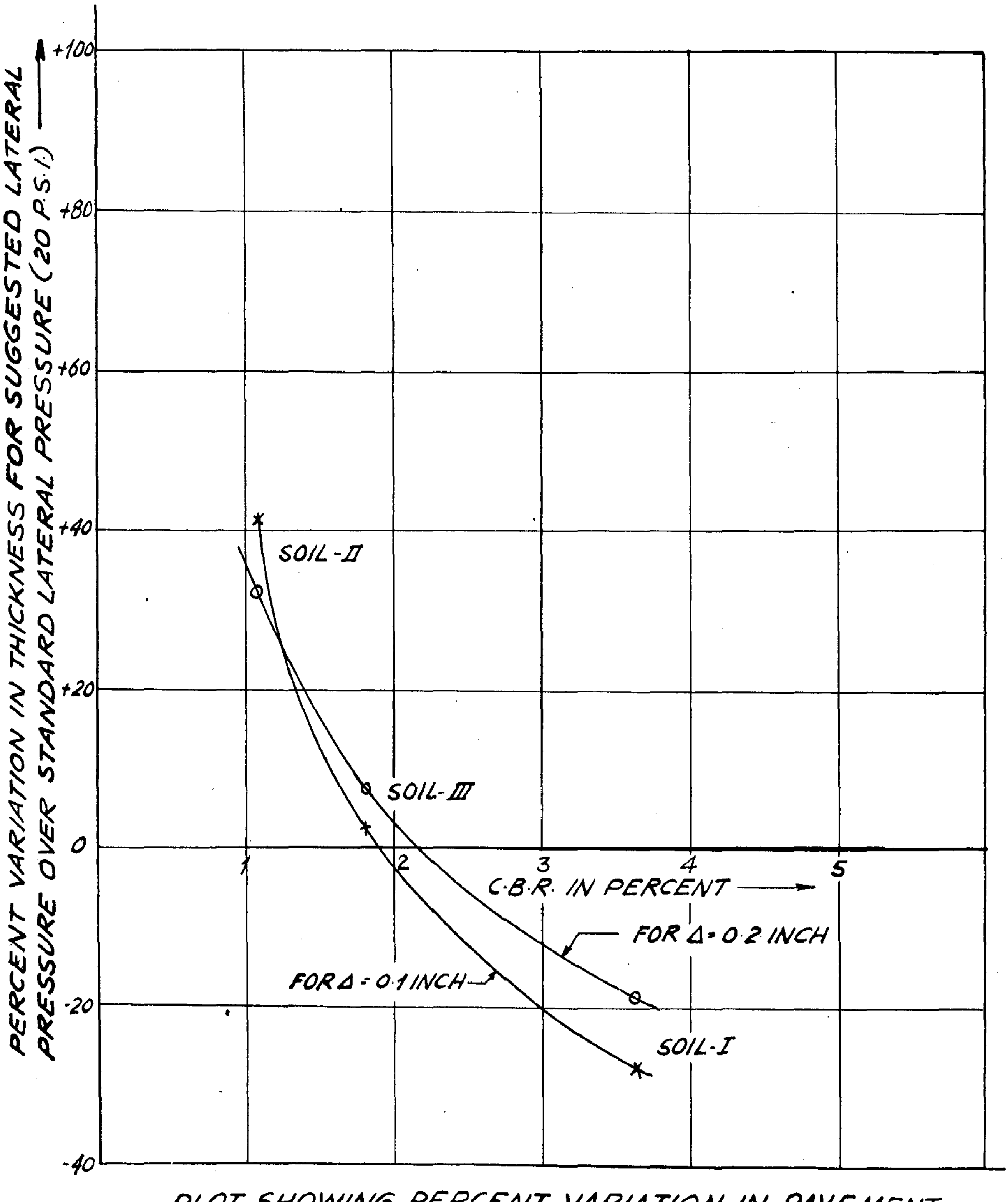
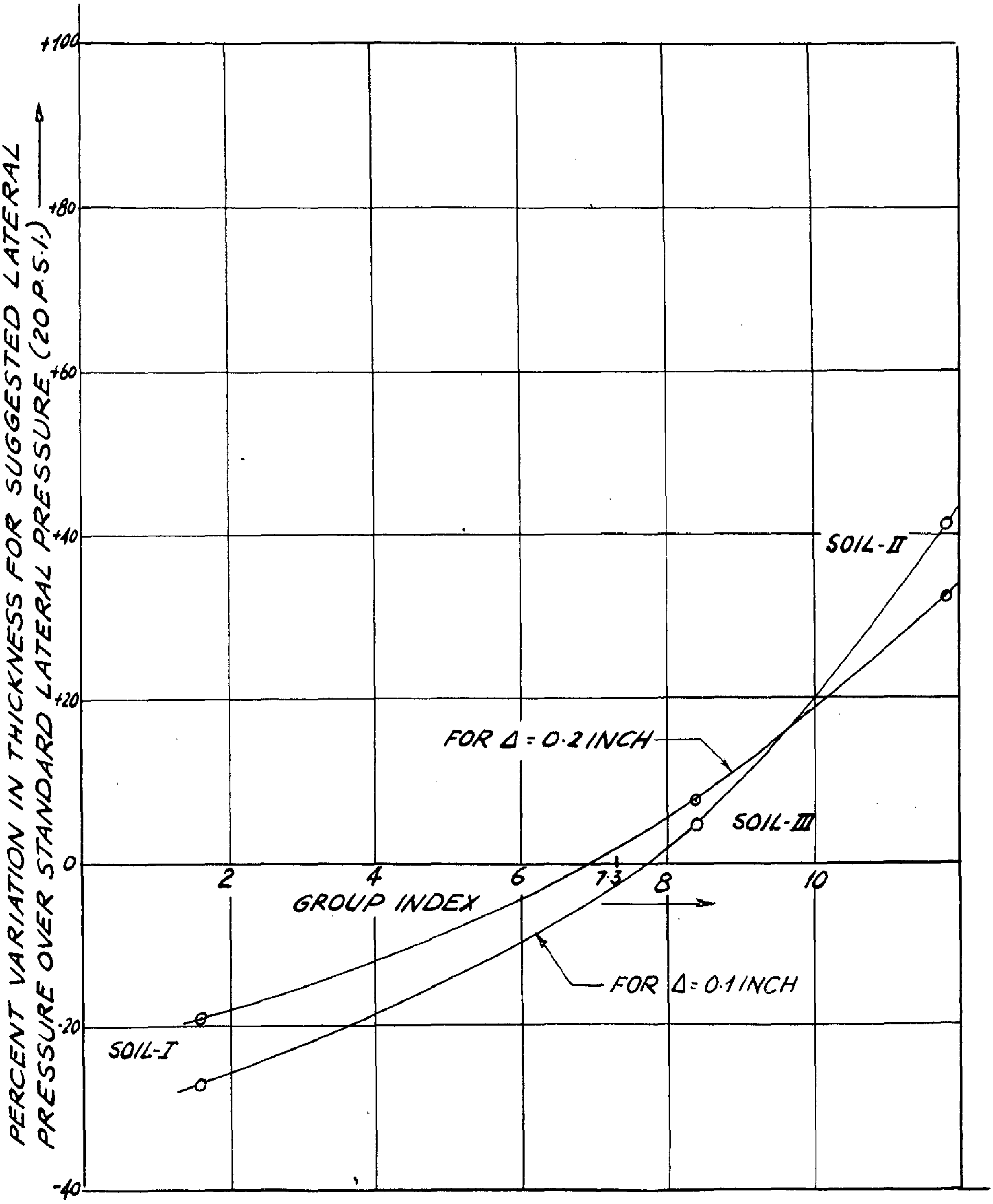

BAR CHARTS SHOWING PAVEMENT THICKNESS FOR STANDARD AND SUGGESTED LATERAL PRESSURES FOR DEFLECTIONS 0·1 INCH AND 0·2 INCH FOR SOIL-III
(BLENDED SOIL)
WHEEL LOAD = 9000 LBS.
CONTACT PRESSURE = 75 P.S.I
CONTACT RADIUS = 6·18 INCH
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 5 P.S.I
STANDARD LATERAL PRESSURE 20 P.S.I
SUGGESTED LATERAL PRESSURE 5 P.S.I
PAVEMENT THICKNESS IN INCHES
0
5
10
15
20
25
30
35
DEFLECTION 0·1 INCH
DEFLECTION 0·2 INCH

FIG. 27



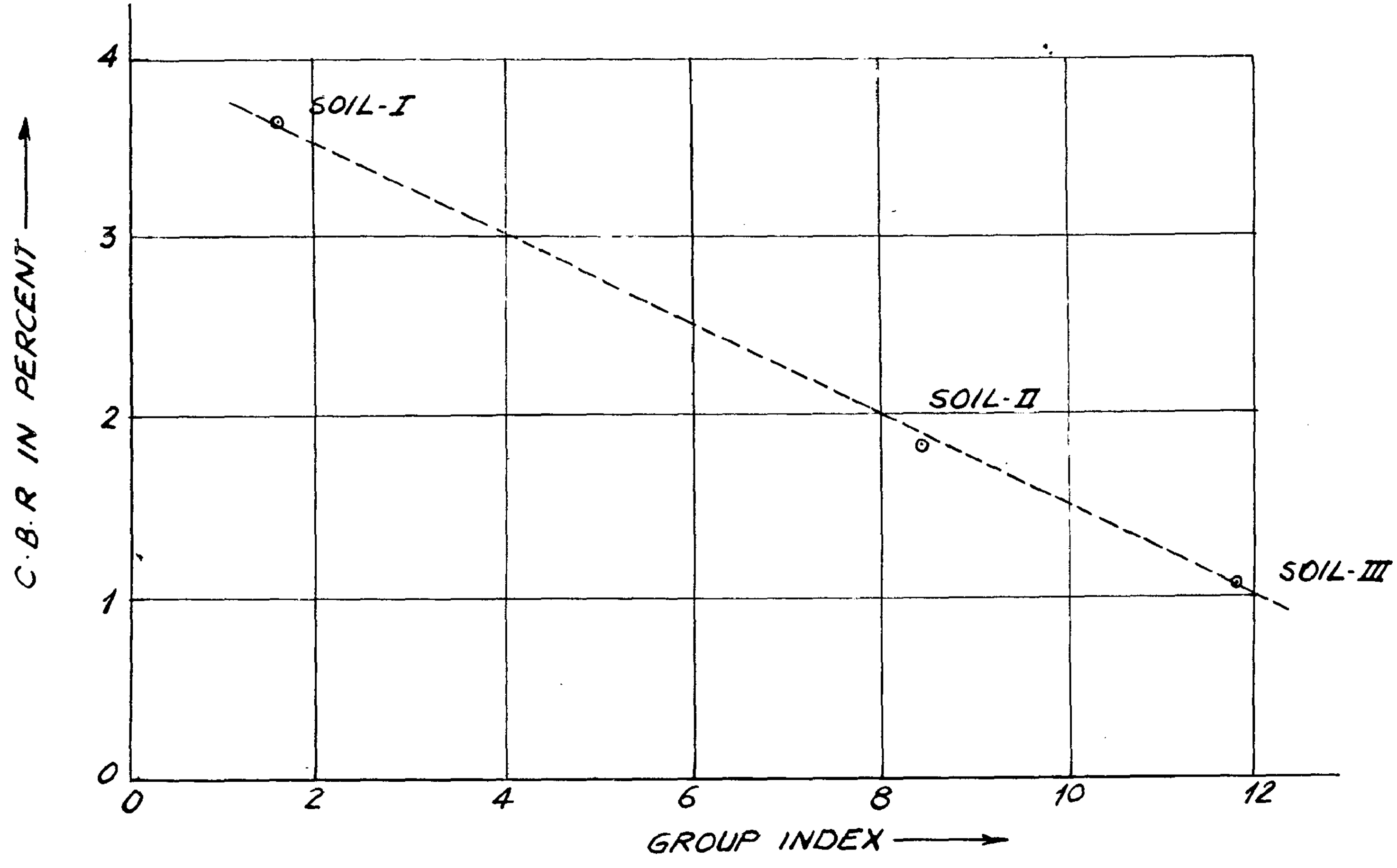
PLOT SHOWING PERCENT VARIATION IN PAVEMENT THICKNESS FOR SUGGESTED LATERAL PRESSURE OVER THE THICKNESS FOR STANDARD LATERAL PRESSURE 20 P.S.I VS C.B.R VALUE OF THE SUBGRADE.

FIG. 28



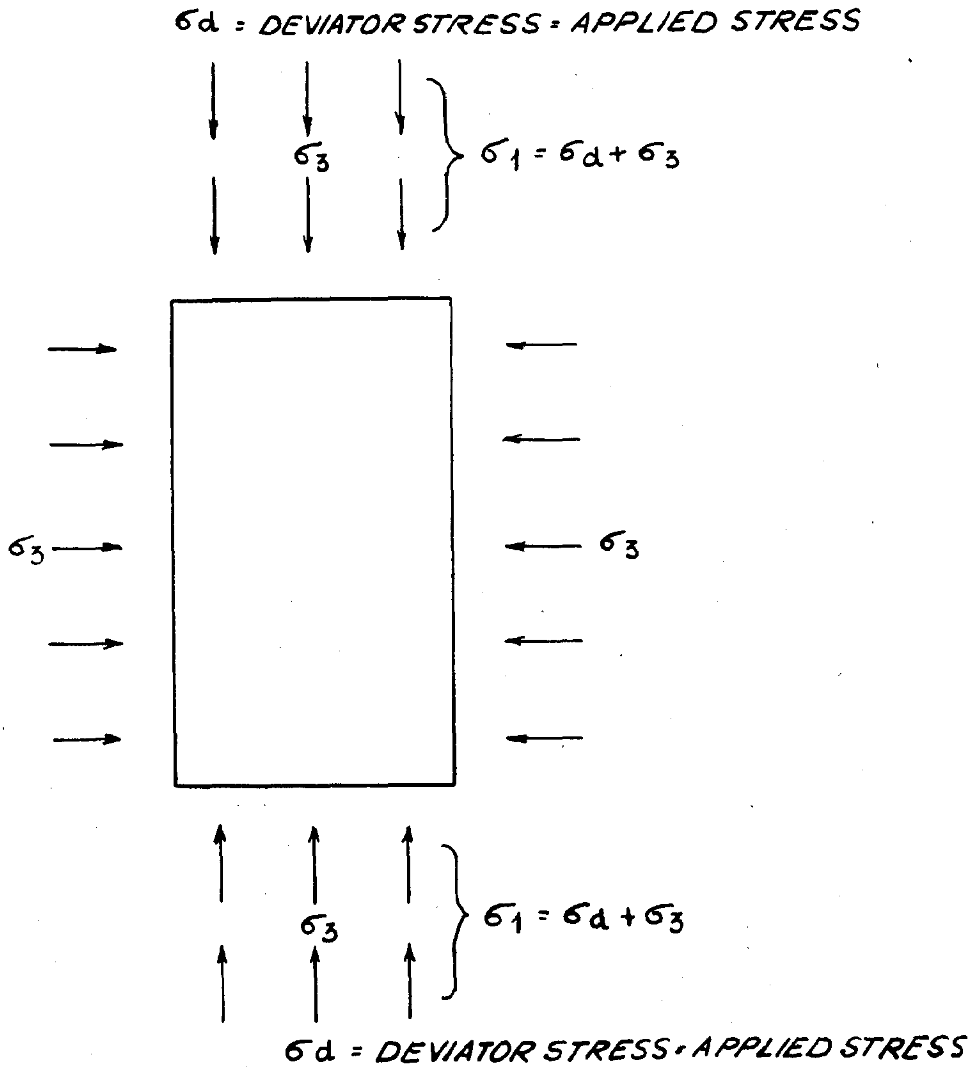
PLOT SHOWING PERCENT VARIATION IN PAVEMENT THICKNESS FOR SUGGESTED LATERAL PRESSURE OVER THE THICKNESS FOR STANDARD LATERAL PRESSURE 20 P.S.I VS GROUP INDEX VALUE OF THE SUBGRADE

FIG.29



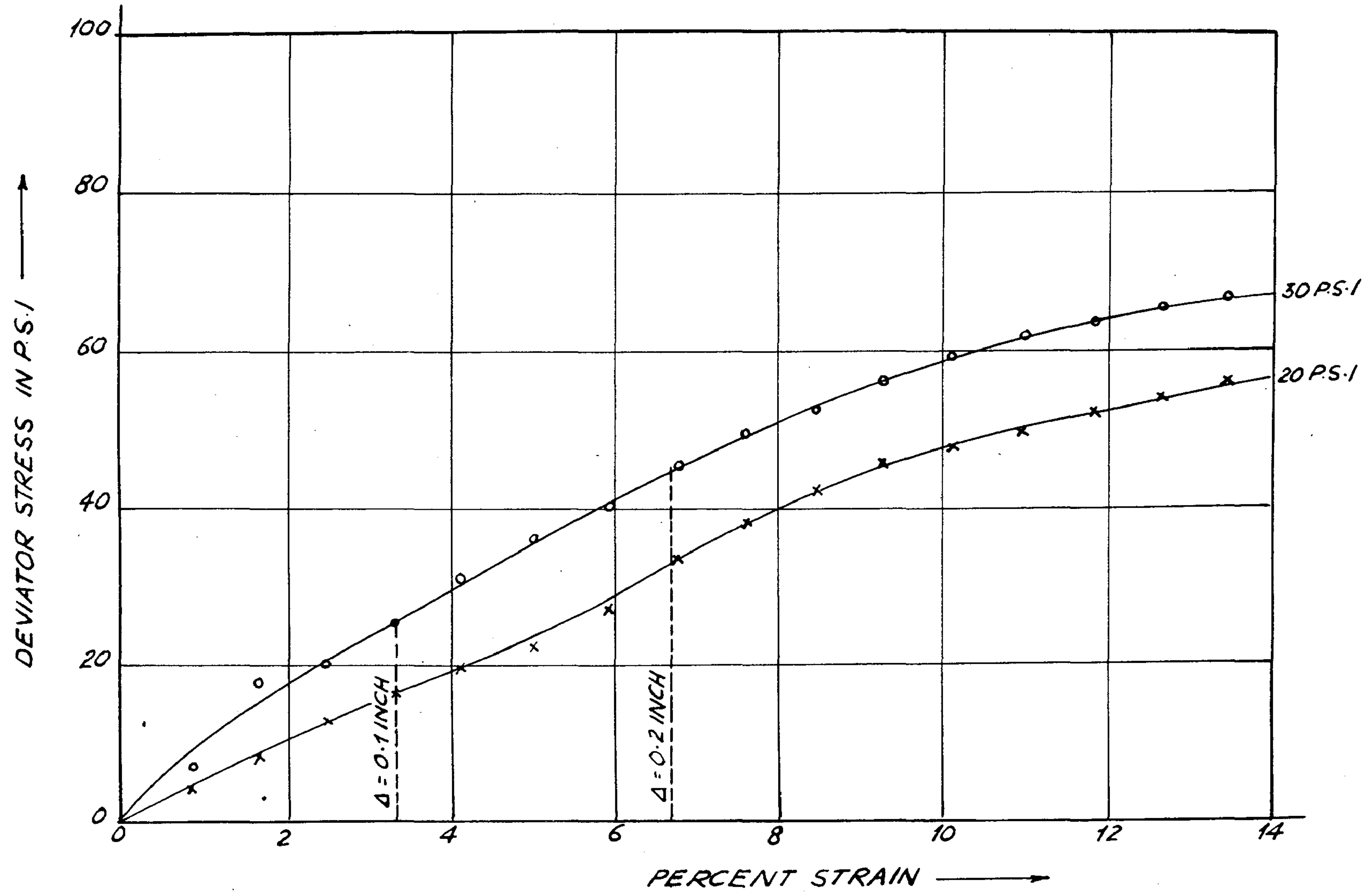
C·B·R· VS GROUP INDEX

FIG. 30



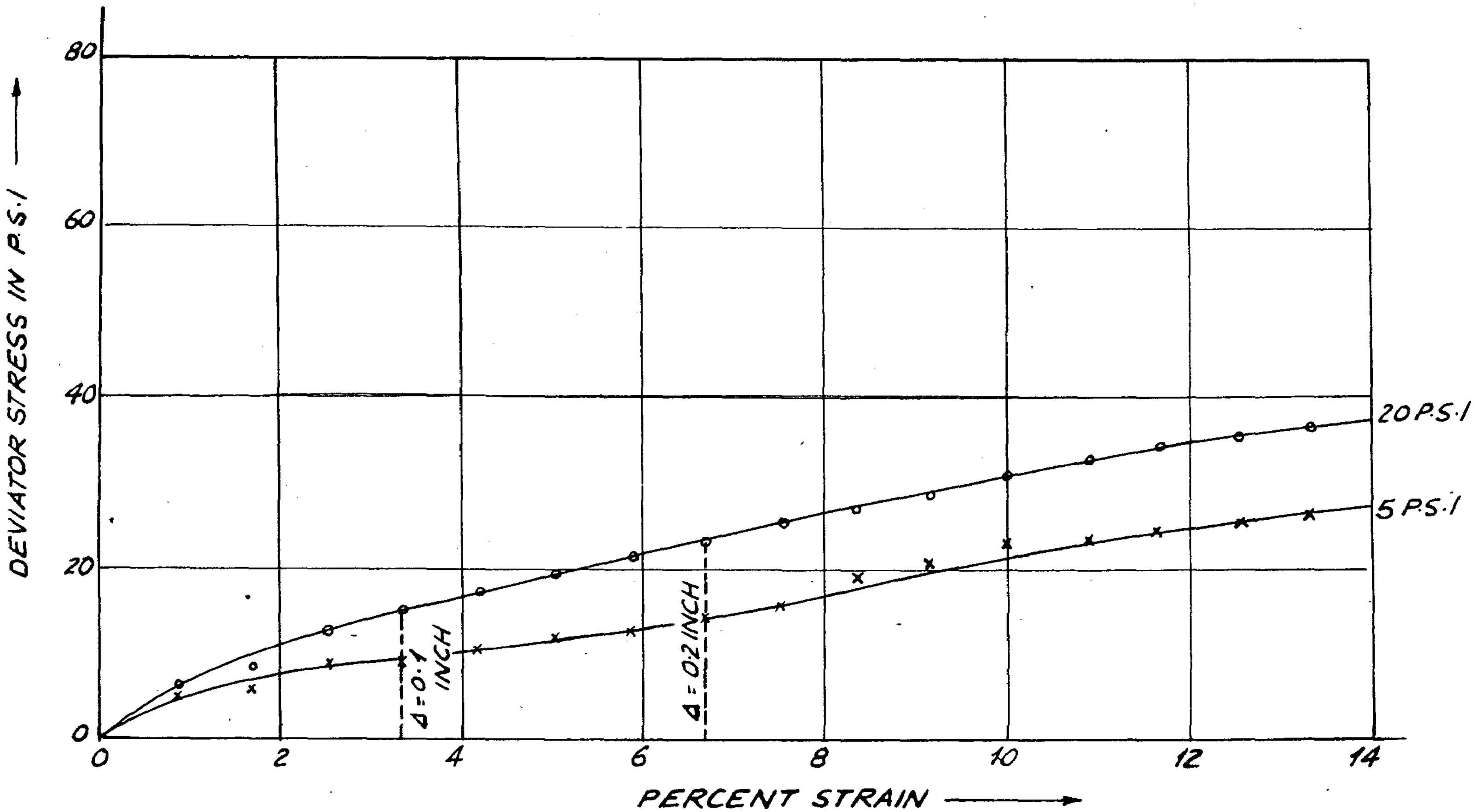
APPLIED STRESSES IN TRIAXIAL TEST

FIG. 31



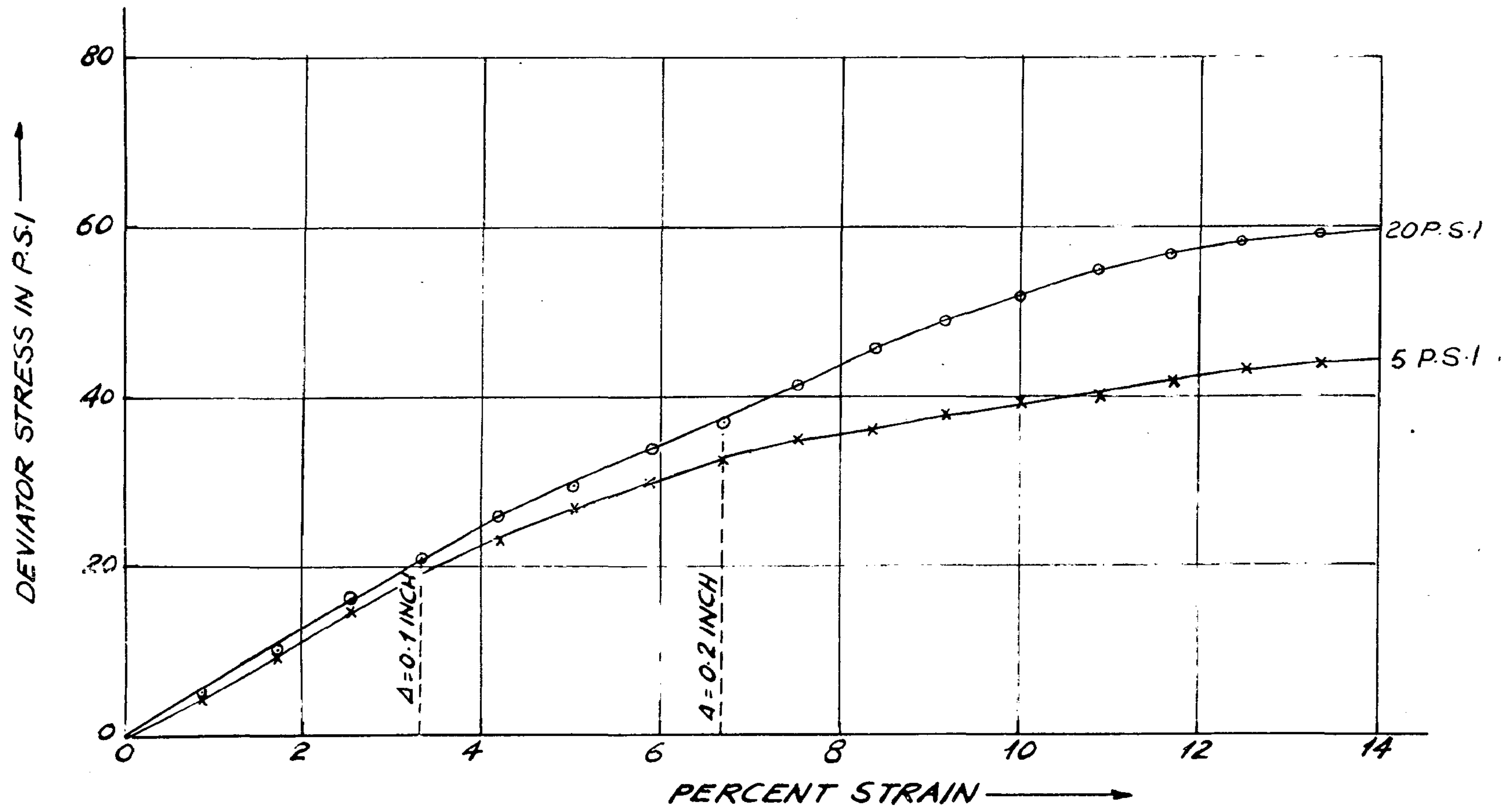
STRESS STRAIN PLOTS FROM TRIAXIAL TESTS FOR STANDARD 20 P.S.I AND SUGGESTED 30 P.S.I VALUES OF LATERAL PRESSURE FOR SOIL-I [C.B.R.I. SOIL]

FIG. 32



STRESS- STRAIN PLOT FROM TRIAXIAL TESTS FOR STANDARD 20 P.S.I. AND SUGGESTED 5 P.S.I. VALUES OF LATERAL PRESSURE FOR SOIL-II [BLUE SOIL]

FIG. 33



STRESS-STRAIN PLOTS FROM TRIAXIAL TESTS FOR STANDARD [20 P.S.I] AND SUGGESTED [5 P.S.I] VALUES OF LATERAL PRESSURE FOR SOIL-III (BLENDED SOIL)

FIG. 34

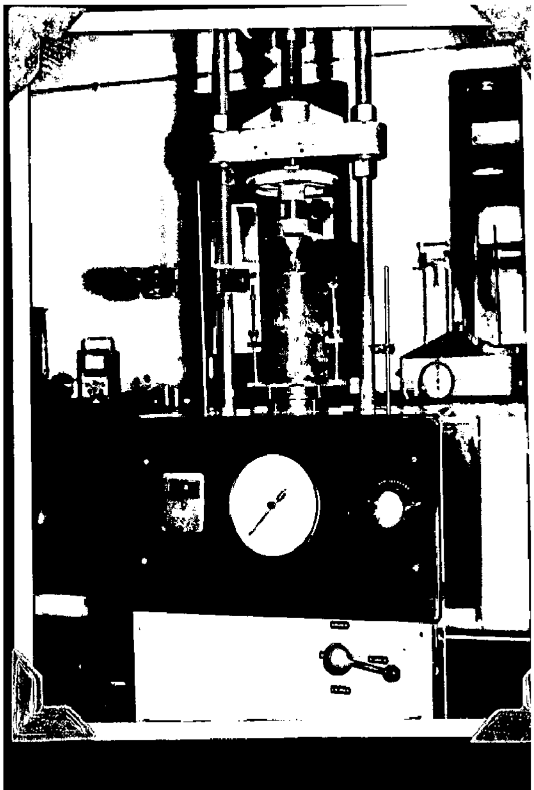


PHOTO.1. COMPACTION

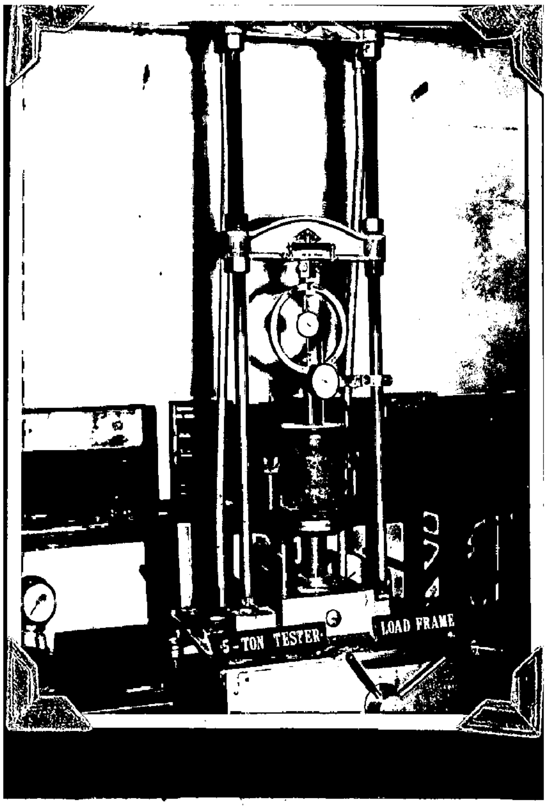


PHOTO.2. PENETRATION TEST.

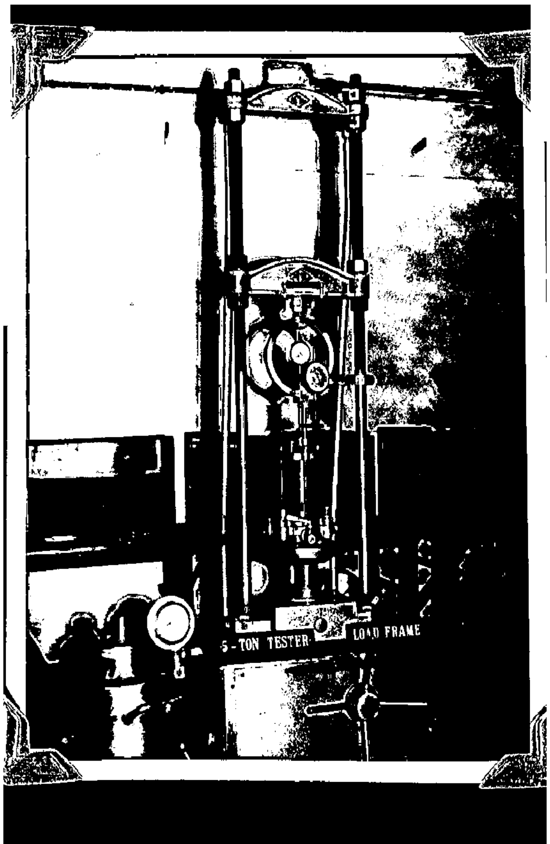


PHOTO.3. TRIAXIAL TEST.

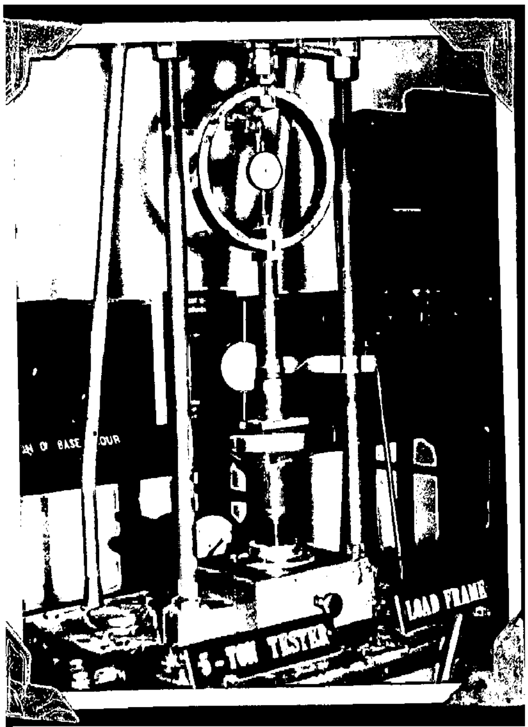


PHOTO.4. TUBE TEST

APPENDICES

APPENDIX - I

CALCULATION OF GROUP INDEX FOR THE THREE SOILS

The Group Index of a soil is obtained from the following equation : (11)

$$GI = 0.2a + 0.005\ ac + 0.01\ bd$$

where a = that portion of the percentage passing ASTM No. 200 Sieve greater than 35 percent and not exceeding 75 percent expressed as a positive whole number (0 to 40).

b = that portion of the percentage passing ASTM No. 200 sieve greater than 15 percent and not exceeding 55 percent, expressed as a positive whole number (0 to 40).

c = that portion of the numerical liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (0 to 20).

d = that portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20)

Data from Table 1 are used in the following calculations.

Group Index of Soil I (CBRI Soil).

$a = 42.7 - 35 = 7.7 = 8$ (Say)

$b = 42.7 - 15 = 27.7 = 28$ (Say)

$c = 0$

$d = 10 - 10 = 0$

Group Index $= 0.2a + 0.005\ ac + 0.01\ bd$

$= 0.2 \times 8 + 0.005 \times 8 \times 0 + 0.01 \times 28 \times 0$

$= 1.6 + 0 + 0$

$= 1.6$

Group Index of Soil II (Blue Soil).

$a = 75 - 35 = 40$

$b = 55 - 15 = 40$

$c = 51 - 40 = 11$

$d = 14 - 10 = 4$

Group Index $= 0.2a + 0.005\ ac + 0.01\ bd$

$= 0.4 \times 40 + 0.005 \times 40 \times 11 + 0.01 \times 40 \times 4$

$= 8 + 2.2 + 1.66$

$= 11.8$

Group Index of Soil III (Blended Soil)

$a = 75 - 35 = 40$

$b = 55 - 15 = 40$

$c = 0$

$d = 11 - 10 = 1$

Group Index $= 0.2a + 0.005\ ac + 0.01\ bd$

$= 0.2 \times 40 + 0.005 \times 40 \times 0 + 0.01 \times 40 \times 1$

$= 8 + 0.4$

$= 8.4$

APPENDIX II

CALCULATIONS FOR WEIGHTS OF DRY SOIL AND WATER FOR AASHO COMPACTION CONDITIONS FOR THE THREE SOILS

Volume of compacted sample in the CBR mould	$= \frac{\pi \times 6^2}{4} \times \frac{5}{12^3}$
	$= 0.08$ cft
Soil I (CBRI Soil)	
Optimum Moisture Content	= 10 percent
Maximum Dry Density	= 110 pcf
Weight of dry soil in compacted sample	$= 110 \times 0.08$ lbs
	= 8.80 lbs
	$= 8.80 \times 453.59$ gms
	= 3990 gms
Weight of water required	$= 3990 \times \frac{10}{100}$
	= 399 gms
Soil II (Blue Soil)	
Optimum Moisture Content	= 18 percent
Maximum Dry Density	= 96 pcft
Weight of dry soil in compacted sample	$= 96 \times 0.08$ lbs
	= 7.69 lbs
	$= 7.69 \times 453.59$ gms
	= 3490 gms
Weight of Water required	$= 3490 \times 18/100$
	= 628 gms.

Soil III (Blended Soil)

Optimum Moisture content	= 14 percent
Maximum Dry Density	= 102 pcf
Weight of dry soil in compacted sample	= 102 x 0.08 lbs
	= 8.16 lbs
	= 8.16 x 453.59 gms
	= 3700 gms
Weight of water required	= $3700 \times \frac{14}{100}$
	= 518 gms

-:0:-

APPENDIX - III

PART A

METHOD OF CALCULATION OF MODULUS OF ELASTICITY E OF SOIL

An undrained triaxial test is used. With the lateral fluid pressure (confining stress $= \sigma_3$) maintained constant, the axial load (deviator stress $= \sigma_d$) is gradually increased at the specified rate till failure results. A plot with deviator stress against percent strain is developed.

In the conventional test it is necessary to add the confining pressure to the deviator pressure to determine the total unit load at failure, as illustrated in Figure 31. The vertical stress σ_1 can be determined by means of equations (1) and (2).

$$\sigma_d = \frac{P(1 - \partial)}{A} \quad \ldots\ldots\ldots\ldots \quad (1)$$

where σ_d = deviator stress

P = applied deviator load (lbs or kgs)

A = original cross sectional area of specimen

∂ = Unit strain

$$\sigma_1 = \sigma_d + \sigma_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where σ_1 = total vertical stress

σ_3 = confining stress.

For the design of flexible pavements, use is made of the modulus of deformation. The modulus is calculated for the straight portion of the stress-strain curve. For most cases, however, the stress-strain curve of the soil will not be straight for any appreciable distance but rather will be curved.

The modulus of deformation for this case is calculated at some arbitrary point, generally at a stress value equal to the stress condition which will exist in the pavement. For example if the vertical stress under the pavement is, say, 10 psi, the modulus of deformation is equal to 10 psi divided by its corresponding deformation value from the triaxial test (8).

The stress-strain data for the three soils used in the present investigation for standard value and suggested value of lateral pressures as presented in Tables 18, 19 and 20. The corresponding stress-strain plots are shown in Figures 32, 33, and 34. For each value of lateral pressure E value is calculated at deflection levels 0.1 inch and 0.2 inch, assuming the soil to be elastic over this range. The calculations for E for the three soils are presented below, in Part - B.

PART - B.

CALCULATIONS FOR THE MODULUS OF ELASTICITY FOR THE THREE SOILS AT DEFLECTIONS OF 0.1 INCH AND 0.2 INCH FOR THE STANDARD VALUE OF LATERAL PRESSURE OF 20 PSI AND FOR THE SUGGESTED VALUES OF LATERAL PRESSURES.

Soil Designation	Soil I (CBRI Soil)	Soil II (Blue Soil)	Soil III (Blended Soil)
Standard value of Lateral Pressure in psi	20	20	20
Suggested value of Lateral Pressure in psi	30	5	5

For Deflection = 0.1 inch

Effective area of cross section of soil sample $= \dfrac{\pi \times 0.75^2 \times 3}{2.9}$

Strain $= \dfrac{0.1}{3}$

$= 0.033$

For Deflection = 0.2 inch

Effective area of cross-section of soil sample $= \dfrac{\pi \times 0.75^2 \times 3}{2.8}$

$= 1.893 \text{ in}^2$

Strain $= 0.2/3$

$= 0.066$

Load values used in the following calculations are taken from Tables 8, 9 and 10.

E of Soil I (CBRI Soil)

Value of Lateral Pressure	For $\Delta = 0.1$ inch	For $\Delta = 0.2$ inch
	Load = 62.9 lbs	Load = 93.1 lbs
	Stress = $\frac{62.9}{1.828}$	Stress = $\frac{93.1}{1.893}$
	= 34.4 psi	= 49.15 psi
20 psi (Standard value)	$E = \frac{(34.4-20)}{0.033} = \frac{14.4}{0.033}$	$E = \frac{(49.15-20)}{0.066} = \frac{29.15}{0.066}$
	= 432 psi	= 437.5 psi
	Load = 95.3 lbs	Load = 131.5 lbs
	Stress = $\frac{95.3}{1.828}$	Stress = $\frac{131.5}{1.893}$
30 psi (Suggested value)	= 52.15 psi	= 69.5 psi
	$E = \frac{(52.15-30)}{0.033} = \frac{22.15}{0.033}$	$E = \frac{(69.5-30)}{0.066} = \frac{39.5}{0.066}$
	= 665 psi	= 592.5 psi

E of Soil II (Blue Soil)

Value of Lateral pressure	For $\Delta = 0.1$ inch	For $\Delta = 0.2$ inch
	Load = 63.1 lbs	Load = 76.9 lbs
	Stress = $\frac{63.1}{1.828}$	Stress = $\frac{76.9}{1.893}$
20 psi (Standard value)	= 34.5 psi	= 40.65 psi
	$E = \frac{(34.5-20)}{0.033} = \frac{14.5}{0.033}$	$E = \frac{(40.65-20)}{0.066} = \frac{20.65}{0.066}$
	= 435 psi	= 310 psi

Contd.........L107

	For $\Delta = 0.1$ inch	For $\Delta = 0.2$ inch
	Load $= 25.00$ lbs	Load $= 35.1$ lbs
	Stress $= \frac{25.00}{1.828} =$	Stress $= \frac{35.1}{1.893}$
5 psi (Suggested value)	$= 13.68$ psi	$= 18.55$ psi
	$B = \frac{(13.68-5)}{0.033} = \frac{8.68}{0.033}$	$B = \frac{(18.55-5)}{0.066} = \frac{13.55}{0.066}$
	$= 261$ psi	$= 203.5$ psi

B of Soil III (Blended Soil)

Value of Lateral Pressure	For $\Delta = 0.1$ inch	For $\Delta = 0.2$ inch
	Load $= 71$ lbs	Load $= 100.9$ lbs
	Stress $= \frac{71}{1.828}$	Stress $= \frac{100.9}{1.893}$
	$= 38.85$ psi	$= 53.3$ psi
20 psi (Standard value)	$B = \frac{(38.85-20)}{0.033}$	$B = \frac{(53.3-20)}{0.066}$
	$= \frac{18.85}{0.033}$	$= \frac{33.3}{0.066}$
	$= 566$ psi	$= 500$ psi

	Load $= 43$ lbs	Load $= 67$ lbs
	Stress $= \frac{43}{1.828}$	Stress $= \frac{67}{1.893}$
5 psi (Suggested value)	$= 23.5$ psi	$= 35.4$ psi
	$B = \frac{23.5-5}{0.033}$	$B = \frac{35.4-5}{0.066}$
	$= \frac{18.5}{0.033}$	$= \frac{30.4}{0.066}$
	$= 555$ psi	$= 457$ psi

APPENDIX - IV

PART A

CALCULATION OF DESIGN THICKNESS OF FLEXIBLE PAVEMENT USING KANSAS DESIGN METHOD

Modulus of elasticity (E) of the subgrade materail is determined with the help of a triaxial test. This value is then substituted in the Boussinesq's equation to obtain the thickness of pavement required. To arrive at this equation for the pavement thickness, Boussinesq's settlement equation for deflection at the centre of a flexible plate is used, which is as follows (12)

$$\Delta = \frac{3\,pa^2}{2E\,(a^2+Z^2)^{1/2}} \qquad \ldots\ldots\ldots\ldots\ldots(3)$$

where Δ = deflection at the centre of flexible plate.

p = Intensity of load on the circular

a = radius of the plate

Z = depth of the point where stresses are calculated.

E = modulus of elasticity of the material of subgrade.

Thus, $a^2 + Z^2 = [\frac{3}{E} \quad \frac{pa^2}{E\,\Delta}]$

or $$Z = \left[\left(\frac{3pa^2}{2E\,\Delta} \right)^2 - a^2 \right]^{1/2} \qquad \ldots\ldots\ldots(4)$$

But total pressure P on the plate = $p\pi a^2$

Therefore, $Z = [(3P/2\pi E\,\Delta)^2 - a^2]^{1/2}$ ....(5)

Assuming that the pavement is incompressible, the depth Z becomes the thickness T of the pavement, and hence we get, (8),

$$T = [(3P/2\pi E\Delta)^2 - a^2]^{1/2} \quad \ldots\ldots\ldots\ldots(6)$$

and this equation may be used for the design of flexible pavements.

In equation (6) the thickness is given in terms of the total load, the modulus of elasticity of the subgrade soil, the radius of the contact area and the deflection. When the load is applied by dual wheels, the effective radius of tire contact area for the combination is not easily determined. To solve this the stresses imposed by each tire are computed separately. The modulus of deformation of the subgrade required to limit the total deflection of the surface to that permitted may then be determined. Thus the theory of the formula is used by computing the effect of each tire separately without the necessity of finding the indeterminate value of the effective contact area (7).

To utilize equation (6) for design purposes it is necessary to assume a limiting deflection. The design deflection of the surface is assumed to be 0.1 inch for flexible pavements. This value was determined from measurements of numerous flexible pavements in various conditions (7,8).

Coefficients :

The design procedure calls for all samples to be tested in a saturated condition. Saturation is deemed desirable in order to obtain a direct comparison for all types of materials, and, at the same time, to obtain a measure of the strength where the material is in its weakest condition. To account for the possibility of the soil not being saturated after construction, a coefficient, n, is introduced which gives a decreased thickness of surface or base course required in localities of less rainfall. The recommended values of n are as shown in table below (7.8).

Saturation coefficient n	Average Annual Rainfall in inches.
0.5	15.0 - 19.9
0.6	20.0 - 24.9
0.7	25.0 - 29.9
0.8	30.0 - 34.9
0.9	35.0 - 39.9
1.0	40.0 - 50.0

If the maximum wheel load is 9000 pounds and the percentage of vehicle carrying maximum loads are related to the other h lighter vehicles is fairly constant, the variation affecting design occurs mostly in the total volume of traffic. Coefficients have been determined according to the volume of traffic. The coefficients and their corresponding ranges of traffic are shown in Table below (9.12).

Traffic Coefficient m	Total traffic vehicles per day.
1/2	50-400
2/3	401-800
5/6	801-1200
1	1201-1800
7/6	1801-2700
8/6	2701-4000
9/6	4001-6000
10/6	6001-9000
11/6	9001-13500
12/6	13501-20000

Introducing the coefficients into equation (6) and using the modification of the formula proposed by Palmer and Barber, equation (7) results ;

$$T = \left[\sqrt{(3Pmn/2\pi E\Delta)^2 - a^2}\right]\left[\sqrt{E/Ep}\right] \qquad \ldots\ldots\ldots(7)$$

where T = thickness required

Ep = modulus of elasticity of pavement or surface course.

E = modulus of elasticity of subgrade or sub base.

P = base wheel load

m = traffic coefficient based on traffic volume

n = saturation coefficient based on rainfall.

a = radius of area of tire contact
corresponding to p

Δ = permitted deflection of surface.

The stiffness factor $(E/Ep)^{1/3}$ was proposed on the basis of the stiffness factor for slabs and was checked against the elastic displacement due to a point load on a two layer system. The stiffness factor takes into account the stress distributing qualities of the paving surface as related to those of the subgrade soil. Disregarding the stiffness factor equation (7) gives the thickness of an incompressible layer which will yield a settlement equal to that of the assumed condition and for the design wheel load. With the stiffness factor equation (6) gives the thickness of a bituminous mat placed directly upon the subgrade soil for the assigned loading conditions. If one wishes to substitute some grannular base course materail for a portion of the bituminous mat, the relationship can be determined on the basis of the stiffness factor for the bituminous mat related to the base course. Thus if a thickness of final wearing surface is assumed, the thickness of base course material to add to the pavement structure is that given in the following equation (8).

$$t_B = (T - t_p) \sqrt[3]{Ep/E_B} \quad \ldots\ldots\ldots(8)$$

where t_B = thickness of base course

T = thickness of bituminous mat

t_p = Assumed thickness of wearing surface.

E_p = Modulus of elasticity of wearing surface.

E_B = Modulus of Elasticity of base course.

It is to be noted that the procedure necessitates assuming a thickness of wearing surface.

The calculations for designed thickness of pavement are presented in Part - B on next page.

PART - B

CALCULATIONS FOR DESIGN THICKNESSES OF FLEXIBLE PAVEMENT FOR THE THREE SOILS FOR THE STANDARD VALUE AND SUGGESTED VALUE OF LATERAL PRESSURES AND CORRESPONDING VALUES OF MODULUS OF ELASTICITY AT DEFLECTIONS OF 0.1 INCH AND 0.2 INCH.

The design formula is

$$T = \left[\sqrt{\left(3Pmn/2\pi E\Delta\right)^2 - a^2}\ \right]\left[\sqrt[3]{E/E_p}\ \right]$$

the notations being as explained earlier in Part A.

The value of P is taken as 9000 lbs as I.R.C. Specifications(13)

The value of tire pressure for highways generally varies between 60 psi and 90 psi (8). A value of 75 psi is adopted for calculations.

The value of m is taken as 1, assuming volume of traffic as 1201 to 1800 vehicles per day (8).

The value of n is taken as 1, assuming average rainfall as 40 to 50 inches (8).

a = 6.18 inches (For P = 9000 lbs and tire pressure 75 psi)

Δ = 0.1 inch and 0.2 inch.

E_p is taken as 15000 psi (8).

Now $3P/2\pi = \dfrac{3 \times 9000}{2\pi} = 4295$

The values of E and deflections used in the following calculations are taken from Table 17.

Pavement Thickness for Soil I (CDRI Soil)

Value of Lateral Pressure: 20 psi (Standard value)

For $\Delta = 0.1$ inch

$$T = \frac{\left[\sqrt{\left(\frac{4295}{432 \times 0.1}\right)^2 - 6.18^2}\,\right]}{\sqrt[3]{\left(\frac{432}{15000}\right)}}$$

$$= \sqrt{(99.5^2 - 6.18^2)} \times \left(\frac{7.55}{24.66}\right)$$

$$= 10.27 \times 9.67 \times 0.306$$

$$= 30.35 \text{ inches}$$

For $\Delta = 0.2$ inch

$$T = \frac{\left[\sqrt{\left(\frac{4295}{437.5 \times 0.2}\right)^2 - 6.18^2}\,\right]}{\sqrt[3]{\frac{437.5}{15000}}}$$

$$= \sqrt{(49^2 - 6.18^2)}\left(\frac{7.6}{24.66}\right)$$

$$= 7.43 \times 6.55 \times 0.308$$

$$= 15 \text{ inches}$$

Value of Lateral Pressure: 30 psi (Suggested value)

For $\Delta = 0.1$ inch

$$T = \frac{\left[\sqrt{\left(\frac{4295}{665 \times 0.1}\right)^2 - 6.18^2}\right.}{\sqrt[3]{\frac{665}{15000}}}$$

$$= \left[\sqrt{64.6^2 - 6.18^2}\,\right]\left[\frac{8.73}{24.66}\right]$$

$$= 8.42 \times 7.65 \times 0.34$$

$$= 21.90 \text{ inches}$$

For $\Delta = 0.2$ inch

$$T = \frac{\left[\sqrt{\left(\frac{4295}{592.5 \times 0.2}\right)^2 - 6.18^2}\,\right]}{\sqrt[3]{\frac{592.5}{15000}}}$$

$$= \left[\sqrt{36.22^2 - 6.18^2}\right.\left(\frac{8.4}{24.66}\right]$$

$$= 6.51 \times 5.48 \times 0.34$$

$$= 12.13 \text{ inches}$$

Pavement Thickness for Soil II (Blue Soil)

Value of Lateral Pressure	For $\Delta = 0.1$ inch	For $\Delta = 0.2$ inch
20 psi (Standard value)	$T = \left[\sqrt{\left(\frac{4295}{435 \times 0.1}\right)^2 - 6.18^2}\right] \sqrt[3]{\frac{435}{15000}}$ $= \sqrt{(98.7^2 - 6.18^2)}\left(\frac{7.575}{24.66}\right)$ $= 10.2 \times 9.62 \times 0.307$ = 30.15 inches	$T = \left[\sqrt{\left(\frac{4295}{310 \times 0.2}\right)^2 - 6.18^2}\right] \sqrt[3]{\frac{310}{15000}}$ $= \left[\sqrt{(69.25^2 - 6.18^2)}\right]\left(\frac{6.77}{24.66}\right)$ $= 8.60 \times 7.94 \times 0.2745$ = 19 inches
5 psi (suggested value)	$T = \left[\sqrt{\left(\frac{4295}{261 \times 0.1}\right)^2 - 6.18^2}\right]$ $= \sqrt[3]{\frac{203.5}{15000}}$ $= \left(\sqrt{(164.5^2 - 6.18^2}\left(\frac{6.39}{24.66}\right)\right)$ $= 1\ .05 \times 12.59 \times 0.259$ = 42.6 inches	$T = \sqrt{\left(\frac{4295}{203.5 \times 0.2}\right)^2 - 6.18^2}]$ $\sqrt[3]{\frac{203.5}{15000}}$ $= \sqrt{(105.5^2 - 6.18^2}\ \left(\frac{5.9}{24.6}\right)$ $= 10.55 \times 9.97 \times 0.2385$ = 25.10 inches

Pavement Thickness for Soil III (Blended Soil)

Value of Lateral Pressure	For Δ = 0.1 inch	For Δ = 0.2 inch
20 psi (Standard value)	$T = \dfrac{\left[\sqrt{\left(\dfrac{4295}{566 \times 0.1}\right)^2 - 6.18^2}\,\right]}{\sqrt[3]{\dfrac{566}{15000}}}$	$T = \dfrac{\left[\sqrt{\left(\dfrac{4295}{500 \times 0.2}\right)^2 - 6.18^2}\,\right]}{\sqrt[3]{\dfrac{500}{15000}}}$
	$= \sqrt{(76^2 - 6.18^2)}\left(\dfrac{8.27}{24.66}\right)$	$= \sqrt{(42.95^2 - 6.18^2}\left(\dfrac{7.93}{24.66}\right)$
	$= 9.07 \times 8.35 \times 0.335$	$= 7.01 \times 6.07 \times 0.3215$
	= 25.35 inches	= 13.67 inches
5 psi (Suggested Value)	$T = \dfrac{\left[\sqrt{\left(\dfrac{4295}{555 \times 0.1}\right) - 6.18^2}\right.}{\sqrt[3]{\dfrac{555}{15000}}}$	$T = \dfrac{\left[\sqrt{\left(\dfrac{4295}{457 \times 0.2}\right)^2 - 6.18^2}\right.}{\sqrt[3]{\dfrac{457}{15000}}}$
	$= \sqrt{77.4^2 - 6.18^2}\left(\dfrac{8.225}{24.66}\right)$	$= \sqrt{(47.1^2 - 6.18^2}\left(\dfrac{7.7}{24.66}\right)$
	$= 9.15 \times 8.45 \times 0.336$	$= 7.3 \times 6.4 \times 0.315$
	= 26 inches.	= 14.72 inches

APPENDIX - V

CALCULATIONS FOR PERCENTAGE VARIATION IN THICKNESS CORRESPONDING TO EQUIVALENT LATERAL STRESS OVER THE THICKNESS CORRESPONDING TO THE STANDARD VALUE OF LATERAL PRESSURE.

The data used in the followed calculations are taken from Table 17.

Soil Designation	For Deflection 0.1 inch	For deflection 0.2 inch
Soil I (CBRI Soil)	Reduction in thickness for suggested lateral stress	Reduction in thickness for suggested lateral stress
	= 30.35-2190	= 15.00 - 12.13
	= 8.45 inches	= 2.87 inches
	Percentage variation = $\frac{8.45}{30.35}$ x100	Percent variation = $\frac{2.87}{15.00}$ x 100
	= -27.82 percent	= -19.14 percent
Soil II (Blue Soil)	Increase in thickness for suggested lateral stress	Increase in thickness for suggested lateral stress
	= 42 60-30.15	= 25 10-1900
	= 12.45	= 6.10
	Percent variation = $\frac{12.45 x100}{30.15}$	Percent variation = $\frac{6.10}{14.00}$ x 100
	= + 41.3 percent	= +32.1 percent

Soil Designation	For deflection 0.1 inch	For deflection 0.2 inch
Soil III (Blended Soil)	Increase in thickness for suggested lateral pressure = 26 00-25.35 = 0.65 inches Percent variation = $\frac{0.65}{25.35}$ x 100 = + 2.56 percent	Increase in thickness for suggested lateral pressure = 14.72 - 13.67 = 1.05 inches Percent variation = $\frac{1.05}{13.67}$ x 100 = + 7.68 percent

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PHOTOGRAPHS