SOIL MECHANICS AND BITUMINOUS MATERIALS
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TEST ROAD TO DETERMINE THE INFLUENCE OF SUBGRADE CHARACTERISTICS ON THE TRANSIENT DEFLECTIONS OF ASPHALT CONCRETE PAVEMENTS

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Soil Mechanics and Bituminous Materials Research Laboratory

TEST ROAD TO DETERMINE THE INFLUENCE OF SUBGRADE CHARACTERISTICS ON THE TRANSIENT DEFLECTIONS OF ASPHALT CONCRETE PAVEMENTS

A report on an investigation

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INTRODUCTION

In recent years considerable attention has been directed to the problem of cracking of asphalt pavements resulting from repeated applications of traffic loads. This form of distress (termed fatigue herein) has been associated by Hveem (1) both with excessive deflection of the structural pavement section under load and with frequent repetitions of heavy vehicles.

Failures of this type in otherwise well designed pavements has created a need for methods of predicting the transient deflections of a pavement in order to assess the magnitude of the strains developed in the surfacing which, in turn, appear to determine the fatigue life of the pavement.

Because fatigue failures result from instantaneous and recoverable deflections in the pavement components and are not necessarily associated with any plastic or permanent deformations, it might reasonably be expected that the deflections to which they are attributed could be calculated, at least approximately, from appropriate elastic theory for layered systems. Information illustrating the feasibility of such an approach has already been presented (2,3,4). With the advent of modern electronic computers, some of the computational difficulties associated with the above noted procedures have been minimized; thus, so long as appropriate material response characteristics are available, it would now (1968) appear practicable to use such techniques to study the response of in-service pavements (5) and to consider the use of these methods in design (6).

Under certain circumstances, the subgrade soil may be a major contributor to these transient deflections (7). Recent laboratory studies of compacted clays have demonstrated that their resilient (or elastic) characteristics are dependent on water content, dry density and method of compaction (8). Moreover, these studies have shown that certain soils when compacted to high degrees of saturation tend to be more resilient than when they are compacted to corresponding densities at lower degrees of saturation. In addition, these investigations indicate that methods of compaction (in the laboratory) which tend to induce large shearing deformations cause fine-grained soils to be more resilient than do those methods which induce lesser amounts of deformation.

From studies of the AASHO Road Test subgrade soil, Seed et al (8) have indicated that kneading compaction in the laboratory can produce the same degree of resilience in laboratory prepared specimens of soil as that developed in the same soil when compacted in the field to high degrees of saturation by pneumatic-tired rollers.

While it has been demonstrated that the resilient characteristics of soils are related to the transient deflections of pavements (3,4,7) and that these characteristics are influenced

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3. Volume-change characteristics (swell, shrinkage and swell)

4. Resilience

by compaction conditions, no systematic studies have developed which (1) relate the influence of water content and dry density of fine-grained soils to the deflection of actual pavement structures and (2) define the influence of compaction equipment in developing varying degrees of resilient behavior in these materials.

The objectives the investigation reported herein were thus (1) to determine the influence of water content and method of field compaction on the resilience characteristics of a compacted clay and the influence of these characteristics, in turn, on the surface deflection of an asphalt concrete pavement; and (2) to ascertain the feasibility of the use of laboratory determined material characteristics together with appropriate theory to predict the response to load of a pavement constructed by conventional techniques.

Because of the scope of such an undertaking, personnel, time, and effort of a number of organizations were required. These organizations included, the County of Contra Costa, the Gordon Ball Company, the Materials and Research Department of the State of California Division of Highways, and the University of California. While the investigation did not attain both of the objectives noted above, it should be emphasized at the outset that it did demonstrate the feasibility of a number of interested groups working together in a unique joint venture to attain a specific objective and hopefully can serve as a pattern for future investigations.

Essentially this report presents the results of the investigations conducted by the staff of the University of California on this project and hopefully emphasizes as well the contributions of each of the agencies involved. In addition, since the investigation was concerned primarily with the response of the subgrade soil both as affected by placement conditions and method of compaction, a brief summary of available laboratory and field data pertaining to compacted clays is included in order to provide a perspective for the results presented herein.

This work was done in cooperation with the California Division of Highways. The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of the Division of Highways.

BACKGROUND

The purpose of soil compaction is to achieve improvement in one or a number of soil properties; among those properties of importance to the paving engineer are;

1. Compressibility
2. Strength
3. Volume change characteristics (i.e., shrinkage and swell)
4. Resilience

In the investigation reported herein, resilience has been the property of major interest. By resilience is meant the recoverable deformation obtained when transient (traffic) loads are applied and removed. This form of deformation has received considerable attention in recent years since it would appear related to the rupture mode of distress (fatigue) occurring, at times, in asphalt concrete surfaces of otherwise well designed pavement structures.

At the same time since considerable information has been developed relative to all of the above noted properties of compacted soils, this section will include not only a discussion of known information on resilience but also pertinent available data on the other properties as well. In this regard the research of Seed et al (8, 9, 10) will be drawn upon particularly for this summary.

In the ensuing discussion, it will be noted that much of the available information is developed from data on laboratory compacted specimens. Only limited information is available on properties of field compacted materials and, as noted in the introduction, is one of the major reasons for this investigation; however, this information is also briefly summarized. In addition, as a part of this section, available information indicating comparisons of the relationships between water content and dry density for laboratory prepared and field compacted materials is presented. This latter is confined, in the case of field compacted materials, to information obtained with sheepsfoot and pneumatic-tired rollers since the field experiment was limited to these types of equipment.

Finally it should be noted that the discussion is limited to compacted cohesive soils since proper compaction specifications may involve more than a requarement for a minimum acceptable density (as is currently the situation) to attain the properties noted above. This is in contrast to the situation for granular materials where high densities generally reflect the most desirable situation with respect to such properties as strength and compressibility.

Structure of Compacted Clay

A knowledge of the arrangement of clay particles in a compacted soil is useful in developing an understanding of properties. Lambe (11) has suggested arrangements of the platelike clay particles in compacted soils as shown in Fig. 1 and has verified these arrangements using techniques developed by Mitchell (12).

In this figure it will be noted that at water contents dry of optimum for particular compactive efforts (i.e., at A and E) the particles are arranged in a random array. This arrangement of grains has been termed a "flocculated" structure. With increase in water content, decreased interparticle attractions and increased repulsions due to expanded double layers of ions surrounding the clay particles permit the development of parallel particle arrangements which are termed "dispersed" structures (i.e., at C and D).

More recently, Seed and Chan (9) have shown that for most soils simply increasing the water content at compaction is not sufficient to create the dispersed structures shown in Fig. 1. Rather it is necessary to also induce shear strains in the material to force the plate-like particles into the parallel arrangements. Their studies have indicated that if the method of compaction does not cause comparatively large shear strains in the soil (e.g., static compaction in the case of laboratory prepared materials) then dispersed structures will not form. They also determined that some soils would not form dispersed structures at any compaction water content or as a result of unlimited shear strain during the compaction process. In general, however, at water contents dry of optimum for a particular compactive effort the structure will be flocculated regardless of the method of compaction.

Dispersed and flocculated compacted soil structures exhibit some district differences in mechanical properties which Mitchell (13) has concisely summarized as follows (note: in each case specimens are assumed to be at the same water content and dry density):

1. Dispersed structures are more compressible than flocculated structures.
2. Dispersed structures will shrink more on drying from the as-compacted state but will swell less on exposure to water than flocculated structures.
3. Flocculated structures are stiffer than dispersed structures; i.e., the slope of the initial portion of the stress vs. strain curve for a material with a flocculent structure is steeper than that for a material with a dispersed structure.
4. Dispersed structures will exhibit more total and resilient strain under repeated loads than flocculated structures.

These points will permit the explanation of the variation of a number of the observed properties with compaction conditions. Moreover the concepts presented above may have major import with respect to selection of field compaction conditions and the selection of the type of field equipment used to bring a particular soil to the desired state. Hopefully, the information summarized in the remaining portion of this section will provide at least a preliminary indication of the thought which should be given to the preparation of subgrades for highway pavements.

Properties of Laboratory Compacted Fine-Grained Soils

Compressibility. Generally, compressibility is minimized by compacting the soil to as high a density as is practicable with current methods of compaction. For structural

pavement sections for heavy duty highways, the dgree of compaction should be expressed in terms of the modified AASHO compaction test (or the California Impact Compaction Test) to achieve the necessary degree of compaction.

The Corps of Engineers have presented a useful guide, termed the compaction index, to assist the engineer in developing the necessary degree of compaction with depth (14) to minimize densification under traffic. Percent compaction vs. compaction index curves are shown in Fig. 2, and form the basis for compaction requirements shown in Table 1 for a typical highway load and a current aircraft loading.

Volume Change Characteristics. In Fig. 3 is shown the influence of molding water content and soil structure on the swelling characteristics of a sandy clay (9). The change in water content shown in this figure has been divided into that required for saturation at constant volume and that due to swelling. From this data it is apparent that the lower the water content at compaction, the larger may be the water content increase due to swelling.

Fig. 4 illustrates variations in swell pressure with molding water content for the same material. As seen in this figure, swell pressures of considerable magnitude may develop when the sandy clay is compacted dry of optimum for a particular compactive effort.

Method of compaction may also influence swelling characteristics as noted in Fig. 5. Kneading compaction wet of optimum produces more dispersed structures than static compaction for the same conditions because of the larger shear strains associated with the kneading process.

The data in general thus indicate that flocculent structures tend to swell more than dispersed structures and that to minimize swell or swell pressure, fine-grained soils should be compacted wet of optimum for a particular compactive effort or to a high degree of saturation. Even under these conditions, however, depending upon the amount of shearing deformation during the compaction process, varying amounts of expansion may be obtained.

To identify a soil to determine whether such considerations may be necessary at compaction, Seed et al (15) have suggested that the plasticity index appears to be a good indication of the swelling potential of cohesive materials under low surcharge (approximately one psi). The higher the plasticity index, as noted in Table 2, the greater the swelling potential.

As with swell characteristics, shrinkage is dependent both on molding water content and method of compaction. Both influences are illustrated in Fig. 6 for specimens of silty clay. In contrast to the recommendation for swell, the data in this figure indicate that to minimize shrinkage it is desirable to compact dry of optimum for a particular compactive effort and, in effect, produce a flocculated structure.

Such data thus indicate a point which will be reiterated in following sections; that is, it may be necessary to compromise in the selection of compaction conditions, keeping in mind a series of confliciing requirements which the soil is expected to meet.

Strength Characteristics. Both the compaction conditions (water content and dry density) and the method of compaction may have a significant influence on the stress vs. deformation and strength characteristics of compacted clays. As in the previous section, data developed by Seed et al (9, 10) will be utilized to illustrate these influences.

In considering the effects of compaction conditions and method of compaction it is desirable to examine separately the stress-deformation and strength characteristics for materials tested in the as-compacted condition and after soaking since both conditions may be applicable to highway subgrades.

As-compacted Conditions. Fig. 7 illustrates the variation in stabilometer "R" values over a range in water contents and dry densities for specimens of silty clay prepared by kneading compaction. Similar data are presented in Fig. 8 for the same material (prepared by kneading compaction) tested in triaxial compression with constant confining pressure. In this figure strength data are presented for two definitions of strength, i.e., the stress required to cause 5 percent strain and the stress to cause 25 percent strain. In most instances, behavior at low strains is of greatest interest in highway work. Accordingly, as noted in Fig. 8b, it can be seen that the highest strengths are obtained at water contents dry of optimum for a particular compactive effort* and that the higher the density the greater the strength. Similar results are shown in Fig. 7 since the stabilometer test is essentially a measure of the response of the soil at small deformations. Wet of optimum, on the other hand, the strength may not always increase with an increase in density as seen in Fig. 8e or in Fig. 7. When the strength is defined as the stress to cause a small strain, the shape of the stress-strain curve, which in turn is influenced by soil structure, has a marked influence on strength. At high strains, however, structural differences have been eliminated and strength increases regularly with density as noted in Fig. 8d.

It should also be noted that the type of behavior illustrated in Fig. 8e may not be representative for all soils since there are certain soil types in which neither water content increases nor shearing strain during compaction is sufficient to cause dispersion wet of the line of optimums. Such behavior is illustrated in Fig. 9 for a highly plastic clay and a sandy clay.

Thus, strength characteristics cannot be predicted simply. At the present time, direct strength determination in the laboratory is the best means available.

Saturation After Compaction. The relationships between soaked strength and initial com-

*Or in general dry of the "line-of-optimums" (i.e. the locus of points of maximum dry density and optimum water content for a series of compactive efforts).

paction conditions depend on a number of factors including initial soil structures, swelling characteristics, surcharge pressure during soaking, and the strain at which strength is defined.

Data presented in Fig. 10 illustrate a procedure for determining the strength after soaking for a range in compaction conditions for a silty clay. Results shown in this figure are for samples soaked at essentially constant volume (i. e., little or no change in dry density). The final diagram in which contours of equal strength have been superimposed on the dry density vs. water content relationship is particularly useful in assisting the engineer to select the compaction conditions to obtain a particular level of strength.

The data presented in Fig. 10 are based on strengths defined at small strains. Fig. 11 illustrates the effect of water content and dry density when strength is defined at a comparatively large strain. As noted earlier, structural differences are eliminated and the expected result that strength increases with density is obtained.

While the above data have been presented for a soil soaked at constant volume, a variety of responses may be obtained for soils exhibiting volume change during the process of saturation and, as noted previously, the relationship between strength after soaking and initial composition will depend on the amount of swell or expansion, initial structure, dry density, water content, strain at failure and the relative influence of these factors for a given soil.

To illustrate the range in response possible, examples will be presented for results of strength measurements on soaked specimens. For the data presented, the CBR will be used to represent a measure of the strength of soil. Like the "R" value, this strength index is a measure of strength at a comparatively small strain. Moreover it should be noted that these data represent soaking under conditions of low surcharge which are representative of the surcharge conditions existing in the subgrades of highway pavements.

Data for a lean clay are presented in Fig. 12. For this material, as noted from the strength contours, the lower the water content and higher the dry density, the higher the soaked strength. Thus, for field compaction at a particular dry density, it would appear desirable (from a strength standpoint) to compact this material comparatively dry.

Fig. 13 presents data obtained for another silty clay. As noted from the strength contours, it appears desirable to compact this material to a condition in the vicinity of the "line-of-optimums." For a particular level of dry density, moreover, water content control at compaction is necessary in order to achieve a specific strength.

Finally, the data for a highly plastic clay are presented in Fig. 14. For this material it will be noted that it is necessary to not only control the water content but also to control the dry density to achieve the highest strength after soaking.

As with strength in the as-compacted conditions strength characteristics of materials

saturated after compaction cannot be simply predicted. Accordingly, as noted previously, direct strength determination in the laboratory is the best means available.

Resilience Characteristics. When a soil is subjected to a series of repeated, short-duration loadings, both permanent and recoverable deformations are observed (8). After a comparatively small number of stress repetitions, the permanent deformation caused by each subsequent repetition is small compared to the total deformation (so long as the stress is not of sufficient magnitude to cause failure), whereas the recoverable deformation may still be a significant amount. This recoverable deformation, referred to as resilient deformation herein, is of particular interest in pavement design since such deformation may contribute to the fatigue mode of distress of otherwise well designed pavement structures.

As with the other properties of compacted soils, the resilience characteristics of fine-grained soils are significantly influenced by compaction conditions. The work of Seed et al (8) can also be drawn upon to indicate the effects of these conditions.

In the investigations of Seed and his associates, soils were subjected to repeated loads of durations corresponding to those which occur in actual pavements and to a range in frequencies of load application. By measuring the resilient strain in a repeated-load triaxial compression test, a resilient modulus can be determined from

$$M_r = \frac{\sigma_d}{\epsilon_r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

M_r = modulus of resilient deformation, psi (analogous to an elastic modulus)

σ_d = repeatedly applied deviator stress, psi

ϵ_r = resilient axial strain corresponding to a particular number of stress repetitions, in. per in.

Typical results from this type of testing are shown in Fig. 15.

From investigations covering a range in compaction conditions and soil types, the factors influencing the resilience of clays may be summarized as follows (3):

1. Number of stress applications. Resilient deformations generally decrease as the number of load repetitions increase (Fig. 15). Accordingly, deformations determined under a relatively small number of stress applications may present a misleading picture of the resilience characteristics of subgrade soils.

2. Stress intensity. As shown in Fig. 16, the resilient modulus increases as the intensity of stress decreases. This factor is extremely important, as will be seen subsequently, when analyzing stresses and deflections in pavement structures.

3. Method of compaction. Methods of compaction which tend to produce dispersed structures on soils tend to produce lower moduli of resilient deformation. This point is illustrated in Fig. 17 in which is presented a comparison of the results of tests on samples prepared by static and kneading compaction procedures.

4. Compaction density and water content. The influence of compaction conditions on the resilience characteristics of the AASHO subgrade material is summarized in Fig. 18. As the degree of saturation at compaction increases the resilient deformation at a particular stress level increases and the resilient modulus decreases. This is also true for other materials as shown in Fig. 19.

5. Changes in water content and dry density after compaction. In general, as the water content of the soil increases due to water absorption after placement, resilience increases; on the other hand, as the density increases, resilience decreases. Both points are illustrated in Figs. 20 and 21.

6. Age at initial loading. Samples compacted to high degrees of saturation increase in strength with time.* The resilient strain determined for small numbers of stress applications decreases as the time interval between compaction and testing increases, as shown in Fig. 22. However, after large numbers of stress repetitions, because of thixotropic changes and deformations occurring during repeated loading, the effects of aging are reduced and essentially the same results are obtained for specimens tested immediately after compaction as for specimens tested after a delay.

From this summary it can thus be seen that the initial compaction conditions as well as the method of compaction have an influence on the resilient response of fine-grained soils. As with the other properties it emphasizes the fact that careful attention must be paid to any laboratory measurement program designed to simulate field conditions. Moreover it emphasizes the need for an indication of the relation between properties of the same material prepared in the field as in the laboratory.

As an indication of the dilemma faced by the designer, consider the situation of a sample which has been prepared by kneading compaction and soaked to a condition representative of that expected at some subsequent time after placement. Resilient response for such a sample is shown in Fig. 23. If the designer were to prepare the sample to the same final conditions immediately by kneading compaction in order to save time in the laboratory (since it

*Although this point was not discussed earlier it may be a factor which should be considered by the designer where the strength of compacted soils are being considered.

takes considerable time for the material to become saturated) he would obtain a different result, if the soil were subject to dispersion, as shown in Fig. 23. If on the other hand this soil were prepared by static compaction to the same conditions recognizing that soil structure plays a role in defining soil response to small deformations, essentially the same result would be obtained (Fig. 23) as for the situation where the sample was prepared dry by kneading compaction and soaked to the particular state. In this case static compaction wet of optimum created essentially the same structure as kneading dry of optimum and for a particular water content and dry density, structure appears to be the controlling factor for resilience.

Whether or not either of the samples would depict the results obtained by field compaction equipment is, however, yet to be ascertained. Nevertheless such an example serves to illustrate the considerations which might be taken by the designer in order to develop realistic designs.

Summary. In general, the data presented in this section indicate that the properties of laboratory-compacted clayey soils depend on soil type, compaction water content and density, compaction method, and method of testing. Moreover, the data indicate that density alone may not be a sufficient criterion to properly reflect desirable soil properties, and that, at times, conflicting requirements for water content and dry density exist relative to the properties under consideration. With respect to this latter point, consider the case of an expansive soil. For such a material it would be desirable to compact the soil to a high degree of saturation initially in order to minimize expansion under a low surcharge. On the other hand, for the same material it would be desirable to compact it dry of the line of optimums to minimize resilience. If volume change is an overriding consideration and the soil is placed wet, then the designer must recognize that the pavement section must be designed to account for the increased resilience resulting from this compaction condition. Similar discussions might be presented for combinations of other soil properties. Thus it can be seen that careful attention must be paid to the selection of compaction conditions in order to achieve reasonable economies in design.

Finally it must be emphasized that this discussion has been limited thus far to properties of laboratory prepared samples. An important consideration, in order that the concepts discussed above can be put to use, is that the engineer must have a knowledge of the relationship between properties of field and laboratory compacted samples. Some discussion of limited studies in this area is presented in the next section.

Properties of Field Compacted Fine-Grained Soils

While a number of different types of compaction equipment are available for field compaction of fine-grained soils only two types were considered for this study, namely pneumatic-tired and sheepsfoot rollers. Available information (e.g., Ref. 16) has shown that these two types of equipment have been and are widely used to compact fine-grained soils. Thus studies of the effects of these types of equipment should provide a general indication as to the properties of field compacted fine-grained materials.

Comparisons Between Field and Laboratory Compaction. In a comparison of properties of field and laboratory materials it is necessary to have an indication of the relationships between water content and dry density produced by both field and laboratory compaction equipment. Accordingly, available data on such comparisons will be presented in this section for the two types of rollers. Most of the available data, it should be noted, are based on comparison of the field data to some form of laboratory impact compaction (generally the Standard AASHO (T-99), Modified AASHO (T-180) or the British Standard Method).

Pneumatic-Tired Rollers. Fig. 24 presents the results of field tests of pneumatic-tired rollers (17) using three different roller weights and three different tire inflation pressures. Also shown in the figure are the results of laboratory impact compaction tests on the same materials.

From a comparison of the field and laboratory curves it will be noted that the field curves are slightly steeper to the left of their line-of-optimums than those produced by laboratory compaction. To the right of their respective lines-of-optimums, however, the compaction curves for both types are similar. In addition it will be noted that the field line-of-optimums lies to the right of that determined in the laboratory.

Data from another study (18), shown in Fig. 25, substantiates this latter point, that is, that the field line-of-optimums lies to the right of that obtained from the laboratory impact compaction methods; in general all available data substantiate this point.

As will be seen subsequently, when considering the effects of compaction on soil properties and when selecting field compaction conditions, this difference should be taken into account.

Sheepsfoot Rollers. A comparison between roller and laboratory results is provided from tests in which three sheepsfoot rollers with foot areas of 7, 14, and 21 sq. in. were used to compact a silty clay (19). Field curves were developed for 6, 12, and 24 passes of the rollers all operating a constant contact pressure* of 250 psi. The lines-of-optimums

*Contact pressure for a sheepsfoot roller is usually determined by dividiing the total weight of the roller (weight as it is to be used) by the product of the number of feet in one row and the contact area per tamper foot. It should be noted that this may have no relation to actual pressures.

for the field and laboratory (by impact compaction) compacted soils are shown in Fig. 26. As noted in this figure in the range of dry densities which might be utilized in construction (between AASHO T-180 and T-99 maximums) the line of optimums for field compaction lies to the left of the laboratory line of optimums.

This position of the field line of optimums with respect to the laboratory determined line has also been shown by the Road Research Laboratory for four soils which they investigated (20). Their results are presented in Fig. 27.

As with the pneumatic-tired rollers such response should be considered when selecting initial placement conditions.

Equipment Use Considerations. Many of the studies reported in the literature (e.g. Ref. 16 contains an excellent summary) have had as their major purpose compaction plant output and the data reported in the previous section has, in reality, been only one of many findings from such investigations. Relative to equipment operation the following summarizes some of the important findings:

Pneumatic-Tired Rollers:

1. The greatest compaction increase results from increasing both tire-inflation pressure and wheel load. A wheel load increase without a corresponding increase in tire-inflation pressure has little effect on compaction while an increase in tire-inflaction pressure without an increase in wheel load tends to produce greater compaction near the surface.

2. The greatest rate of increase in dry density occurs during the first eight passes of the roller and only small increases are observed after 16 passes.

3. For the normal range of loads and contact pressure compacted lift thicknesses should be limited to 6 to 8 inches to insure a uniform, high degree of densification. However, heavy wheel loads and high tire-inflation pressures may provide greater depths of compaction but may require precompaction with lighter rollers to prevent initial sinkage and shaving of the loose lift.

Sheepsfoot-Type Rollers:

1. As a general rule, roller feet should have as large areas as practicable and yet be compatible with adequate unit pressure requirements and proper spacing for cleaning purposes.

2. The number of passes required to obtain a given dry density depends on the foot pressure and percent coverage per roller pass. These in turn depend upon the gross weight of the roller, the area of each foot, and the number of feet in contact with the ground. Generally the best results are obtained when the contact pressure is as large as possible without exceeding the bearing capacity of the soil.

3. Compacted lift thickness should not exceed the length of the foot by more than about two or three inches to achieve a comparatively uniform degree of densification.

Properties of Field Compacted Soils. As noted previously, many of the full-scale field compaction studies have been concerned primarily with effectiveness of various compaction equipment in different types of soils. As a part of these studies properties of the field compacted materials were at times measured (17, 19, 21, 22, 23). The majority of such reported data describes the effects of compaction on strength. The study of Seed et al (8) does include, however, measurements of the resilient characteristics of undisturbed samples of the AASHO subgrade soil. Data of Zube and Forsyth (7) also include a measure of the resilience characteristics of a number of compacted subgrades from throughout the State of California.

Strength Characteristics. Laboratory and field studies conducted by the Corps of Engineers on compaction included a comparison of laboratory and field strength determinations for both a silty clay and sandy clay compacted in the field by Sheepsfoot and pneumatic-tired rollers. Water content, dry density and strength (as defined by the CBR) relationships for the silty clay are shown in Figs. 28 and 29.

In Fig. 29 it will be noted that there are significant differences in the strength characteristics (both soaked and unsoaked CBR) between the field and laboratory prepared specimens. In addition it can be seen that there are differences in the strength characteristics of the field samples compacted by the sheepsfoot and pneumatic-tired rollers, even though essentially the same water content - dry density relationship had been obtained. One might therefore, conclude that the soil structure induced by the laboratory and field compaction procedures is different and that different methods of field compaction induce different soil structures.

In this study stress vs. strain characteristics in undrained triaxial compression tests were also developed for laboratory prepared (impact compaction) and undisturbed samples compacted by both types of equipment. At water contents below optimum for a particular compactive effort, the slopes of the stress-strain curves for both laboratory and field prepared specimens were essentially the same. Above optimum, however, the laboratory prepared materials appeared to have a more dispersed structure than the field prepared specimens since the slopes of the stress-strain curves for the laboratory specimens were flatter.

From such data it is difficult to generalize in the relative characteristics of field and laboratory prepared specimens. However, these data do indicate that the structure of soils compacted in the field may be different than that developed in the laboratory and that different

types of field equipment may produce different structures in fine-grained soils which may, in turn, affect their strength characteristics.

Resilience Characteristics. Some data are available on the resilience characteristics of field compacted fine-grained soils and their comparison with laboratory prepared specimens.

Seed et al (8) have presented data comparing the resilient response of undisturbed samples of the subgrade from the various loops of the AASHO Test Road with specimens of the same material compacted in the laboratory by both static and kneading compaction procedures. Results of their tests are presented in Fig. 30 for the untrafficked section and in Fig. 31 for the sections subjected to traffic. Initially the subgrade was compacted to a high degree of saturation (wet of the line of optimums) with pneumatic-tired rollers.

From the data presented in the figures it will be noted that the laboratory kneading compaction procedure produced specimens with about the same degree of resilience as field compaction with the pneumatic-tired rollers. From this data it may be concluded, for this soil at least, that kneading compaction induces a similar structure to that produced by pneumatic-tired rolling.

The State of California in developing its resilience design procedure (7) examined the response of undisturbed samples and corresponding samples of the same materials prepared in the laboratory. Data supplied by Forsyth (24) and shown in Table 3 would tend to indicate that laboratory compaction by the kneading process will not necessarily reproduce the same response as obtained from specimens computed in the field.

Summary. From the data presented it can be seen that the effects of field compaction must be considered in any systematic laboratory investigation of soil properties for pavement design purposes. Since the structure of fine-grained soils influences their properties, any laboratory compaction procedure which is used to prepare test specimens must reflect the structure induced by field compaction procedures.

For samples compacted dry of the line optimums it would appear that static or kneading compaction should produce resilience characteristics similar to those obtained by field compaction procedures to the same conditions of dry density and water content. Wet of the line of optimums the same generalization cannot be made. In this region, depending upon the water content and the amount of shearing strain which takes place during the compaction process, different structures will, in all probability be induced by different compaction procedures. Moreover, as seen from this discussion, the fact that pneumatic-tires and sheepsfoot rollers produce lines-of-optimums differing from that determined in the laboratory (at least by impact compaction) an additional factor is introduced in attempting to

assess the proper method for laboratory conditioning of the soil for testing purposes, particularly when considering placement conditions near the optimum water content for a particular compactive effort.

Finally, the ability to place fine-grained soils to specific conditions of water content and dry density in the field must be considered since variations in properties (such as resilience-illustrated in Fig. 18) should be accounted for in any preliminary program.

It is with such items in mind that the investigation described in the subsequent sections was undertaken. More specifically its purpose was to ascertain the effects of sheepsfoot and pneumatic-tired rolling on the resilience characteristics of a fine-grained soil and the effects of these characteristics, in turn, on the surface deflections of an asphalt concrete pavement.

From the discussion presented one might hypothesize that if there is to be a difference in structure induced by compaction equipment, this should occur with the sheepsfoot roller, particularly wet of the line of optimums since this type of equipment has the potential for inducing comparatively large shearing deformations in the soil. While it is recognized that shearing deformations are present under pneumatic-tired rollers, it seems reasonable to assume that they are not as large as those developed under the sheepsfoot roller, particularly if the tire pressure and roller weight are controlled. This increased shearing deformation should result in more dispersion and thus increased resiliency in the subgrade. Accordingly, larger transient deflections would be expected at the surface of the finished pavement structure.

TEST ROAD

To meet the objectives discussed in the previous section, a full-scale test road rather than a prototype pavement appeared in order. Through the efforts of Contra Costa County and the Gordon H. Ball organization a site was made available to construct the required test road in early 1966.

Variables in the test road were to be a range in water contents for the prepared subgrade together with two different methods for compacting this material through the use of sheepsfoot and pneumatic-tired rollers. It was expected that the range in water contents as well as the different methods of compaction would produce different structures in the compacted clay. By constructing a pavement section of uniform thickness over the subgrade, any variations in the response of the pavement to load could thus be attributed to the variations in the subgrade soil which had been induced by compaction water content and method of compaction.

Description

The test road site is shown in Fig. 32. Total length of the roadway which has been designated Via de Mercados, is approximately 1100 ft. Six test sections were planned to provide three different water contents for each compactor. The water contents were selected to provide compacted soil conditions in the test sections dry of, in the vicinity of, and wet of the line of optimums. These six test sections are shown schematically in Fig. 33 and each are approximately 150 ft in length. Also, as noted in Fig. 32 and in Fig. 33, the test sections were placed only on the east half of the road which has a total width of 48 ft curb to curb.

The structural pavement section, Fig. 34, consists of 1.5 ft of the compacted cohesive soil on top of the original ground, 0.92 ft (11 in.) of compacted untreated Class II granular base, and 0.60 ft (7.2 in.) of asphalt concrete placed in three lifts.

Subgrade. The subgrade material for the test road (1.5 ft of compacted cohesive soil, Fig. 34) consisted of a silty-clay which was stockpiled near the site prior to the start of construction. Laboratory tests on this material prior to construction indicated that it would be a material whose compacted structure would be sensitive to water content at compaction and to shearing deformations during the compaction process.

Standards test results for this material are shown in Table 4 and in Fig. 35. According to the Unified Soil Classification, the material would be classed as a CL soil.

To obtain the desired water content conditions referred to above without establishing roller compaction curves in the field it was necessary to utilize the results of impact compaction tests in this material together with available information on the compaction characteristics of the two types of rollers (such characteristics have been described in a previous section).

While it was desirable to attempt to compact the soil to a constant density to simplify interpretation, it was recognized that it would be virtually impossible to do this. It was recognized also that it would be desirable to compact the soil to a degree of compaction which would correspond to that required by the user agency (i.e., Contra Costa County). Accordingly, a relative compaction of 90 percent ± 2 percent (based on the California Impact Compaction Test) was selected.

Laboratory compaction data for this material are presented in Fig. 36. Shown in this figure also are the acceptable range in dry densities, the laboratory line of optimums, and estimated field lines of optimums for the two compactors.

Based on the information presented in Fig. 36, water contents of 14, 19, and 22 percent were selected. These water contents would appear satisfactory to achieve the desired conditions providing that some care would be exercised in preparing the section at 22 percent.

Fig. 37 illustrates the resulting layout of sections which was selected to obtain the best control of water content.

Untreated Aggregate Base. The aggregate used in the base course was obtained from a source located on the northeast slope of Mt. Diablo (PCA Clayton pit). This material meets the requirements for a State of California Class 2 Aggregate Base with a 1-1/2 in. maximum size. Some test properties for the aggregate are presented in Table 5 and its grain size distribution is illustrated in Fig. 38.

Asphalt Concrete. The asphalt concrete conformed to the State of California requirements for a Type B material and was composed of an 85-100 penetration asphalt and a crushed aggregate from the same source as the untreated base. For the first (bottom) course a 3/4 in. maximum size material was used whereas a 1/2 in. maximum size material was used in the succeeding two courses (Fig. 34).

Construction

Since the test sections were to be located on the site of the access road to the Gordon H. Ball Equipment Yard, it was necessary to construct the non-test or west position of the highway (Fig. 32) in its entirety prior to construction of the test road. Following completion of this portion, traffic was routed over it and construction of the test road was begun in October 1966.

Site Preparation and Grading. The existing roadway was removed and the resulting surface graded and compacted in preparation for the silty clay subgrade. Compaction was deemed necessary to insure uniformity of the surface disturbed in the preparatory operations. An attempt was made to check the uniformity of the resulting section using a Benkelman Beam and loaded truck. Because of the relatively soft nature of the in-place materials erratic results were obtained and thus are not included in the report. *

A trapezoidal fill, shown in Fig. 33, was then constructed to provide a trench section to contain the subgrade and base materials.

Subgrade Construction. The subgrade was placed in three lifts using a John Deere self-loading rubber-tired scraper. This process required three days of construction time and proceeded in the following manner. After eight inches of loose material was placed on the roadway, a farm-type disc pulled by a track-laying tractor was used to mix the soil. A water wagon with a pressure apparatus was driven slowly down the nontest roadway spray-

*The Dynaflect, (27) had it been available at the time, or some form of vibratory testing would appear to be a practical method to make such a check.

ing the full length of the loose subgrade material. The nozzle was set to cover the entire width of the subgrade of the test side in one pass. A second pass was made covering the middle four sections. On the third pass only the middle two sections were sprayed. After the disc had mixed the entire width of the subgrade, the watering operation was repeated. To allow for water loss during compaction, the watering and mixing procedures were continued until the desired molding water content for each section was slightly exceeded. At that time the rolling operation was started. *

The sheepsfoot-type roller, a 5x5 pulled by a track-laying tractor, readily achieved the desired degree of compaction with the 8-in. loose lift. The pneumatic-tired roller, an 11-wheel roller which could not be ballasted (gross wt. 11,200 lbs.), required that lifts of approximately 2-in. be used before the required degree of compaction could be obtained. This was accomplished by using a motor grader to place the mixed subgrade material in a berm on one side of the test section and then spreading it in thin lifts and rolling. For the first 8 in. lift this procedure worked reasonably well since the weather was foggy and cool. However, during the placement of the second two lifts, the weather was clear, sunny and hot. Accordingly, the hot weather and the 2-in. lifts made it virtually impossible to retain the water in the soil thus limiting the success of the operation.

After rolling, density tests which are summarized in Appendix A-1 were performed using a Hidrodensometer provided by Pacific Air Industries. The Unit's calibration curve using the backscatter technique for wet density and water content were checked using the Standard Sand Density Test of the State of California and high temperature (400°F) oven drying. Results of the check tests are presented in Appendix A and a comparison of nuclear and conventional appeared to provide good correlation.

After the first lift of subgrade was compacted and density and water content were checked, the second lift of material (8-in. loose thickness) was placed to reduce the possiblity of drying of the compacted subgrade layer. On the following day the mixing, watering and compacting processes were repeated. Density and water content were checked in this lift after compaction using only the nuclear device. As before, the compacted material was

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*Originally, it was planned to use a travel plant for incorporation of water into the subgrade soil; just prior to construction, however, this idea was rejected because of the possibility of the soil in the wetter sections jamming the mixer. A preliminary compaction study developed by the IIT but which was not available until after the test road was completed provided some useful suggestions. For example, it was determined that adding water to soil and mixing using a disc was not satisfactory since water was not uniformly mixed into the soil and depth of mixing could not be controlled. In addition it was noted that consideration should be given to using a pulverizing mixer for mixing water into the lift (25).

covered with the next loose layer. The operation of the second day was repeated on the third day.

Upon completion of compaction of the third lift, the subgrade surface was trimmed to proper elevation with a motor grader and the pneumatic-tired roller was used to roll the surface tight. An attempt to obtain undisturbed samples of the subgrade was made at this point. This sampling was not successful and resampling was accomplished after field tests on the subgrade were completed.

To provide a barrier for reducing moisture loss, an asphalt emulsion was applied to the subgrade surface at the rate of 0.25 gal. per sq. yd.* The subgrade was not watered prior to the application of the emulsion and the dust provided a barrier which prevented the asphalt from being absorbed into the subgrade. In retrospect it probably would have been better to cover the subgrade with sheets of plastic to obtain a more positive seal against loss in water and to provide cleaner working conditions.

Placement of Aggregate Base. Aggregate base was placed from bottom dump trucks in slurry form and did not require the addition of water. It was dumped on the subgrade, spread with a motor grader and compacted using a smooth, steel-wheeled roller. Density control and uniformity checks were to have been made using the Hidrodensometer; however, a licensed operator was not available at the time of construction. Thus, in-place densities were obtained by means of two sand volume tests and the desired check on uniformity was not completed.

Repeated plate load tests were started at the surface of the aggregate base at one end of the road before the material was placed on the entire road in order to complete this phase of the testing and place the first lift of asphalt concrete prior to the first seasonal rain.

Placement of Asphalt Concrete. After completion of repeated plate load tests on the base, the first 2.4-in. lift of asphalt concrete was placed. Prior to paving, a prime coat of asphalt emulsion was applied to the base course. Placement of the asphalt concrete was accomplished using a Barber-Greene Paver. Standard State of California practice was followed with one exception. Because of availability, the same 11-wheel pneumatic-tired, 11,200 lb. roller which was used to compact the subgrade was used in lieu of a 12-ton (24,000 lb.) roller required by California Standard Construction Specifications. This light roller did not achieve the desired degree of compaction with the result that a pervious

*Originally, it was planned to use an asphalt canal lining material supplied by the Chevron Research Company. Because necessary equipment was not available to emulsify this material for application a regular asphalt emulsion was utilized.

surface was obtained; in addition, the first rain occured on the day following paving.

Repeated plate load tests were run on each of the six sections. Block samples of the surfacing were removed from the pavement by a State Division of Highways sampling crew. After a delay of about three weeks following testing of the first pavement surface caused by unfavorable weather, the second and third lifts of asphalt concrete was placed. A tack coat of asphalt emulsion was applied to the pavement surface prior to placing the second lift. No tack was provided between the second and third lift. The paving operation was similar to that previously described; however, a 12-ton pneumatic-tired roller was substituted for the pneumatic-tired roller used previously. After repeated load tests on the top of the third lift, samples of the in-place asphalt concrete surfacing w re removed from the pavement. During the sampling operating some indication of a poor bonding between the intermediate and top courses of asphalt concrete was obtained.

Summary

While numerous problems occurred during the construction operations, the inability to obtain the desired molding water contents in the subgrade was the only one of major consequence. The combination of hot weather, the light pneumatic-tired roller and an ineffective moisture barrier eliminated the possibility of comparing the resilient characteristics for material compacted at water contents wet of the line of optimums using the sheepsfoot-type and pneumatic-tired rollers.

FIELD TEST PROGRAM

To extend the applicability of prior studies on prototype pavements (3,4) it was planned to accomplish field testing of the roadway sections using repeated plate load tests on the surface of the subgrade, base, and first and third lifts of asphalt concrete in the test road. So that the approach might also be used to predict the response of full-scale pavements to traffic loading, deflections under a loaded truck as measured by the Benkelman Beam were planned at the surface of the first and third lifts of the asphalt concrete. In addition the State of California planned to supplement these measurements using the Dynaflect.

Equipment and Procedures

Repeated Plate Load Tests. Repeated plate load tests to determine resilient response of the various test sections were conducted using a modification of equipment developed in an earlier investigation (4), the modification necessitated because of the larger load required in this study. This equipment consisted of a pneumatic-hydraulic piston (shown schematically in Fig. 39) bolted to a reaction frame together with air surge tanks, a timer, and pressure gauges mounted in a cabinet to facilitate field oper;ation. Reaction for the loads was

obtained from the weight of beam and concrete blocks (each weighing approximately 3000 lbs.). Typical installations for tests at the surface of the subgrade and at the surface of the asphalt concrete are shown in Figs. 40 and 41 respectively.

Load was applied through rigid steel plates ranging from 8 in. to 30 in. diameter at a frequency of 20 applications per minute and a duration of 0.25 sec. Load vs. time characteristics of the piston were checked periodically using a load cell, the output of which was recorded by means of a Sanborn Strip chart recorder. The magnitude of the load was continuously checked by means of a pressure gauge attached to the surge tank on the pressure side of the piston (Fig. 39).

Resilient deflections were measured from a reference beam 20 ft in length and stiffened laterally to prevent sway. (Figs. 40 and 41.)

Electricity for operation of the test equipment was generated in the field and regulated to provide 115 volts at 60 ± 1 cycles. Air required to operate the loading piston was obtained from a compression regulated at 120 ± 5 psi.

Repeated loads were applied to the surface of the subgrade, the untreated aggregate base, the first 2.4 in. lift of asphalt concrete and to the surface of the finished structure through a series of 1-in. thick steel plates 30, 24, 18, 12, and 8 in. in diameter. When the larger diameter plates were used, a rigid system was provided by using in combination all of the smaller plates. Fig. 42 illustrates a plate test on the subgrade using an 18 in. diameter plate.

The asphalt emulsion seal previously described was required in order to test the sections at placement water contents since one day was needed to test each section making a minimum total of six working days before the aggregate base could be placed. This test plan was predicated on a 10 or 11 hour day which allowed 1-1/2 to 2 hours for moving the test equipment and 8 to 9 hours for testing.

To provide a maximum number of plate sizes and pressures for the tests, testing was terminated after 500 applications at each stress level for a particular plate size. This was done after examining previous data from prototype pavements (3,4) and after carrying several pressures to 1,000 applications in the first subgrade section tested. It appeared that beyond 500 applications little change in resiliency would occur. In addition, since construction did not start until the first of October a minimization of time spent testing was deemed desirable in order to permit paving before the first rain of the season.

A 10'x 10' sheet of plastic was placed at the center of the location of the tests and the plates were seated above this on a thin layer of hydrostone. The purpose of the hydrostone was (1) to assure conformity with the subgrade, and (2) to provide a level surface for the plate. Deformations were measured with three dial indicators located at 1/3 points around

the circumference of the plate. The resilient deformation for the top of the course under test was taken as the average of the three dial readings. For tests with the 30-in. diameter plate on the subgrade, 0.0001-in. least reading dial indicators were used. In all other tests 0.001-in. dial indicators were used.

For tests at the surface of the two and three-layer systems, resilient deformation of the base-subgrade interface at the center of the plate was also measured using a removable unit. This unit consisted of a 1/4-in. diameter rod of the correct length with threaded ends to which 1/2-in. diameter discs were attached both top and bottom. The rod was inserted in a casing which was bushed on both ends to minimize friction and to prevent the entrance of soil. At a particular test site, the cased rod was placed in a 9/16-in. diameter hole which had been drilled by hand (star drill and hammer) through the overlying material to the base-subgrade interface. During installation the bottom of the casing was positioned approximately 3/8-in. above the lower disc and grouted into place using hydrostone to prevent movement of the casing which in turn might result in movement of the rod. Movement of the rod was measured by a dial indicator. Fig. 43 shows a schematic section of this device.

Benkelman Beam Measurements. Measurement of resilient deflections between the dual tires of a truck with a 15,000-lb. axle load and a 70 psi tire-inflation pressure were performed using a Benkelman Beam. Single observations at 20-ft intervals were made along the length of the test road together with three observations at each plate load test location. Such measurements were made at the surface of the first and third lifts of asphalt concrete using the Canadian procedure for rebound deflections (26).

Dynaflect Measurements. Measurement of dynamic response at the surface of each of the layers was made by the California Division of Highways using the Dynaflect (27). These readings were taken at the locations of the Benkelman Beam measurements on the two surfaces of the asphalt concrete. Also, at the same locations, readings were taken on the surfaces of the subgrade and aggregate base.

Test Results

Results from the various field loading tests are presented in this section. For plate load tests, results are presented for typical test sections graphically since all of the data have essentially the same form. It should also be noted that the response of the pavement structure to load prior to and after paving is, in all probability, different. Prior to paving the base and subgrade had a comparatively low degree of saturation which changed after the placement of the first lift of asphalt concrete and resulted from the previous nature of the surface and the heavy rain experienced immediately following construction of this course.

Repeated Plate Load Tests. Four series of repeated plate load tests were performed on each of the six test sections, details of which are shown in Table 6.

Subgrade. The prepared subgrade was subjected to 500 applications of each applied pressure for the plate sizes shown in Table 6. A measure of elastic response, termed modulus of resilient deformation (or resilient modulus) was determined from:

$$M_R = \frac{1.18 \times \sigma_o \times r}{\Delta_R}$$

where:

M_R = modulus of resilient deformation, psi

σ_o = pressure applied to plate, psi

r = radius of plate, in.

Δ_R = average resilient deformation, in.

Fig. 44 illustrates the relation between modulus and plate pressure for each of the six test sections. It should be noted that the form of the relationship is the same as that observed in earlier tests on prototype pavements (3,4).

Two Layer-System. For tests at the surface of the aggregate base, measurements of resilient deformation were obtained at the top of the plate and at the base-subgrade interface. A complete summary of data obtained at 500 stress applications is presented in Appendix B. Fig. 45 illustrates the test results for Section 1 for plate diameters of 8, 12, and 18 in. These results are also similar in form to those obtained earlier from tests on prototype pavements.

Three-Layer System. As with the tests at the surface of the two-layer structures, resilient deformations were measured at the surface of the plate and the base-subgrade interface. No measurements were obtained at the asphalt concrete-aggregate base interface since previous investigations (3) had indicated the resilient deformation in the asphalt concrete to be very small. In section 4, for tests at the surface of the 2.4 in. thick asphalt concrete layer, no measurements were obtained at the subgrade-base interface due to mechanical difficulties in the measuring device.

Fig. 46 presents the results of tests in section 5 at the surface of the 2.4 in. asphalt concrete layer. Similarly Fig. 47 illustrates the results for the same section for tests at the surface of 7.2 in. of asphalt concrete. A summary of all tests results is included in Appendix B.

During the testing, temperatures were measured in the asphalt layer. A complete summary of the temperature data is also included in Appendix B and typical plots of temperature variation with thickness are shown in Fig. 48 for section 5.

Earlier it had been noted that the degree of saturation of the pavement components was lower during tests on the two layer systems than during tests on the three layer systems because of rain following completion of the first asphalt concrete course. Evidence of this difference was noted when drilling the holes for rods to measure the subgrade-base deformation for the tests at the surface of the 2.4 in. asphalt concrete layer, in that a free water surface was observed in the aggregate base in four of the test sections. It should also be noted, however, that a free water surface was not observed in the base during the tests at the surface of the 7.2 in. asphalt layer.

Benkelman Beam Measurements. Results of the Benkelman Beam measurements at each of the test sites are shown in Table 7. In addition to deflection measurements at the repeated load test sites, deflections were also measured at 40 points over a 750 ft length along a line 12 ft from the back of the east curb and parallel to the longitudinal axis of the road. These results are summarized in Fig. 49.

Dynaflect Measurements. In Fig. 50 are summarized the results of the Dynaflect measurements made at the same points as were the Benkelman Beam measurements. It will be noted that the results presented in Fig. 50 have the same form as those presented in Fig. 49.

LABORATORY TEST PROGRAM

Specimens of the various materials used in the pavement cross section were tested in the laboratory to determine their response to repeated loading which could in turn be used with appropriate theory to predict the response of the pavement sections to load. Comparisons between measured and predicted response (transient deflections in this investigation) should thus indicate the validity of the approach.

Equipment and Procedures

Silty Clay Subgrade. Two types of equipment were utilized to load both laboratory prepared and undisturbed samples of the subgrade in repeated loading. The first type used both for specimens 1.4 in. in diameter by 3.5 in. high and 2.8 in. in diameter by approximately 6 in. high is essentially that developed by Seed and Fead (28). The second type, a modification to the first, was developed to apply small stresses corresponding to those resulting in subgrades of well designed pavements (usually less than 3 psi for highway loadings).

This modified unit consists of a 3/8 in. diameter rolling diaphragm mounted in a cylinder clamped to the top of the triaxial cell containing the specimen. A small solenoid actuated valve is incorporated in the plastic tubing through which regulated air flows. At an operating pressure of 120 psi, a maximum load of about 6 pounds is applied to the specimen. Typical setups are illustrated in Fig. 51.

Deformations were measured either by dial gauges or by means of linear variable transformers (LVDT's). Fig. 51 illustrates both types of measuring devices and in the case of the LVDT's illustrates the technique wherein small deformations were measured by clamping the device to the specimen.

Laboratory tests on the subgrade material consisted of (1) preliminary studies on laboratory prepared specimens for selection of a suitable subgrade material; (2) tests on undisturbed samples of the test road subgrade; and (3) a correlation study to attempt to produce field conditions by means of laboratory compaction.

Both the preliminary and correlation studies were conducted on 1.4 in. diameter by 3.5 in. high specimens prepared using a modified version of the Harvard minature kneading compactor.

Undisturbed specimens were 2.8 in. in diameter by approximately 6 in. high. Essentially two conditions were obtained in the undisturbed sample. For those taken soon after the plate tests on the subgrade, the degree of saturation was comparatively low; whereas for those taken approximately six months following the tests at the surface of the completed pavement, the degree of saturation was comparatively high. For the comparatively dry specimens, trimming of the ends to make them plane resulted in irregularities. Accordingly, each of these specimens was mounted on its cap and base in hydrostone.

To prevent changes in water content during testing each specimen was surrounded by a rubber member which in turn was sealed to the cap and base by a neoprene o-ring.

The trimmed specimens were subjected to 70,000 repetitions of deviator stress ranging from 1.0 to 10 psi in unconfined compression at a frequency of 20 applications per minute and with a duration of loading of 0.25 sec.

Untreated Aggregate Base. For tests on aggregate base the repeated loading unit mounted on a single bay loading frame shown in Fig. 52 was used. The cell shown in this figure is capable of testing specimens up to 6-in. in diameter and 13.5-in. high. Actually two different cells were used for tests on the aggregate base, one capable of testing specimens 3.9-in. diameter by 7.8-in. high and the other for 6-in. diameter specimens.

Initially, tests on aggregate base were conducted using both 6-in. diameter and 3.9-in. diameter specimens. Material grading used for preparation of the 6-in. diameter specimens was an average of the grading analyses of samples from the test site. The grading for the

3.9-in. diameter specimens was modified to eliminate all material greater than 3/4-in. while maintaining the same percentages of the minus No. 4 as in the 6-in. diameter specimens. These grading curves have already been presented in Fig. 38.

The sample preparation method using a V-55 Syntron Electric Vibrator described by Seed, et. al (3) was used to compact 3.9-in. samples. This procedure was modified for 6-in. diameter specimens by using a 15-lb. surcharge weight and compacting in three equal layers with one minute of vibration per layer. Specimens of 3.9-in. diameter were prepared to a density of 142.1 pcf which corresponds to the density of 146.8 pcf used for the 6-in. diameter specimens. *

Two methods were used to prepare partially saturated 3.9-in. diameter specimens. Initially dry material was compacted to the desired density; following compaction de-aired water was introduced under a small head through a tube in the base and allowed to permeate through the specimen. After approximately one hour, when water reached the top of the sample, the cap was set into place and testing proceeded. An improved technique developed during the investigation was to mix water (e.g., 7 percent) with the material before placing it in the mold and then to proceed as with the dry material.

Repeated load tests were conducted on the dry and partially saturated specimens at confining pressures ranging from 1.0 psi to 40 psi and at total vertical stresses ranging from 6.0 psi to 60 psi. Principal stress ratios varied from 1.5 to 6.0. Frequency of stress application was 20 per minute at a duration of 0.25 seconds.

Asphalt Concrete. Laboratory tests on specimens of asphalt concrete surfacing were conducted at two temperatures to define the relationship between modulus and temperature using equipment developed by Deacon (29).

A water-cooled diamond saw was used to trim flexural beam specimens 1.5 x 1.5-in. cross section and 15-in. in length from blocks of asphalt concrete obtained from the surface course. These specimens were subjected to repeated flexure with a load duration of 0.1 second and a frequency of 100 applications per minute. Flexural stiffnesses were determined for each beam at a variety of stresses by measuring the dynamic deflection at the center of the beam under each stress after 200 applications. These tests were conducted at 41°F and 68°F, since these temperatures represented the limits of those obtained in the actual pavement during the field repeated load plate tests.

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*These dry densities correspond to a value which is approximately 96 percent of the maximum dry density as determined by the State of California Test Method No. Calif. 216-F for materials passing the 3/4-in. sieve. This was the average density of the aggregate base as determined from field density tests.

Test Results

Silty-Clay Subgrade Soil. Prior to construction, three materials were investigated in the laboratory to determine their suitability for use in the test road. Results of repeated load tests on the material finally used are shown in Figs. 53 and 54.

Fig. 53 illustrates the variation in resilient strain with water content and dry density at a repeated deviator stress of 5 psi and a confining pressure of 2 psi. In this figure it will be noted that the material exhibits a four-fold increase in resilient strain for a change in water content from 14 to 22 percent (the range planned for the test sections) at a relative density of 90 percent based on the California Impact Compaction Test. It was thus expected that under field conditions this variation should provide readily observable differences in resilience characteristics between the test sections.

In Fig. 54 is shown the influence of molding water content on the modulus of resilient deformation, this modulus being determined from:

$$M_R = \frac{\sigma_d}{\epsilon_R}$$

where:

M_R = modulus of resilient deformation, psi
σ_d = applied deviator stress, psi
ϵ_R = resilient axial strain, in. per in.

It would appear from the data presented in Fig. 54 that the modulus of this material is little affected by density for the range considered in this investigation.

Results of repeated load tests on 2.8 in. diameter undisturbed specimens of the subgrade sampled immediately after completion of the plate load tests are shown in Fig. 55 for section 2. Also shown in this figure are modulus values determined from plate load tests. It should be noted that the abscissa have not been corrected using the relationship suggested by Seed et al (8) to compare field and laboratory test results. Even without this correction, considerable discrepancy in modulus exists between the two methods. From an examination of both density and water content data for these samples (Fig. 56, Appendix A) and a comparison with data obtained at the time of construction (Appendix A) it must be concluded that the subgrade had dried somewhat prior to sampling and also that the sampling operation itself resulted in densification of the "undisturbed" samples. Accordingly, the data for the first undisturbed sampling have not been used in the analysis.

Because of this discrepancy, additional undisturbed samples were taken after the completion of the plate load tests at the surface in sections 1 and 4. These samples should be

more representative of the conditions existing at the time of the plate load tests on the surface of the asphalt concrete. Results of laboratory repeated load tests on these samples are presented in Fig. 57. Also shown in the figure are the corresponding dry densities and water contents. By comparing the dry densities with those presented in Appendix A it will be noted that values are in better agreement than are those for the first sampling. In addition 100 percent sample recovery was obtained in each of the tubes, thus substantiating this hypothesis. It is interesting to compare these results with those in Fig. 54 for the laboratory prepared samples in the preliminary studies. For the same deviator stress and water content, approximately the same modulus values were obtained in both instances.

Untreated Aggregate Base. In Fig. 58 is presented a relationship between resilient modulus and confining pressure for the untreated aggregate base in a dry condition. The form of this relationship is the same as that obtained for granular materials in other investigations (3,5). In this instance the resilient modulus is that based on a deformation determined at 500 stress applications and the resulting relationship between modulus and confining pressure is:

$$M_R = 13,500 \times \sigma_3^{0.495} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

It will also be noted that the data obtained from tests on the 4 in. diameter specimens correspond to the data obtained for the 6 in. diameter specimens. Thus it would appear that the resilient modulus was not affected by the replacement of 1-1/2 in. to 3/4 in. material by aggregate in the 3/4 in. to No. 4 range. In terms of laboratory operations this should prove to be a considerable advantage.

Results of tests for the partially saturated specimens (4 in. diameter) are also shown in Fig. 58. It will be noted, however, that while the data exhibit the same slope, at a particular confining pressure, the modulus of the material is less in a partially saturated condition than when dry. The equation of this line is:

$$M_R = 9300 \times \sigma_3^{0.495} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Fig. 59 contains the same data plotted in terms of the sum of the principal stresses (i.e., in this case $\sigma_1 + 2\sigma_3$). The equations for this relationship for the partially saturated material is:

$$M_R = 2835\theta^{0.587} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

where:

θ = sum of principal stresses $(\sigma_1 + 2\sigma_3)$

Asphalt Concrete. Beams sawed from the pavement were tested in repeated flexure over a range in stresses selected to correspond to those estimated to occur in the pavement during the field load tests. At 68°F, the stresses ranged from 45 to 90 psi while at 41°F they ranged from 150 to 250 psi. For the range of stresses at both temperatures the stiffness (analogous to a modulus (4)) was determined to be sensibly independent of stress level. Accordingly, average stiffness at a time of loading of 0.1 sec. vs. temperature data are presented in Fig. 60.

To determine stiffnesses corresponding to the time of loading used in the field plate tests, 0.25 sec., use was made of the technique developed by Heukelom and Klomp (30). Table 8 presents a summary of the resulting computations.

Essentially the modified flexural stiffness values at 0.25 sec. loading time were determined from:

$$\text{Mod. flexural stiffness (0.25 sec.)} = \text{Measured flexural stiffness (0.1 sec.)} \times \frac{\text{Computed stiffness (0.25 sec.)}}{\text{Computed stiffness (0.1 sec.)}}$$

The modified stiffness vs. temperature relationship is shown in Fig. 61. In comparing these results with those in Fig. 60 approximately a 30-40 percent reduction in stiffness occurred for a change in loading time from 0.1 to 0.25 sec. While this change should not have a significant influence on computed deflections and stresses, the modified values were used in the analyses to be presented subsequently.

ANALYSIS OF PAVEMENT SECTIONS

This section will present the results of analyses to predict the response of the various test sections to load using laboratory and field measured properties and appropriate theory; as noted earlier the pavement response which has been chosen in deflection. Analyses will be presented both for sections consisting of untreated aggregate base and subgrade (two-layer systems) and asphalt concrete, untreated base, and subgrade (three-layer systems). Thus the test road provides an excellent opportunity, as noted earlier, to assess the validity of previously developed techniques for predicting pavement response for a roadway constructed with conventional methods and control.

Two-Layer Systems

To analyze the two-layer sections consisting of subgrade and aggregate base, the method developed by Seed et al (3) was utilized. In this method lateral and vertical stresses are computed by assuming a Boussinesq stress distribution. Approximate values for deflections in the two-layer structure may then be calculated using this stress distribution together

with modifications suggested by Vesic (31) to provide for the variability in deformation characteristics of the granular material. Only a very brief outline of the procedure will be provided and no sample calculations are included. Details of the procedure may be found in Reference (3).

In this analysis the modulus of resilient deformation of the base-course under the loaded plate is assumed constant along a horizontal plane. Variation of modulus in the vertical direction is approximated by subdividing the base-course into a number of horizontal layers and computing a modulus for each layer. Such variation in modulus results since the modulus of the granular material is dependent on stress and the stress varies with depth.

Procedure. For the analysis and prediction of deflections for the repeated plate load tests the following procedure has been followed:

1. The base course and subgrade were divided into several horizontal layers to a depth (from the surface of the base) four times the radius of the plate.

2. At a depth corresponding to the center of each horizontal layer in the base, the horizontal normal stresses resulting from the applied load were computed using the solution for stresses presented by Ahlvin and Uhlery (32). At the same points, lateral pressures resulting from the weight of the material above were also determined using a coefficient of earth pressure at rest, Ko, equal to 0.5. These stresses were computed along a vertical line offset a radial distance from the centerline 0.7 times the plate radius for Poisson's ratios of 0.35 and 0.50.

3. Stresses computed in the previous step for each horizontal element were added together to obtain the confining pressure (σ_3) on the element.

4. Corresponding to a particular value of σ_3 the modulus was then obtained from the relationship $M_R = K \times \sigma_3^n$.

5. The vertical stresses induced in the subgrade by the applied load were computed at the midpoint of each horizontal element under the center of the plate. These vertical stresses are assumed to be the principal stress differences or deviator stresses applied to the subgrade.

6. Using these stresses, the moduli of the various layers of the subgrade were then estimated from the modulus vs. applied stress relationships obtained in the field plate load tests (Fig. 44). It should be noted that the laboratory test results (e.g., Fig. 54) could be used for this determination.

7. The resilient deformation of each horizontal layer in the subgrade was then computed using the deflection factors presented by Ahlvin and Uhlery (32) and the appropriate modulus

previously determined. It should be emphasized that this deflection was determined along the vertical axis through the center of the plate.

8. By summing the deflections in each of the layers, the resilient deflections of the base, the subgrade, and the two-layer system were estimated.

Resilient Deformation of the Base. Using this analysis, resilient deflections were determined for the base-course corresponding to the three plate sizes and three pressures for each plate. As previously noted, data were not available relative to possible material differences (i.e., aggregate grading, water content, and dry density) at each plate load test location; accordingly, variations in resilient deformation of the base will be regarded as experimental scatter without further speculation. The laboratory data presented in Fig. 58 were used to represent the modulus vs. confining pressure relationships for the base in the dry and partially saturated conditions. Utilizing the relationship for partially saturated specimens (i.e., $M_R = 9300 \times \sigma_3^{0.495}$) deformations for the aggregate base-course corresponding to various plate sizes and pressures were computed for Poisson's ratios of 0.35 and 0.50. Results of these computations are plotted on Fig. 62 for the three plate sizes. The same calculations were also made utilizing the modulus relationship for dry aggregate base ($M_R = 13,500 \times \sigma_3^{0.495}$) for all plate sizes with a Poisson's ratio of 0.35 and additionally for the 12 in. diameter plate with a Poisson's ratio of 0.5. Also shown in this figure are the resilient deformations of the aggregate base obtained from field plate load tests for the 500th stress application. These measured values for the resilient deformation of the base were obtained by subtracting the total resilient deformation from the resilient deformation of the base-subgrade interface.

In examining Fig. 62 it appears that the majority of the field data are encompassed by the computed data based on modular values obtained for the partially saturated condition.

Resilient Deformation of the Subgrade. Resilient deformations at the top of the subgrade were computed for the 12-in. diameter plate for three sections; section 3 was selected to represent a condition of low resilient modulus, section 1 an intermediate resilient modulus, and section 5 a high resilient modulus. Values for resilient modulus used in analysis were obtained from field plate load tests at the surface of the subgrade, Fig. 44. Results of these computations are shown in Fig. 63; a Poisson's ratio of 0.5 was assumed for the subgrade. Also shown in the figure are the measured results. In general reasonable agreement between computed and measured values was obtained.

Total Resilient Deformation. Total resilient deformations were calculated for sections 1, 3, and 5. In Fig. 65 the measured resilient deformations are compared to the computed

deformations which were obtained by adding the subgrade deformations shown in Fig. 64 to those for the base (obtained for partially saturated conditions and a Poisson's ratio of 0.35. Reasonable agreement is noted between measured and computed values.

Summary. While the two-layer system which has been analyzed in this section may not represent a realistic pavement structure for a heavy duty highway, the reasonable agreement between computed and measured deformations lends credence to the method developed by Seed et al (3). The fact that deformations for an actual structure constructed using standard procedures could reasonably be estimated from the results of laboratory tests and available theory appears to the authors, at least, a forward step in the improved design and analysis of pavement structures.

Three-Layer Systems

Analyses of the three-layer structures consisting of subgrade, untreated aggregate base, and asphalt concrete surface were accomplished using two procedures. The majority of the analyses made use of an adaptation of the finite element technique (33,34); a few loading conditions were also analyzed using multilayer elastic theory (5).

Finite Element Method. An adaptation of the finite element procedure developed by Duncan et al (35) for pavement structures was used herein.

For analysis by the finite element technique, the body to be analyzed, such as the cylinder shown in Fig. 65, is divided into a set of elements connected at their joints or nodal points. On the basis of an assumed variation of displacements within elements (i.e., linear, parabolic) together with the stress-strain characteristics of the element material, the stiffness of each nodal point of each element is computed. For each nodal point in the system, two equilibrium equations may be written expressing the nodal point forces in terms of the nodal point displacements and stiffnesses. These equations are then solved for the unknown displacements. With the displacements of all nodal points known, strains and stresses within each element are then computed. Detailed descriptions of the method and its application to a wide variety of problems are contained in a number of recent publications (e.g. 33,34). Analysis of realistic systems commonly requires formulation and solution of several hundred simultaneous equations. For this reason the technique is only practicable when formulated for high-speed digital computers.

The digital computer program used for the present study is described in reference (34). Modifications have been made to this program to automatically generate suitable finite element configurations for analysis of axisymmetric pavement structures and to accommodate types of modulus dependency on stress which would appear appropriate to represent

the behavior of granular base and cohesive subgrade materials under conditions corresponding to moving traffic (e.g., Fig. 57 and 58).

For analysis with this computer program, the structure to be analyzed is divided into a series of quadrilaterals. Each quadrilateral is subsequently divided into four triangles by the computer program as shown in Fig. 65. Displacements are assumed to vary linearly within each triangle; this assumption insures that no gaps will develop in the deformed structure and that displacements will be compatible throughout the structure as well as at the nodal points.

Besides the finite element configuration to be used, additional items of input consist of specifying loads or displacements for each nodal point and material properties (elastic modulus and Poisson's ratio) for each element. In the nonlinear analyses, an initial gravity stress (corresponding to no applied load on the pavement) was calculated for each element, requiring that the density of each material be specified as well. The computer output consists of radial and axial displacements at each of the nodal points exterior to the quadrilateral elements, and the complete state of stress at the centroid of each quadrilateral. Quadrilateral stresses are computed as the average of the stresses in the four triangles into which they are divided by the program.

The plate loading tests at the surfaces of the various test sections would appear to lend themselves to analysis by this procedure. Accordingly configurations such as that illustrated in Fig. 66 were utilized for plate tests at the pavement surface for Sections 1 and 4.

In the analysis, modulus of resilient deformation of the subgrade was idealized as a bilinear material by means of expressions of the form

$$M_r = K_2 + K_3 \, [K_1 - (\sigma_1 - \sigma_3)] \qquad K_1 > (\sigma_1 - \sigma_3)$$

$$M_r = K_2 + K_4 \, [(\sigma_1 - \sigma_3) - K_1] \qquad K_1 < (\sigma_1 - \sigma_3) \qquad \ldots\ldots\ldots (5)$$

The relationship of the constants K_1, K_2, K_3, and K_4 to the measured properties is illustrated in Fig. 67 and is a simplified representation of the data shown in Fig. 57. Modulus of resilient deformation of the base-course was represented by equation (3) for partially saturated base illustrated in Fig. 58. For the range of stresses investigated, the stiffness of the asphalt concrete appeared to be independent of stress level; data illustrated in Fig. 61 were thus used to represent the time and temperature dependent response of the asphalt concrete.

Briefly the procedure that has been utilized may be outlined as follows:

1. Idealization - The pavement section is divided into a system of rectangular finite elements (e.g., Fig. 66).

2. Element analysis - Stiffness matrices which relate element nodal point forces and displacements of the elements are evaluated (load and stiffness values used in the first approximation are based on linear material properties).

3. Assembly of elements - The nodal stiffness matrix for the entire structure is formed by superimposing the appropriate individual element stiffnesses.

4. Displacement analysis - Nodal displacements which result from the applied nodal forces are determined by solving nodal equilibrium equations expressed in terms of the structural stiffness matrix.

5. Stress analysis - Element stiffness matrices are used to compute the element stresses which result from nodal displacements computed in step 4.

6. Successive approximations - Steps 2 through 5 are repeated using elemental moduli based on stresses computed in step 5 since the moduli are dependent on stresses. This procedure is repeated until compatibility between stresses and moduli are obtained.

As noted earlier the analysis has been programmed for solution on the digital computer (35) and a simplified flow diagram of the process is illustrated in Fig. 68.

Results of a series analyses by this procedure are presented in Figs. 69 through 72. Table 9 provides a summary of the configurations used in the analyses and Table 10 indicates the results of the iterative procedure for analyses performed for the tests on section 1.

In most instances the computed deflections are somewhat larger than the measured deflections although the agreement is reasonable considering some of the problems associated with obtaining proper material characteristics. For example it is interesting to note that the computed subgrade deflections are higher than those actually measured. This difference could be due in part to the use of a single relationship between deviation stress and modulus (e.g., Fig. 57) for each of the two sections, namely 1 and 4. It is possible that the entire subgrade had not increased in water content to the extent indicated by the data in Fig. 57 at the time of the plate load tests. If this were the case then the computed subgrade deflections would be larger than those measured (e.g., comparison of the moduli data in Figs. 57 and 55). Some indication of this influence will be presented in the next section.

Multilayer Elastic System Analysis. A few of the loading conditions were also analyzed using the computer solution for a multilayer elastic system developed by the Chevron Research Company (36). This particular program has been modified to incorporate an iterative procedure taking into account the nonlinear response observed for untreated aggregate bases and cohesive subgrade soils (6) such as that illustrated in Figs. 57 and 59.

For the first solution an analysis of the 12-in. diameter plate load test at the surface of the 7.2 in. asphalt concrete layer corresponding to a pressure of 80 psi was made in section 4. The configuration used in the analysis as well as the moduli resulting from the iteration process are shown in Fig. 73. Moduli for the untreated aggregate base were obtained from the data presented in Fig. 59 while the subgrade response corresponded to that shown in Fig. 57. Stiffness of the asphalt concrete was obtained from the data presented in Fig. 61. The resulting computed deflections were:

Computed Deflection-in.			Measured Deflection-in.		
Surface	Subgrade	Base	Surface	Subgrade	Base
0.045	0.038	0.007	0.017	0.011	0.006

While the computed base deflection is quite comparable, it will be noted that the computed total deflection is considerably larger. It would thus appear that the modulus used for the subgrade was too low and could be in part due to the fact that only the upper portion may have increased in water content to the values shown in Fig. 57.

Accordingly, additional analyses were performed by considering the pavement as a four-layer system using a single modulus for the untreated base and subdividing the subgrade into two layers, the upper layer of which was 12 in. thick. Data for the lower portion of the subgrade were obtained from the plate load tests performed directly on the surface of the subgrade (Fig. 44) and which were substantially larger than those shown in Fig. 57. The resulting computations are shown in Table 11. In this table it will be noted that as the lower portion of the subgrade is stiffer the deflection is reduced. Moreover, considering that the deflection under a rigid plate may be somewhat less than that determined under a flexible plate (for which the computations were made), the resulting data shown in Table 11 appear quite reasonable.

These results, like those for the finite element procedure, indicate that suitable theory does have the potential to predict deflections reasonably well but emphasizes that proper material characteristics must be utilized in order that meaningful results be obtained.

SUMMARY AND CONCLUSIONS

The Via de Mercados test road was developed to provide information in two areas, namely:

1. To determine the influence of field compaction methods (or equipment) together with compaction conditions (in this case water content at a specific dry density) on the resilience characteristics of a fine-grained subgrade soil and the effect, in turn, of these characteristics on the surface deflection of an asphalt concrete pavement structure.

2. To verify procedures previously developed and checked on prototype pavements for prediction of pavement response to load utilizing laboratory tests and appropriate theory.

Because of the difficulties encountered during the actual construction of the test road, it was not possible to develop satisfactory data to provide information to satisfy the first objective. From the material presented, however, it would appear that the second objective has been met. Accordingly, a majority of the conclusions obtained from the results of the investigation are directed to this latter aspect.

As a result of the investigations, the following conclusions appear warranted:

1. It is possible to study the resilient characteristics of compacted soils and their influence on the resilient response to load of pavement sections by means of a test road. Moreover, since little data are available on the properties of field compacted soils and the influence of compaction equipment on these properties, such studies are extremely desirable. From the experience obtained in the construction of this test road, however, more detailed construction planning and allowance for variations in weather conditions must be considered than that exercised in the investigation in order to obtain the desired results. This suggests that it might be desirable to develop the same type of investigation which was reported herein on a smaller scale initially within a controlled environment.

2. A number of agencies, each with certain capabilities and interests, can work together to examine jointly many of the pressing problems concerned with the design and construction of pavements.

3. The results of field plate load tests on the various components of the test road exhibit characteristics similar to those obtained from the results of tests on prototype pavements, e.g.:
   a. The resilient modulus of fine-grained soils is dependent on applied stress, increasing in value in the low stress range.

b. The modulus of granular materials is dependent on stress, the stress dependence of which results in a nonlinear response of pavements to load at least in the low stress range.

4. The results of repeated load test on dry granular material containing 1-1/2 in. maximum size aggregate can be duplicated reasonably well by replacing material in the 1-1/2 in. by 3/4 in. by No. 4 size range. This finding would appear to be significant in that the work involved in testing base course materials can be reduced somewhat since specimens 4 in. in diameter by 8 in. high rather than 6 in. in diameter by 12 in. high can be used to measure resilient response.

5. The resilient response of granular materials in drained repeated load tests is affected by the water content at the time of testing. An indication of this influence may be obtained from a comparison of the results of the modulus vs. deviation stress relationships obtained for the dry condition and for the material at a water content of 7 percent:

   Dry: $M_R = 13,500 \times \sigma_3^{0.495}$

   Partially saturated: $M_R = 9300 \times \sigma_3^{0.495}$

   In effect, approximately a 30 percent reduction in modulus was obtained.

6. Within the limits of variability that can result from normal construction procedures, it would appear that the results of laboratory tests and appropriate theory can be successfully used to predict the response of an actual pavement structure to load. However, it must be emphasized that for such techniques to be successful laboratory determined characteristics must reflect those existing in-situ.

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TABLE 1 — COMPACTION REQUIREMENTS FOR HIGHWAY AND AIRFIELD PAVEMENTS BASED ON SPECIFIC LOADING CONDITIONS**

Compaction %	Cohesionless Soils*			Cohesive Soils		
	Compaction Index	Thickness, in.		Compaction Index	Thickness, in.	
		18,000-lb. Axle	DC-8		18,000-lb. Axle	DC-8
105	42	--	9	--	--	--
100	9	10	32	19	6	17
95	3.5	17	61	8.6	10	33
90	1.8	27	92	5.0	14	49
85	--	--	--	3.2	18	63
80	--	--	--	2.4	22	79

* PI = 0.
**After Corps of Engineers(14)

TABLE 2 — APPROXIMATE RELATIONSHIP BETWEEN TOTAL EXPANSION AND PLASTICITY INDEX

Plasticity Index	Swelling Potential %	Total Expansion %
10	0.4 - 1.5	4.5 - 10.0
20	2.2 - 3.8	13.5 - 18.7
30	5.7 - 12.2	21.4 - 28.0
40	11.8 - 25.0	28.0 - 35.0
50	20.1 - 42.6	33.0 - 40.0

(After Seed et al (15))

TABLE 3 — COMPARISON OF RESPONSE OF UNDISTURBED AND LABORATORY PREPARED SAMPLES IN RESILIOMETER*

Material		Specimen	Water Content %	Dry Density 16 per cu. ft.	Resiliometer Test Data									
					Vertical Surcharge psi	Resilience - cu. in. at pressures (psi) of:								
Liquid Limit	Plasticity Index					3	5	1	15	20	25	30	40	50
36	15	Undisturbed	15.5	116.0	0.4			.085		.215		.310	.400	.510
		Laboratory Comp.	15.5	117.5	0.3			.050		.080		.125	.150	.190
		Undisturbed	17.7	108.5	1.3			.085		.320		.530	.690	.775
		Laboratory Comp.	17.7	110.0	1.1			.050		.155		.205	.330	.410
27	9	Undisturbed	14.7	117.0	0.5			.095		.180		.295	.400	.500
		Laboratory Comp.	13.9	122.8	0.4			.140		.420		.470	.550	.640
31	13	Undisturbed	15.0	119.0	0.5			.175		.340		.345	.375	.415
		Laboratory Comp.	16.2	113.5	0.5			.390		.605		.870	1.030	1.130
33	14	Undisturbed	14.9	120.0	0.7			.095		.195		.225	.235	.270
		Laboratory Comp.	15.8	116.0	0.8			.245		.455		.580	.660	.750
		Undisturbed	17.3	113.0	1.3			.100		.205		.300	.350	.395
		Laboratory Comp.	16.9	116.0	1.3			.171		.415		.605	.756	.848
46	25	Undisturbed	23.4	97.0	1.6	.020	.070	.180						
		Laboratory Comp.	23.4	96.5	1.5	.020	.040	.170	.335	.475	.625	.765	.920	.820
57	38	Undisturbed	21.0	102.0	1.7			.150		.325		.455	.550	.650
		Laboratory Comp.	18.5	108.7	1.7			.290		.485		.695	.825	.865
51	26	Undisturbed	23.6	99.5	1.2			.165		.390		.465	.520	.625
		Laboratory Comp.	24.2	99.5	1.2			.130		.295		.445	.515	.590
51	25	Undisturbed	34.5	85.5	1.5			.080		.160		.190	.230	.270
		Laboratory Comp.	33.4	85.6	2.3			.235		.415		.685	.855	.955

TABLE 4 — TEST RESULTS FOR SILTY CLAY SUBGRADE SOIL

Specific Gravity	2.77
Atterberg Limits:	
Liquid Limit	32.5 percent
Plastic Limit	17.2 percent
Plasticity Index	15.4
Design "R" value*	8

*State of California Test Method No. 301

TABLE 5 — TESTS RESULTS FOR UNTREATED AGGREGATE BASE

Specific Gravity:	
Retained in 3/4 in. sieve	2.81
Passing 3/4 in. sieve	2.78
Sand Equivalent*	44
Maximum Dry Density**	148.7 lb. per cu. ft.
Optimum Water Content**	8 percent

* R value requirement waived for class 2 aggregate with a sand equivalent greater than 35.

**Based on State of California Test Method 216-F for material passing 3/4 in. sieve.

TABLE 6 — SUMMARY OF PLATE SIZES AND PRESSURES FOR REPEATED PLATE LOAD TESTS

Test Series	Plate diameter in.	Plate pressure -psi
Subgrade	30	0.5, 1.5, 3.0
	24	2.5, 4.0, 7.0
	18	5.5, 8.0, 10
Two-Layer System (Aggregate base and subgrade)	18	5.0, 10, 20
	12	10, 25, 40
	8	10, 25, 40
Three-Layer System* (Asphalt concrete, aggregate base, and subgrade)	12	20, 40, 60, 80
	8	25, 50, 75, 100

*Two test series, one at the surface of 2.4 in. of asphalt concrete (first lift) and one at the surface of 7.2 in. of asphalt concrete.

TABLE 7 — BENKELMAN BEAM REBOUND MEASUREMENTS* AT REPEATED PLATE LOAD TEST SITES

Section No.	Rebound Reading** - in.	
	At surface of 2.4 in. asphalt concrete	At surface of 7.2 in. asphalt concrete
1	0.066	0.024
2	0.027	0.015
3	0.034	0.021
4	0.029	0.020
5	0.025	0.023
6	0.030	0.019
Average Air Temperature, °F	65	45

* 15,000 lb. axle load, 70 psi tire pressure.
**CGRA rebound procedure.

TABLE 8 – COMPARISON BETWEEN MEASURED FLEXURAL STIFFNESS AND COMPUTED STIFFNESS FOR TIMES OF LOADING OF 0.25 SEC. AND 0.1 SEC.

Paving Course	Asphalt Content %	Air Voids %	Pen. @ 77°F dmm[a]	So. Pt.[a]	Temp. °F	Duration of Loading sec.	Measured[b] Stiffness psi × 10^5	Computed[c] Stiffness psi × 10^5	Mod.[d] Stiffness psi × 10^5
Top	5.5	8.6	57	119	41	0.1	11.29	8.15	
						0.25		5.83	8.08
					68	0.1	3.20	1.54	
						0.25		0.98	2.02
Middle	5.5	7.3	47	121	41	0.1	11.59	8.79	
						0.25		6.53	8.62
					68	0.1	3.63	1.79	
						0.25		1.31	2.30
Bottom	5.0	13.8	57	118	41	0.1	9.38	5.77	
						0.25		4.03	6.61
					68	0.1	1.06	0.98	
						0.25		0.65	0.69

a. Tests performed by California Division of Highways, Materials and Research Laboratory personnel.
b. Laboratory determined flexural stiffness values which are presented on Fig. 60.
c. Computed stiffness using Shell procedure.
d. Modified flexural stiffness values which are presented on Fig. 61.

TABLE 9 — SUMMARY FINITE ELEMENT CONFIGURATION FOR ALL PAVEMENT SECTIONS

Test Section	Thickness of Pavement Section (for analysis) in.				Plate Diameter in.	Plate Thickness (for analysis) in.	Radius of Configuration	No. Nodal Points	No. Elements
	Total	A. C.	Base	Subgrade					
1	198.4	2.4	11	183	8	2	48	180	153
	298.2	7.2	11	280	12	2	70	239	208
4	200.2	2.4	11	182	8	2	48	202	173
	298.2	7.2	11	278	12	2	70	239	208

TABLE 10 — RESULTS OF COMPUTATIONS FOR TEST SECTION 1

Computed Resilient Deformations			
Iteration No.	Center & Top of Plate-in.	Top of Base-in.	Top of Subgrade-in.
8" Plate 100 psi 2.4" A. C.			
1	0.0341	0.0341	0.0259
2	0.0601	0.0592	0.0395
3	0.0425	0.0415	0.0298
4	0.0523	0.0516	0.0351
5	0.0453	0.0443	0.0313
6	0.0449	0.0442	0.0320
12" Plate 80 psi 2.4" A. C.			
1	0.0559	0.0555	0.0475
2	0.1006	0.0996	0.0778
3	0.0813	0.0802	0.0665
4	0.0878	0.0869	0.0692
5	0.0850	0.0838	0.0700
8" Plate 100 psi 7.2" A. C.			
1	0.0125	0.0121	0.0102
2	0.0129	0.0126	0.0102
3	0.0126	0.0123	0.0102
4	0.0128	0.0124	0.0103
5	0.0127	0.0123	0.0103
6	0.0127	0.0124	0.0103
12" Plate 80 psi 7.2" A. C.			
1	0.0222	0.0219	0.0188
2	0.0240	0.0237	0.0193
3	0.0226	0.0223	0.0189
4	0.0240	0.0236	0.0195
5	0.0227	0.0223	0.0189
6	0.0239	0.0236	0.0196

TABLE 11 — COMPUTED DEFLECTIONS FOR SECTION 4 CORRESPONDING TO 12 DIAMETER PLATE, 80 PSI PRESSURE FOR 7.2 IN. ASPHALT CONCRETE THICKNESS

Component	Modulus psi	Computed Deflection - in.		
		Surface	Subgrade	Base
Asphalt concrete	420,000			
Base	7,000	0.034	0.027	0.007
Subgrade[a] 1	2,700			
Subgrade 2	5,000			
Asphalt concrete	420,000			
Base	7,000	0.026	0.019	0.007
Subgrade[a] 1	2,700			
Subgrade 2	10,000			
Asphalt Concrete	420,000			
Base 7,000	0.022	0.015	0.007	
Subgrade[a] 1	2,700			
Subgrade 2	20,000			

(a) Thickness of upper layer of subgrade assumed to be 12 in.

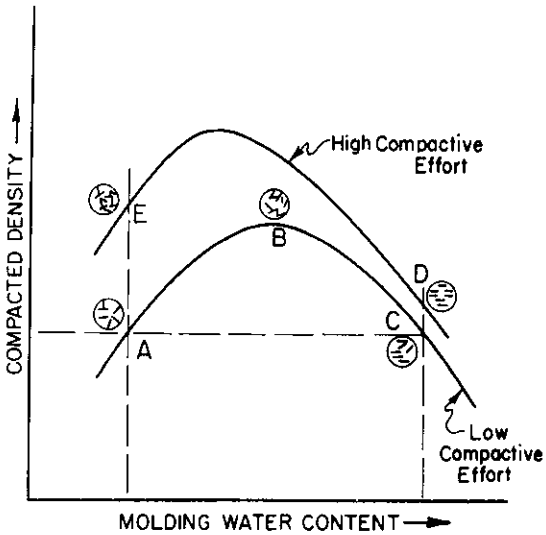


Fig. 1 — Effect of compaction on soil structure. (After T. W. Lambe.)

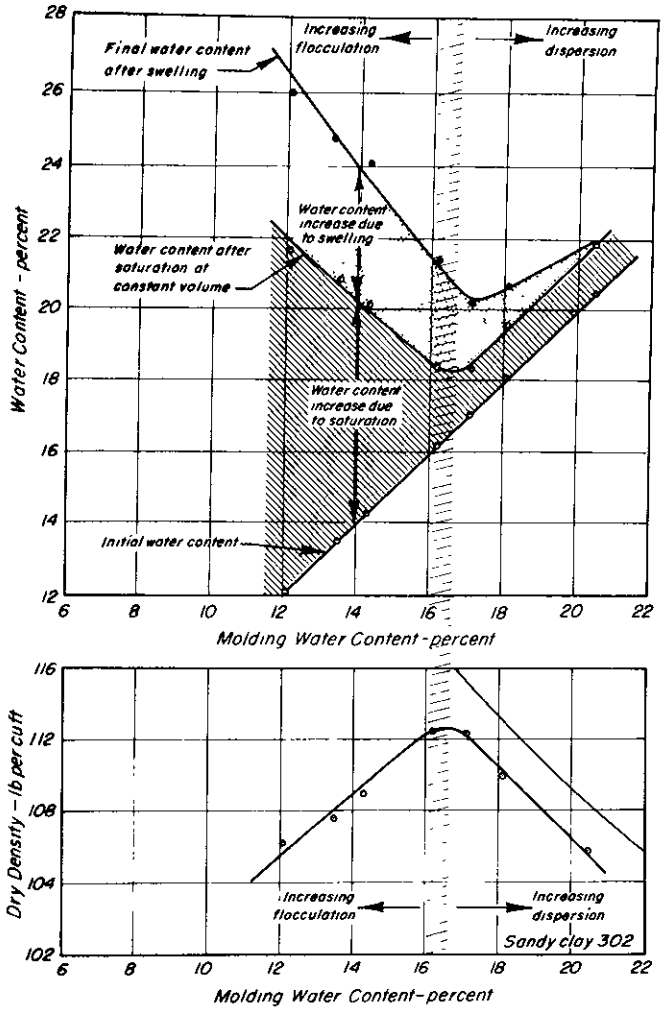


Fig. 3 — Influence of molding water content and soil structure on swelling characteristics of sandy clay. (After Seed et al.)

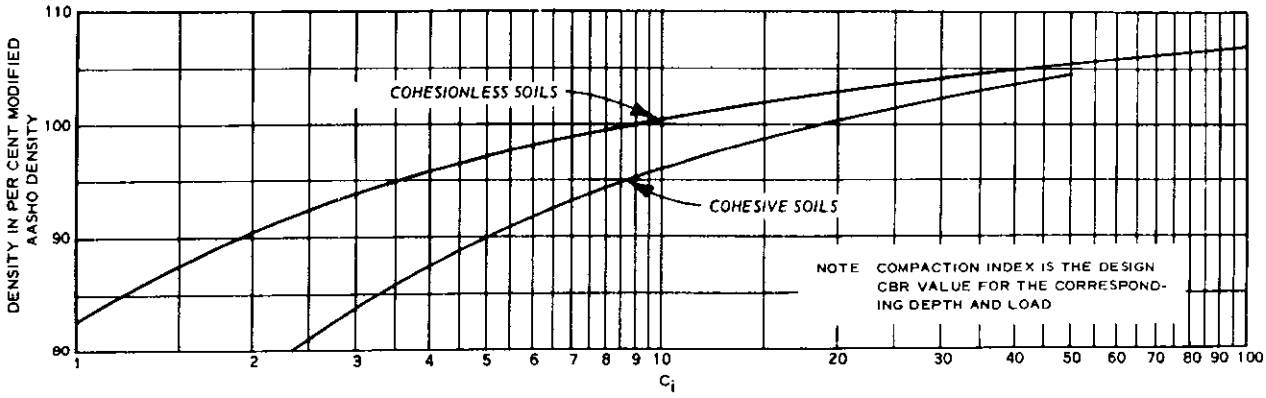


Fig. 2 — Compaction requirements for flexible pavements. (After Corps of Engineers.)

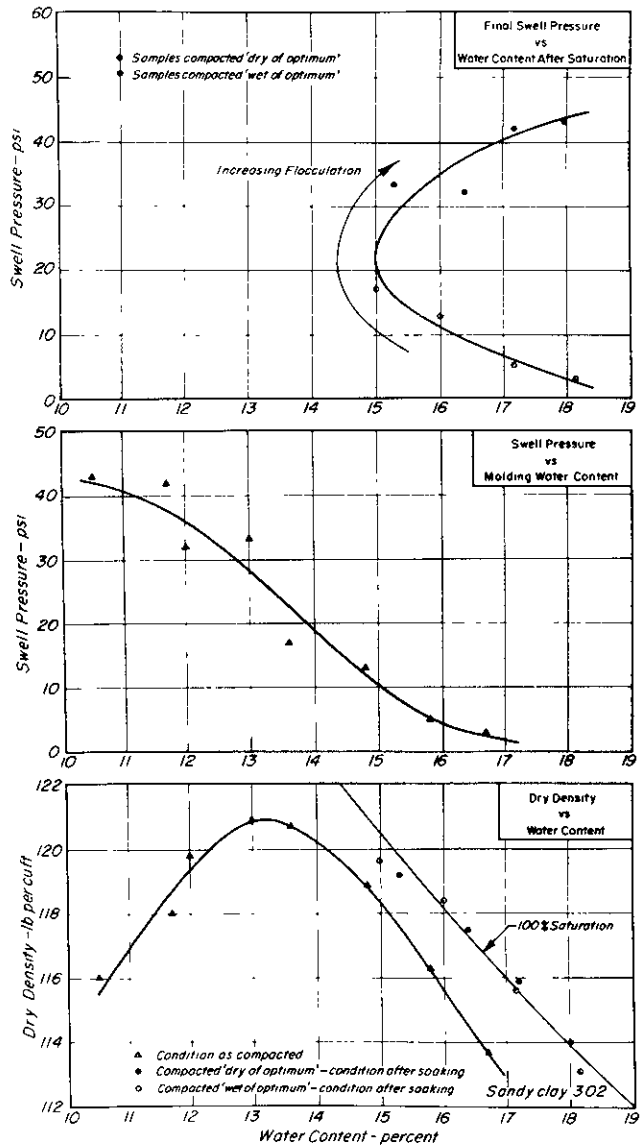


Fig. 4 – Influence of molding water content and soil structure on swell pressure of sandy clay. (After Seed et al.)

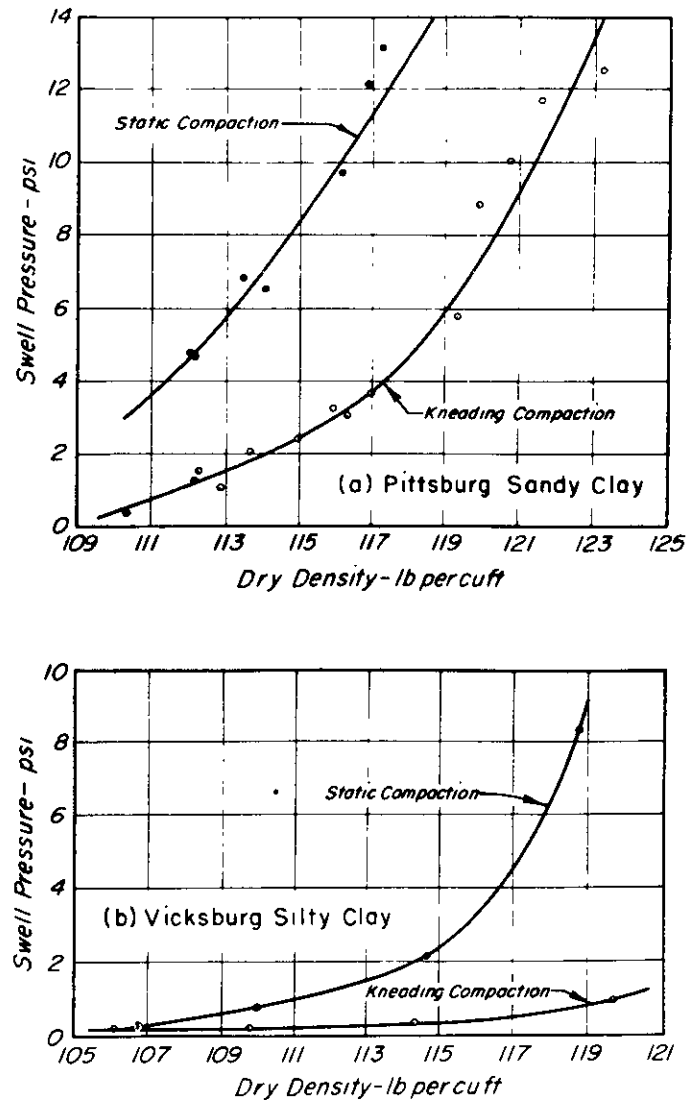


Fig. 5 – Effect of method of compaction on swell pressure for samples compacted to high degree of saturation. (After Seed et al.)

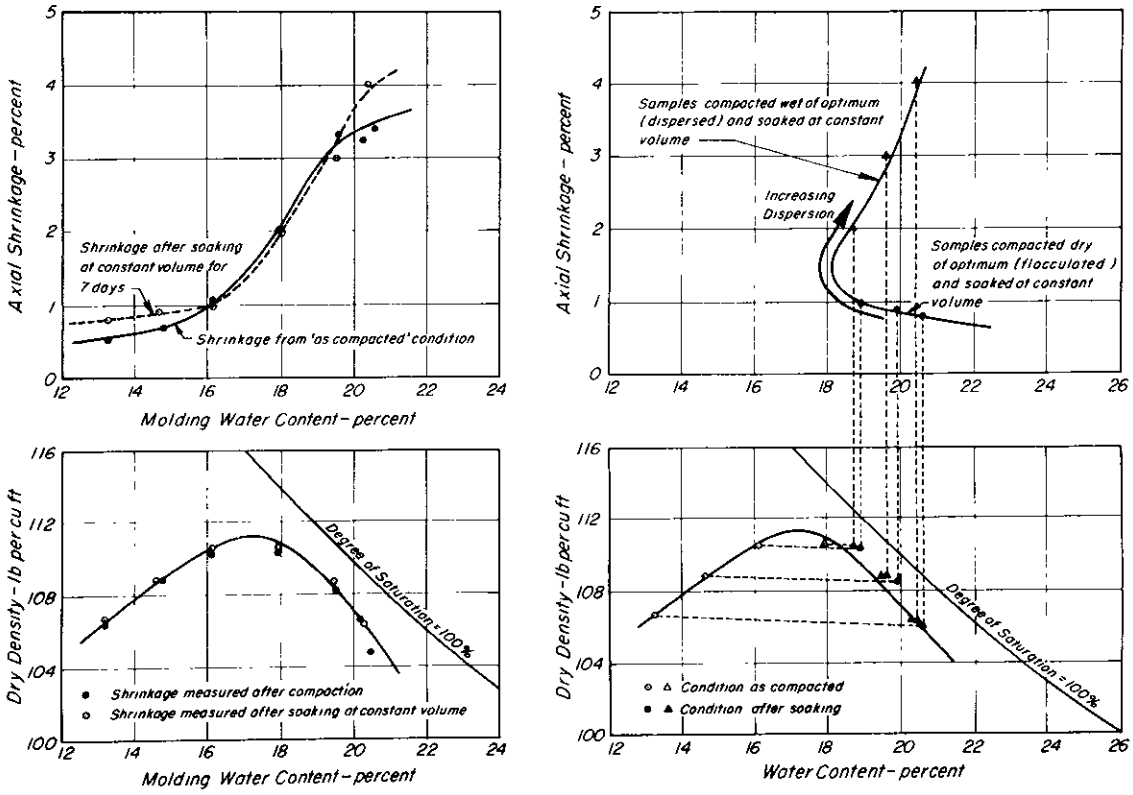


Fig. 6 – Influence of soil structure on shrinkage of silty clay. (After Seed et al.)

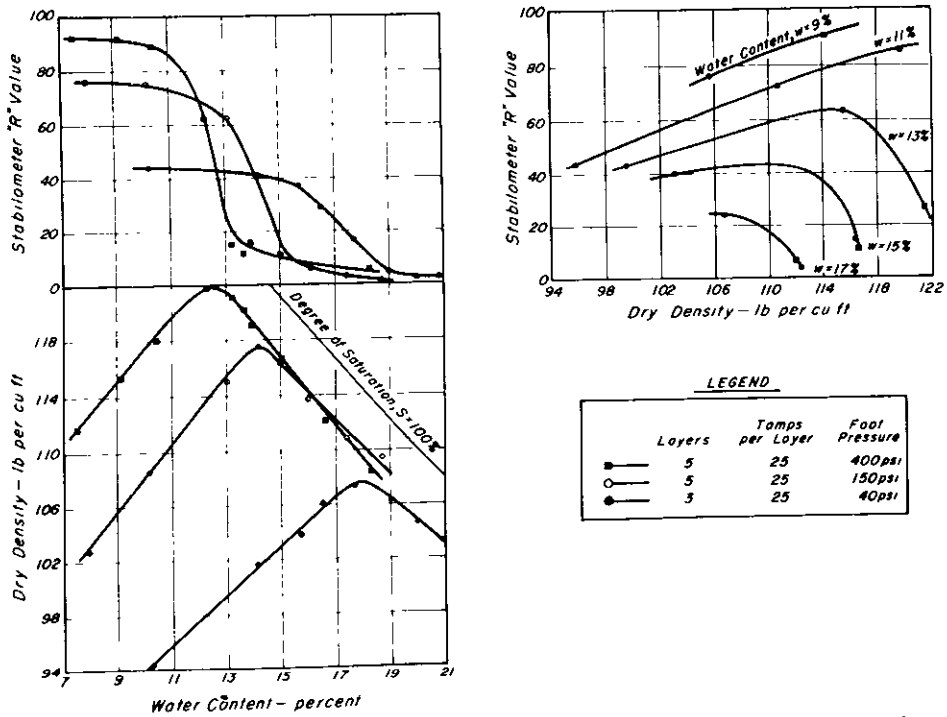


Fig. 7 -- Water content, density, and stability relationships for silty clay – kneading compaction. (After Seed et al.)

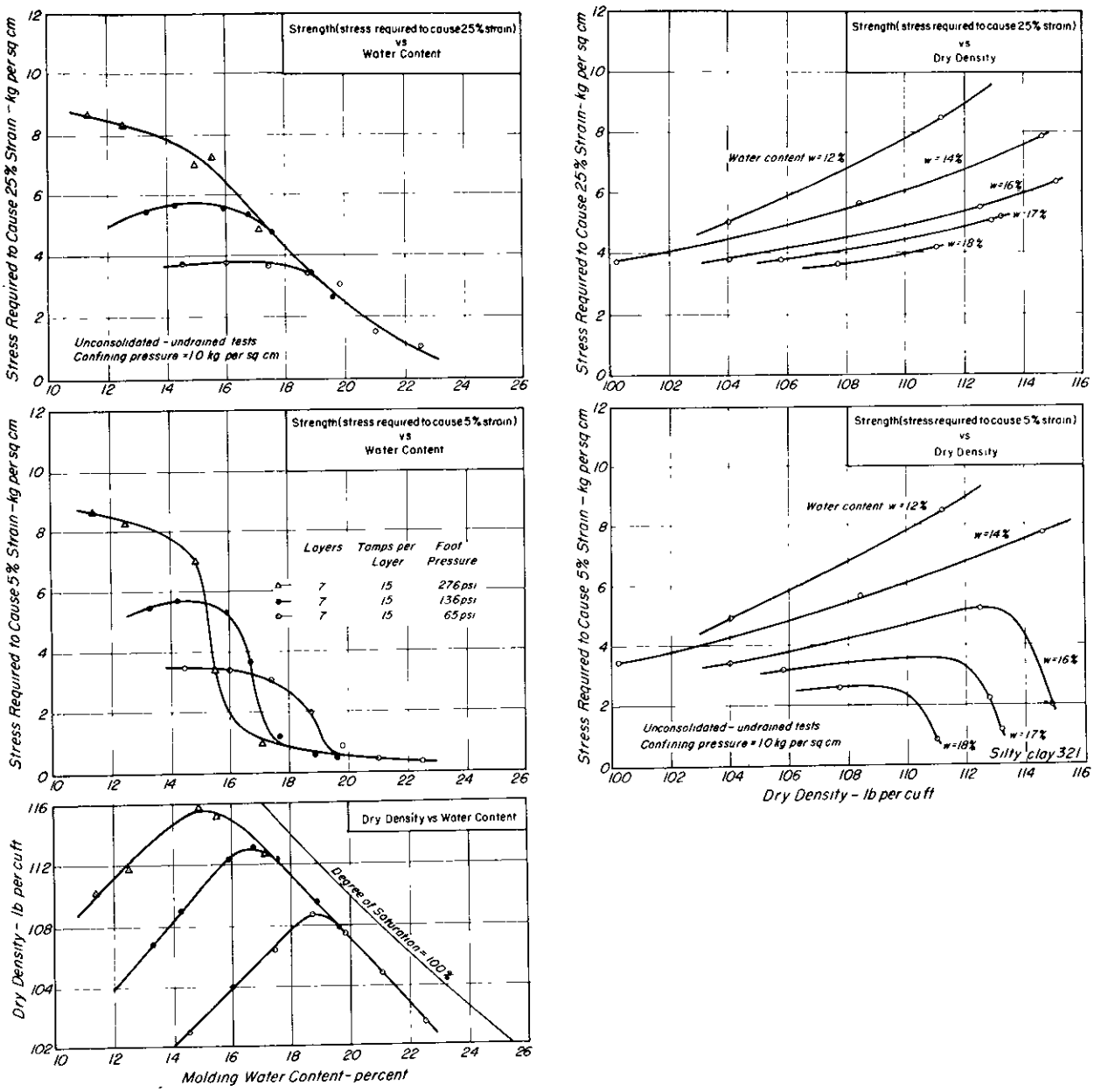


Fig. 8 — Relationship between dry density, water content and strength as compacted for samples of silty clay — kneading compaction. (After Seed et al.)

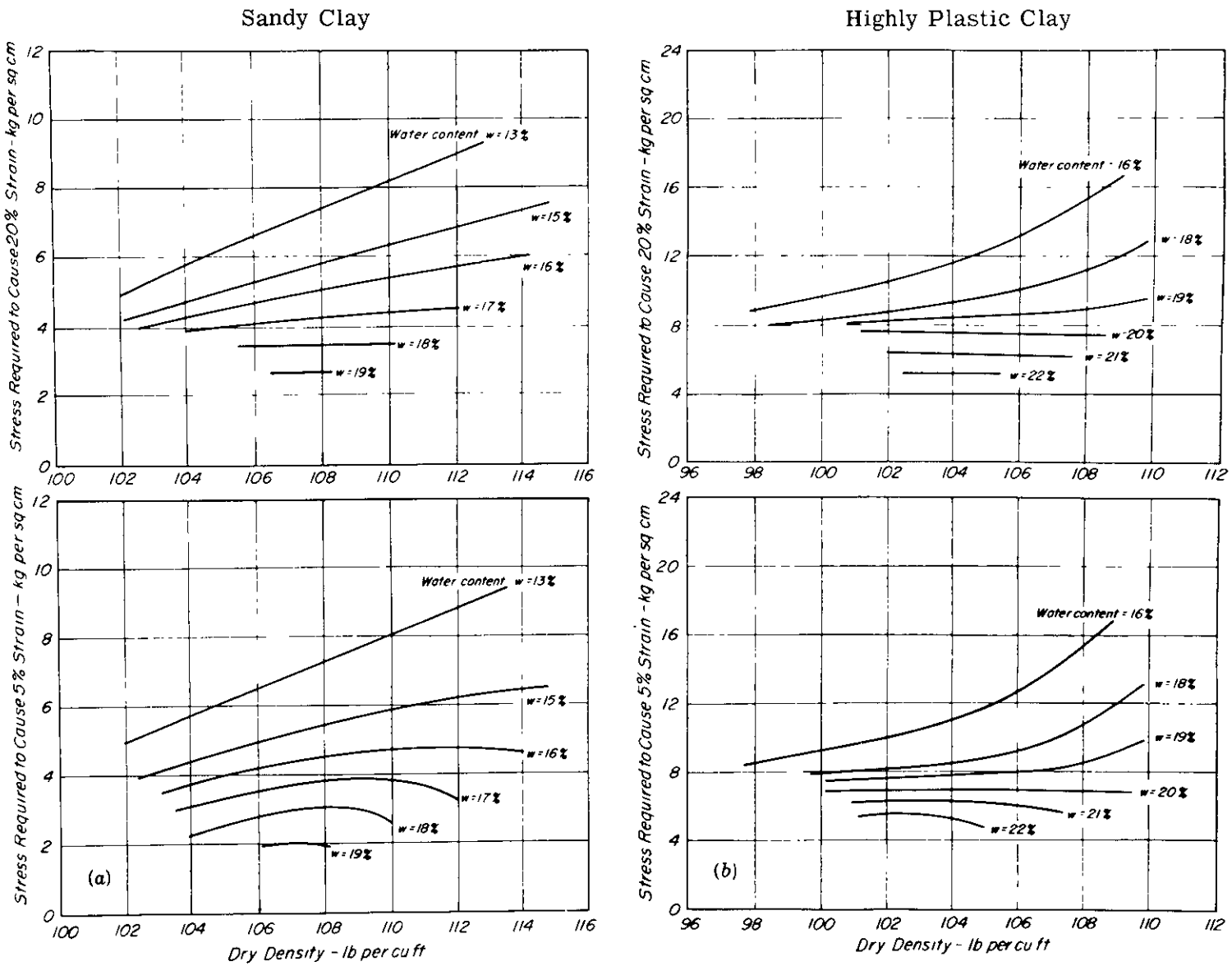


Fig. 9 – Relationship between dry density, water content, and strength as compacted for clay samples. (After Seed et al.)

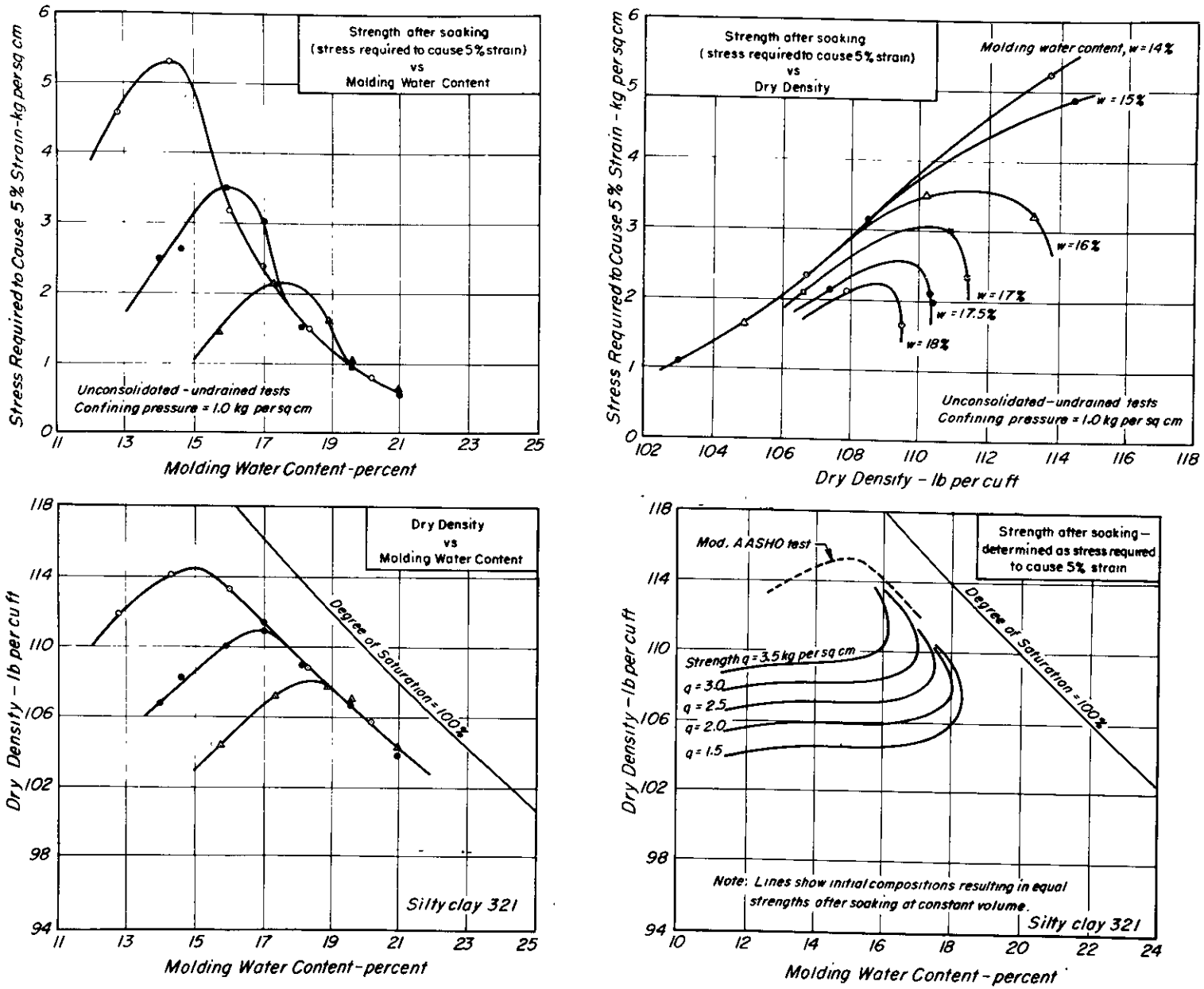


Fig. 10 – Influence of compaction conditions on strength after soaking at constant volume for silty clay prepared by kneading compaction (stress to cause 5% strain). (After Seed et al.)

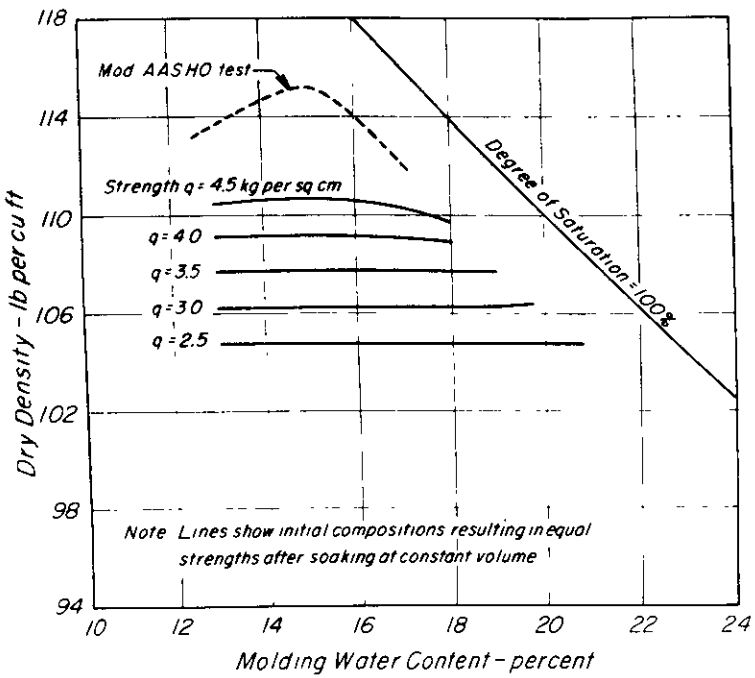


Fig. 11 – Strength after soaking at constant volume (stress to cause 20 percent strain.) (After Seed et al.)

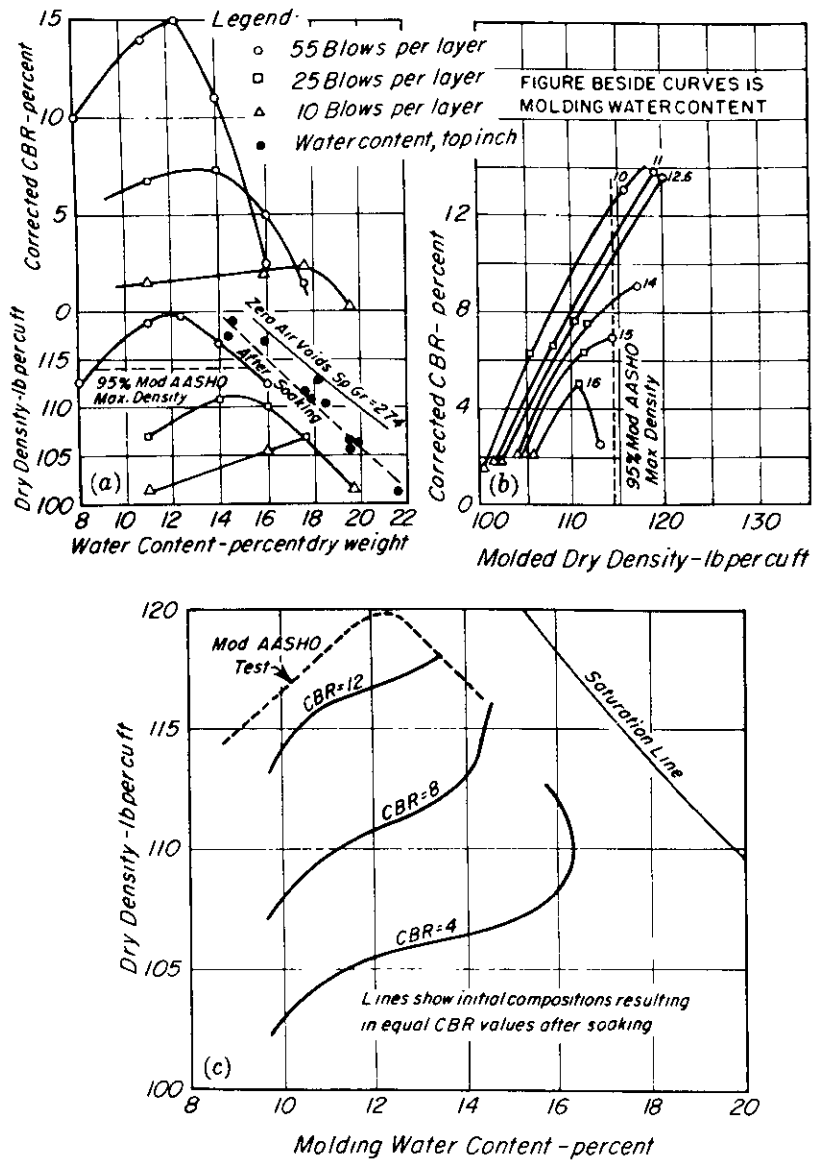


Fig. 12 – Relationship between initial composition and soaked CBR for samples of lean clay. (After Seed et al.)

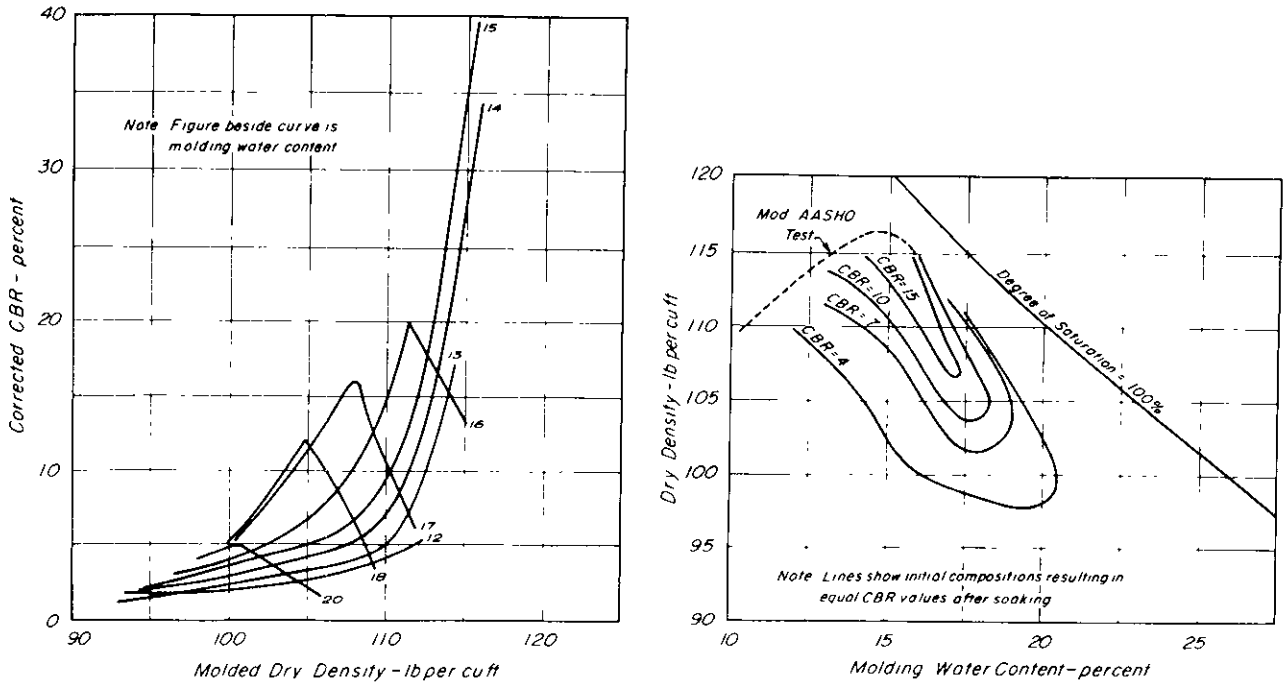


Fig. 13 -- Relationship between initial composition and soaked CBR for samples of silty clay. (After Seed et al.)

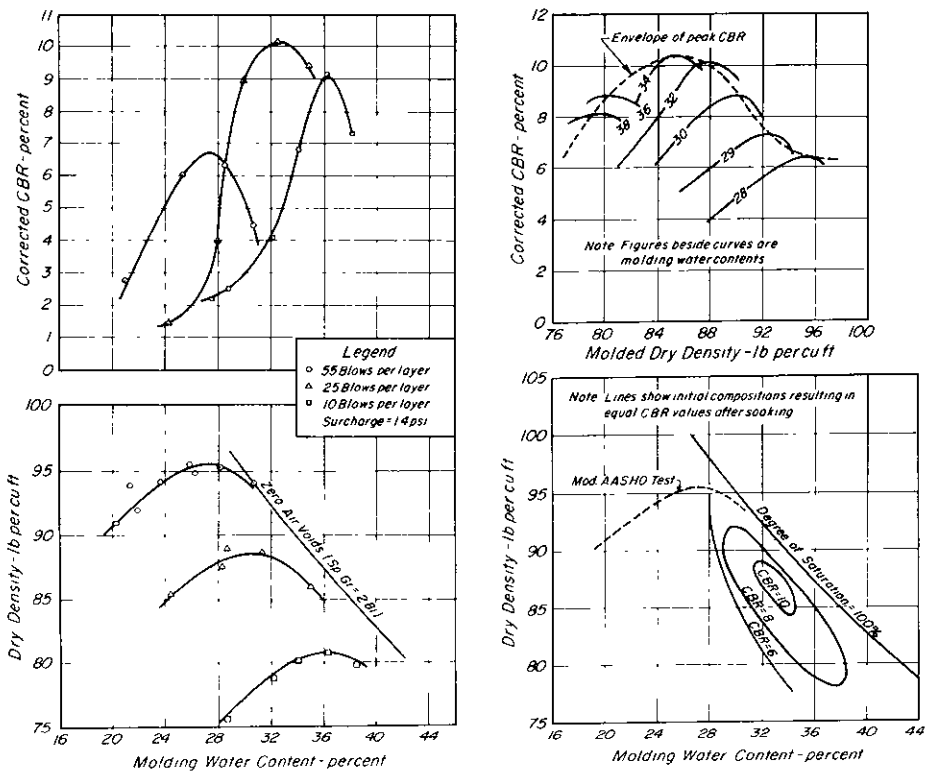


Fig. 14 -- Relationship between initial composition and soaked CBR for samples of highly plastic clay. (After Seed et al.)

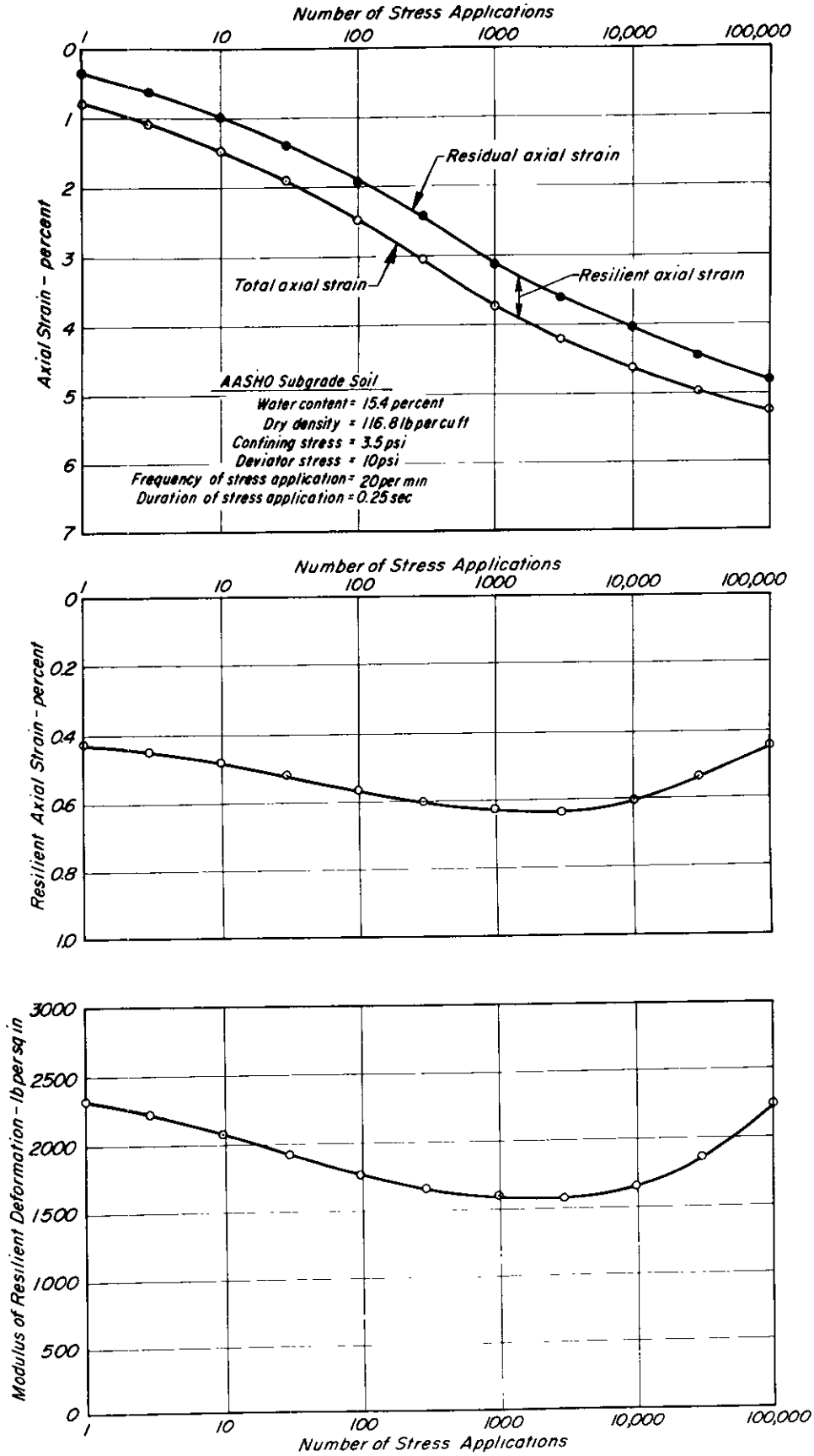


Fig. 15 — Typical results of repeated loading triaxial compression test. (After Seed et al.)

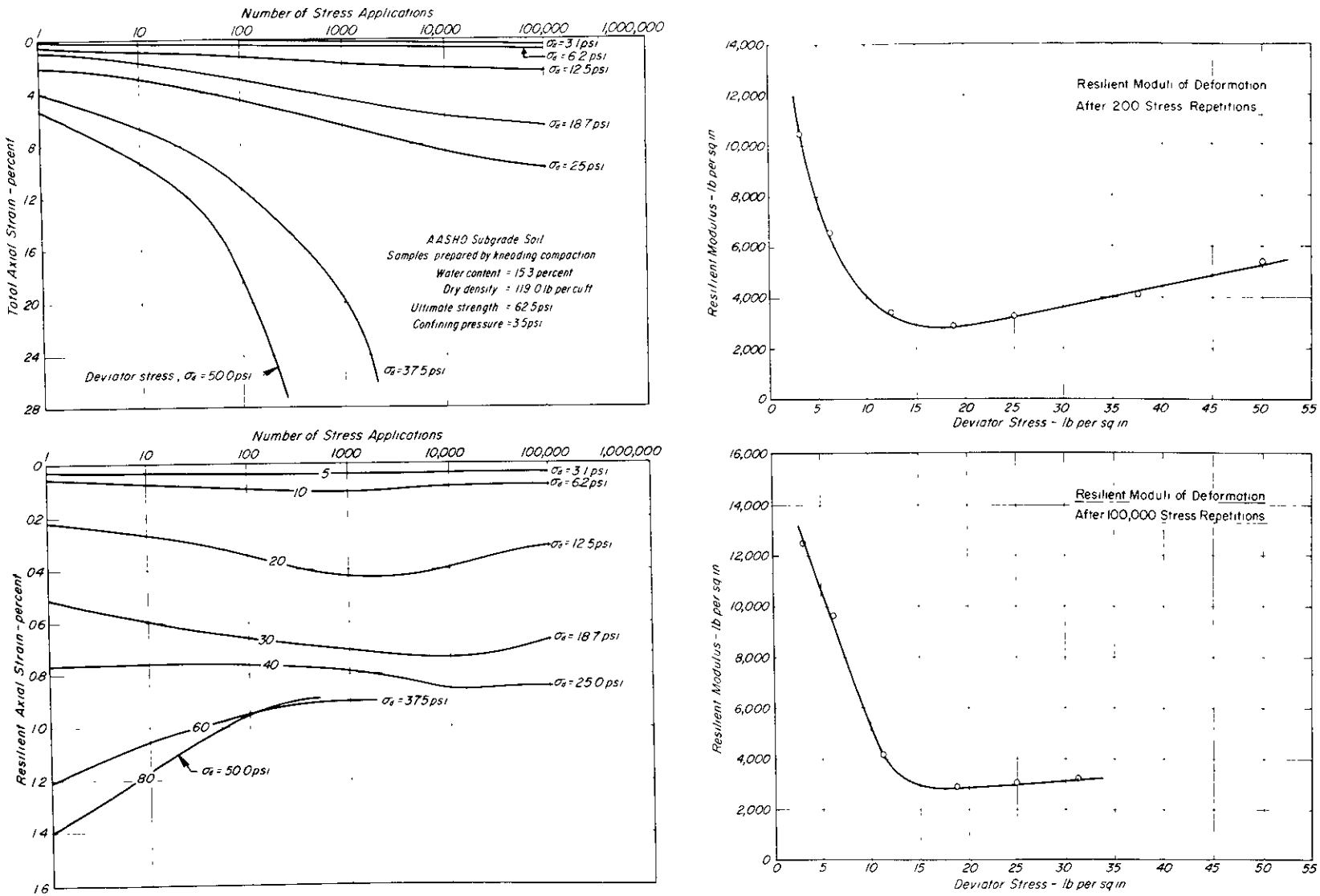


Fig. 16 — Effect of stress intensity on resilience characteristics — AASHO Road Test subgrade soil. (After Seed et al.)

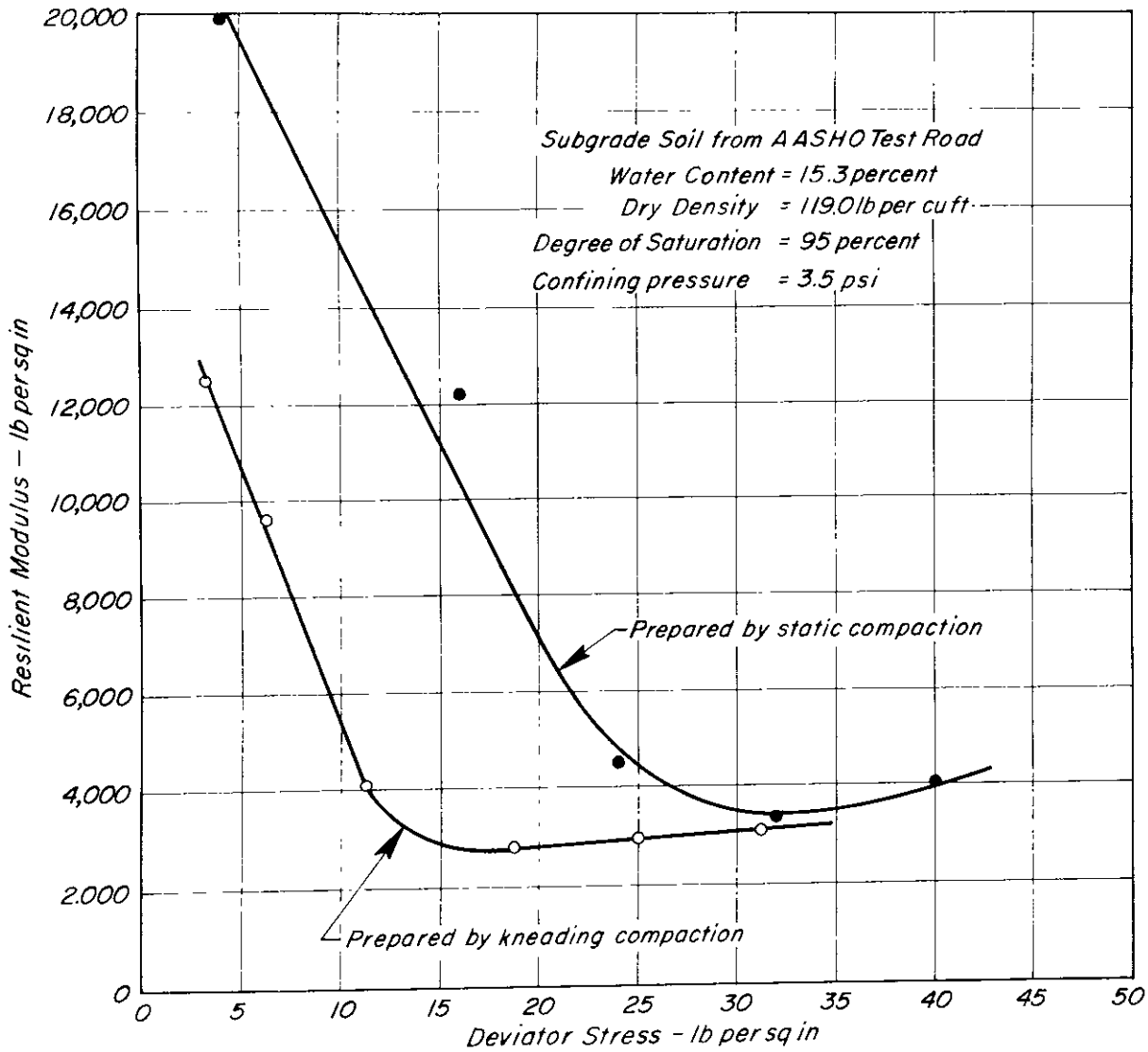


Fig. 17 – Effect of method of compaction on relationship between resilient modulus and stress intensity. (After Seed et al.)

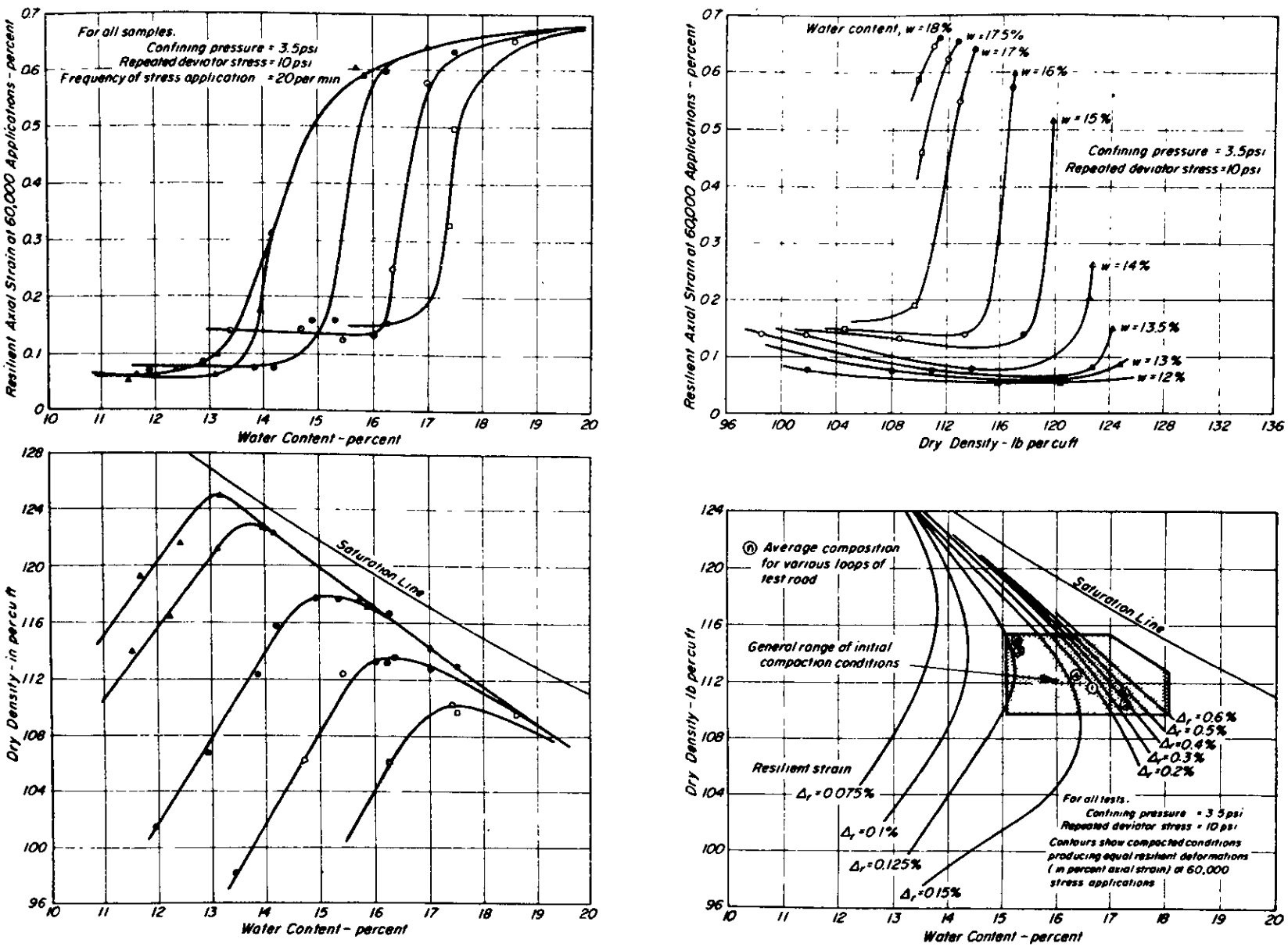


Fig. 18 — Relationship between dry density, water content and resilient strain — AASHO Road Test subgrade soil prepared by kneading compaction. (After Seed et al.)

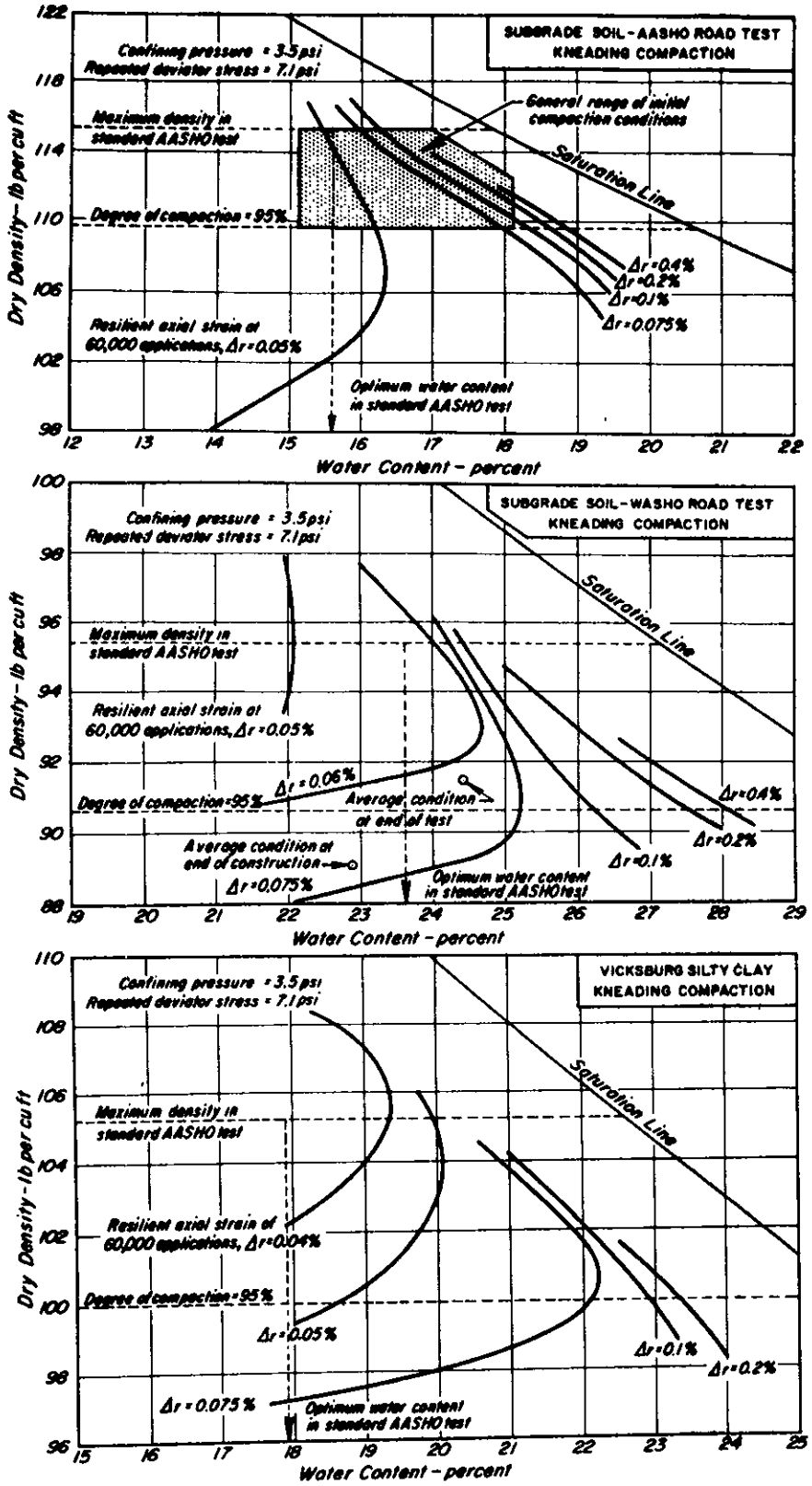


Fig. 19 — Effect of compaction conditions on resilience characteristics. (After Seed et al.)

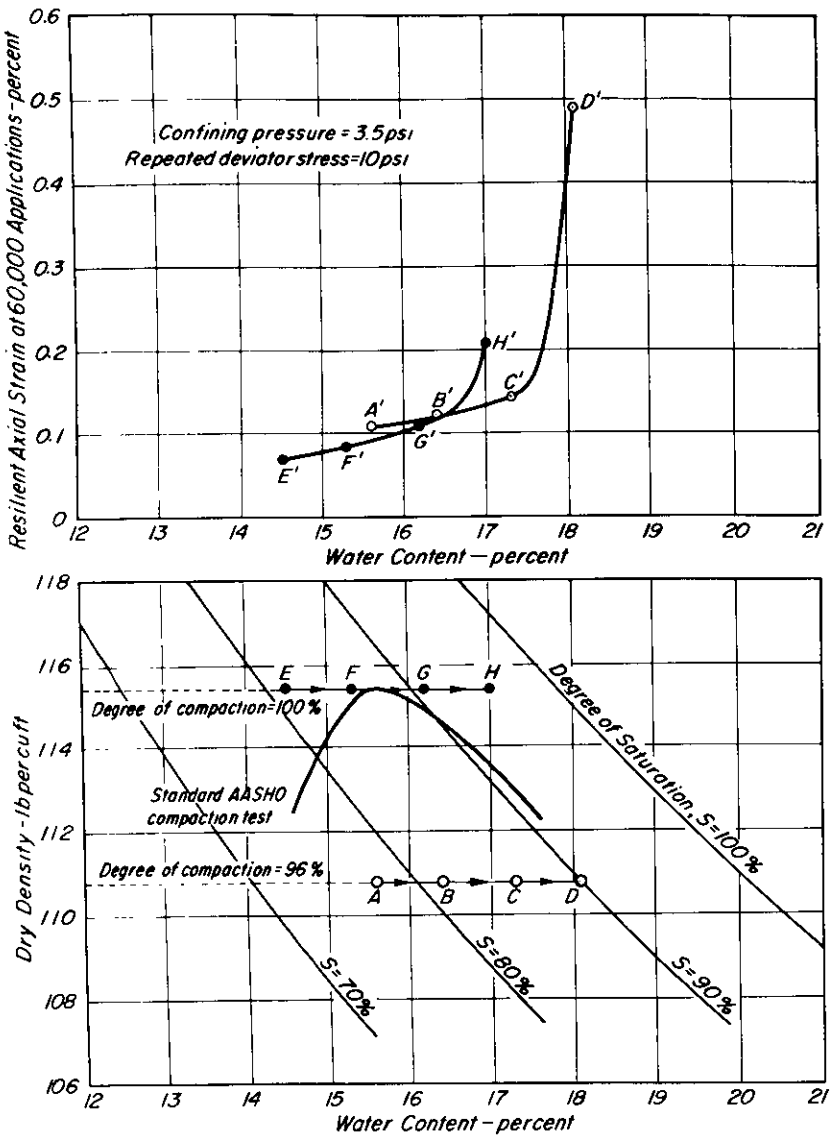


Fig. 20 — Effect of increase in water content after compaction on resilient deformations — AASHO Road Test subgrade soil. (After Seed et al.)

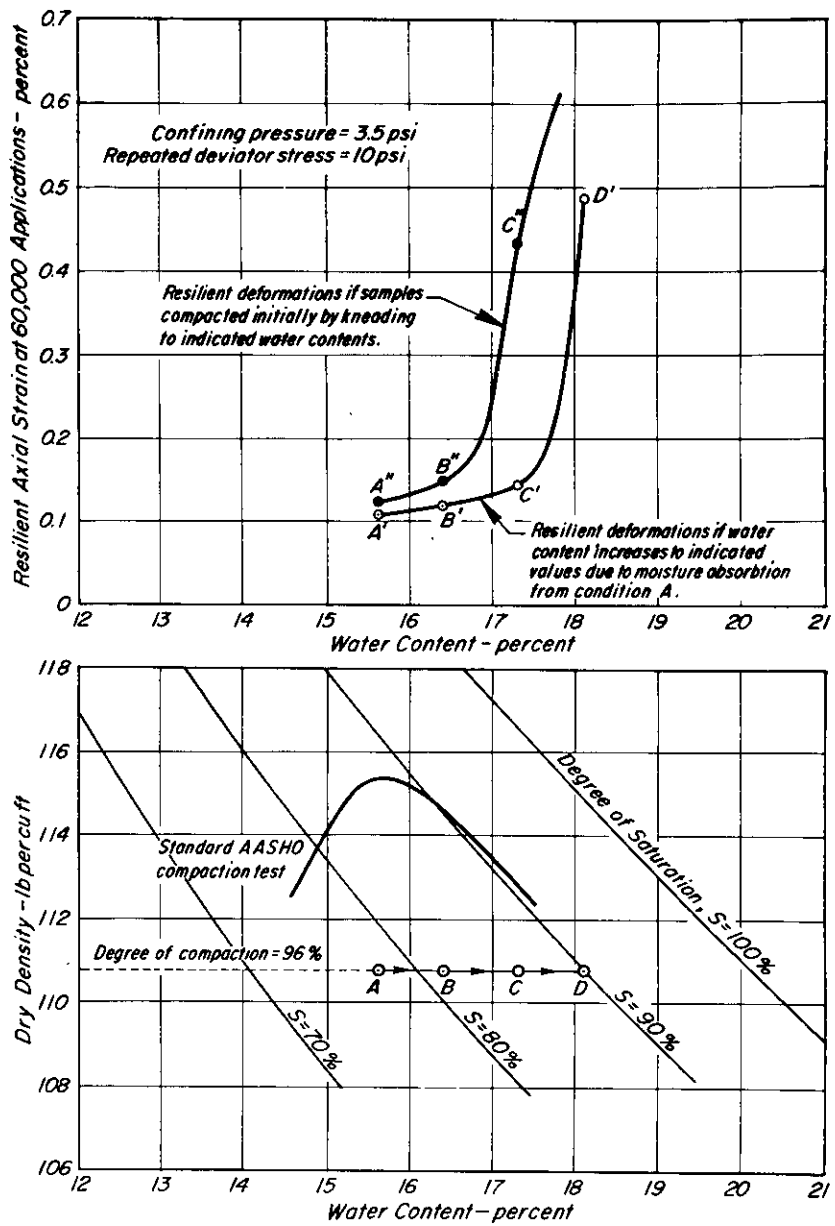


Fig. 21 — Effect of method of attaining final moisture condition on resilient strains. (After Seed et al.)

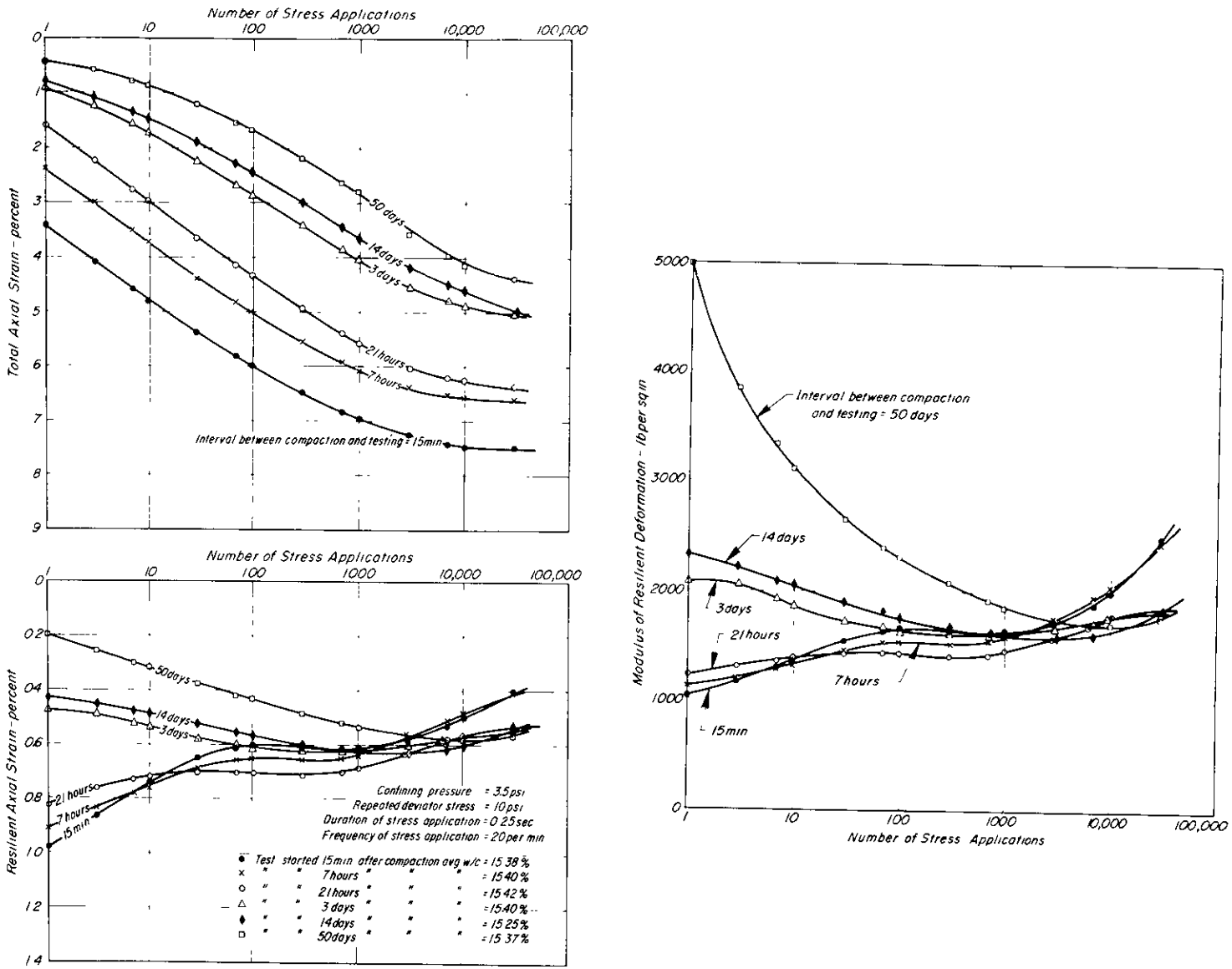


Fig. 22 – Effect of thixotropy on resilience characteristics -- AASHO Road Test subgrade soil. (After Seed et al.)

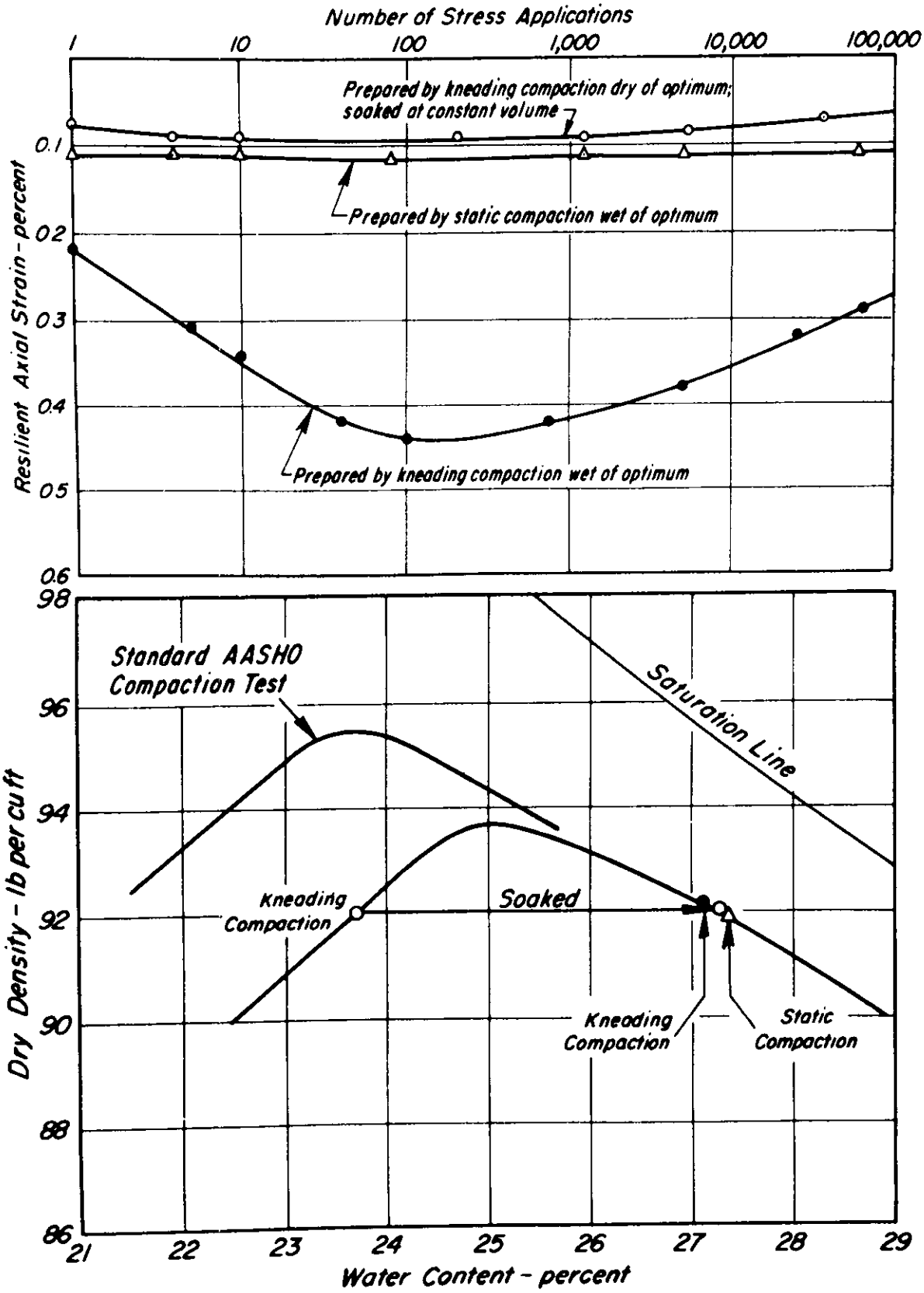


Fig. 23 — Comparison of resilience characteristics of specimens prepared by static compaction "wet of optimum" and by kneading compaction "dry of optimum" and soaked to the same degree of saturation. (After Seed et al (31)).

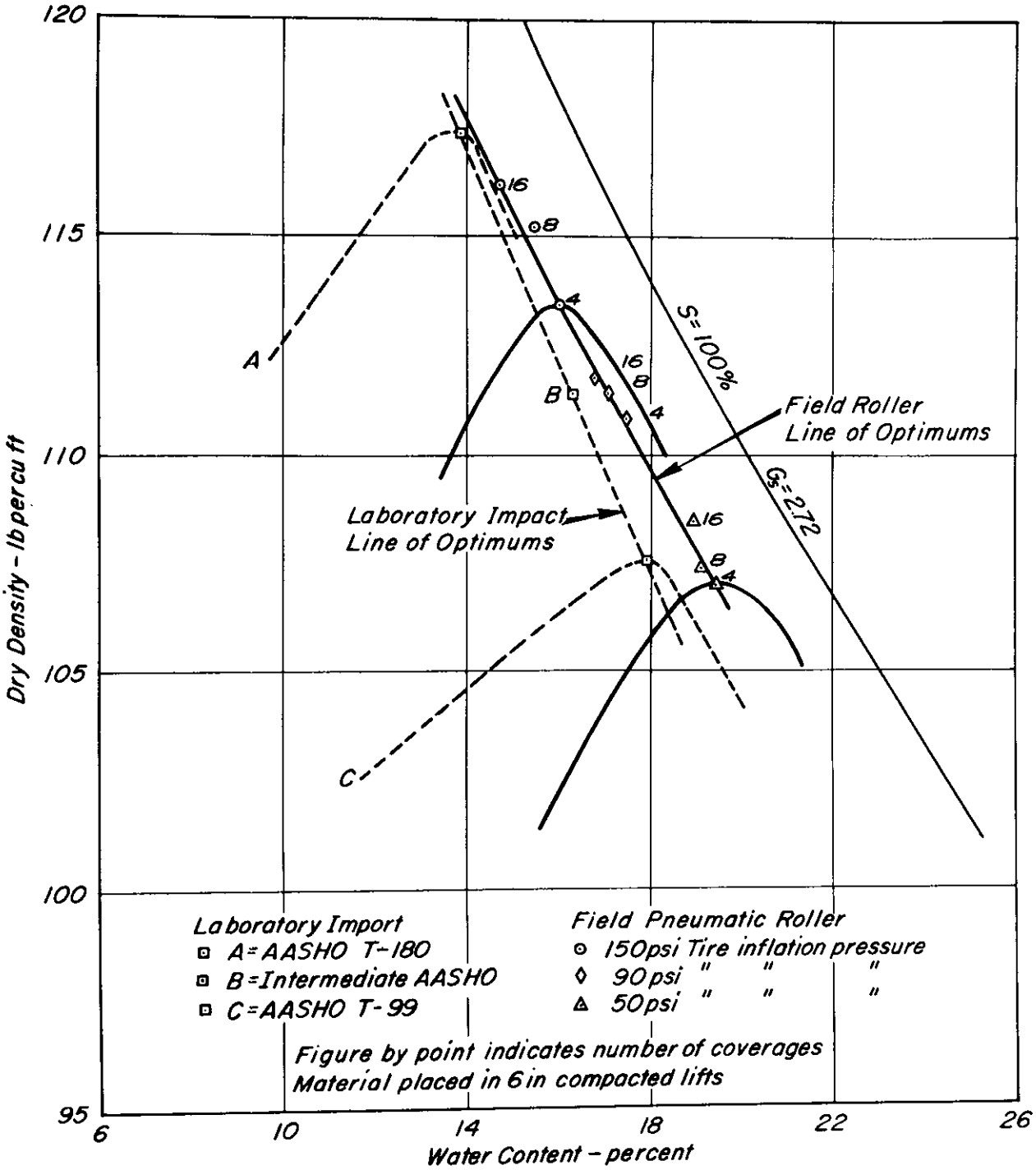
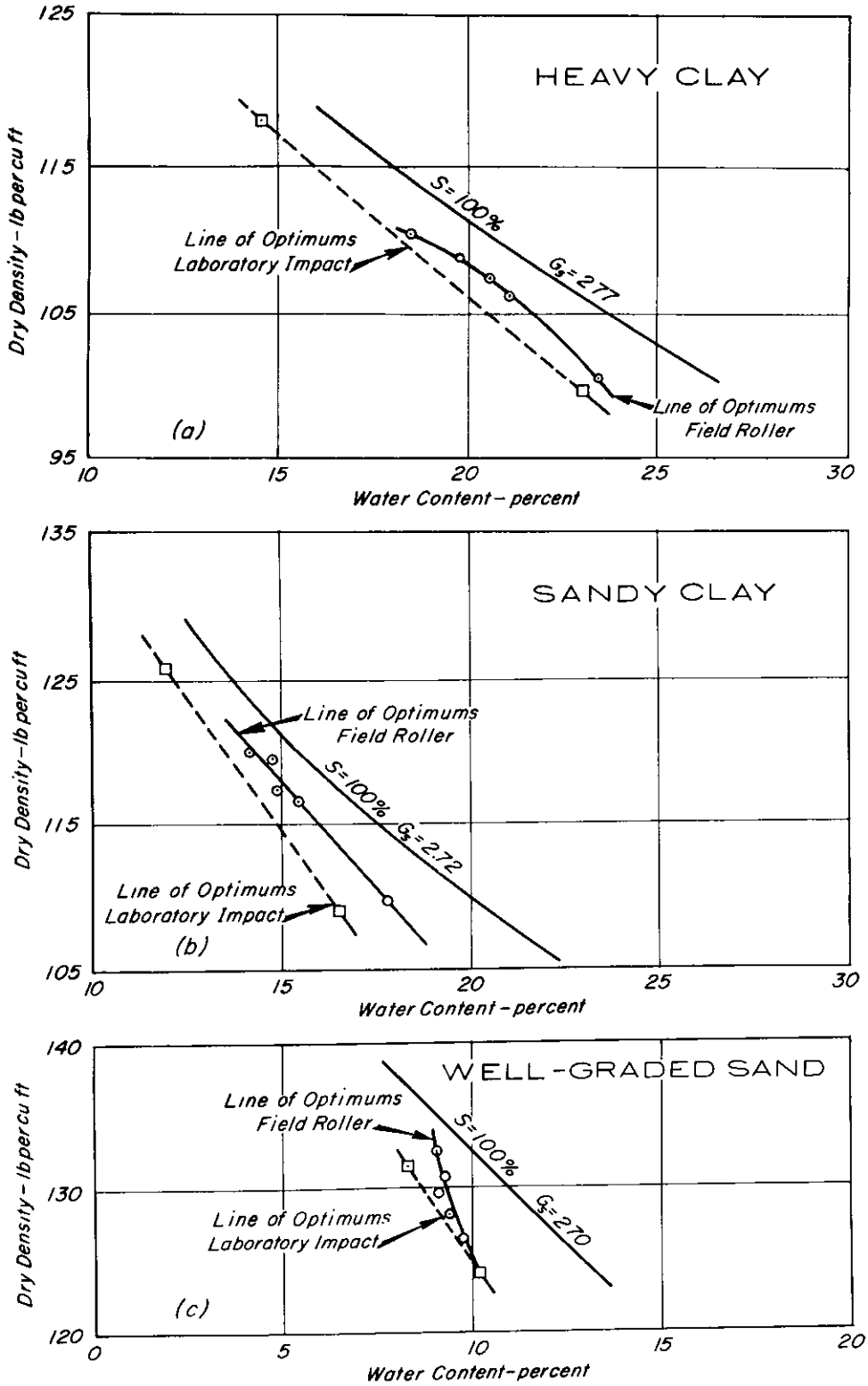


Fig. 24 — Relationship between line of optimums for laboratory impact compaction and field pneumatic-tired roller compaction for a silty clay and the relationship between constant compactive effort curves for the same conditions. (Ref. 46)



Note: All field compaction values are for 32 roller passes. Loose lifts of 12-in. for all rollers except 36 psi roller which was 9-in. Dry densities determined on upper 6-in. of compacted lift. Figures by roller optimum points are tire inflation pressures.

Fig. 25 — Comparison of laboratory line of optimum and field line of optimum for three soils. (Ref. 18.)

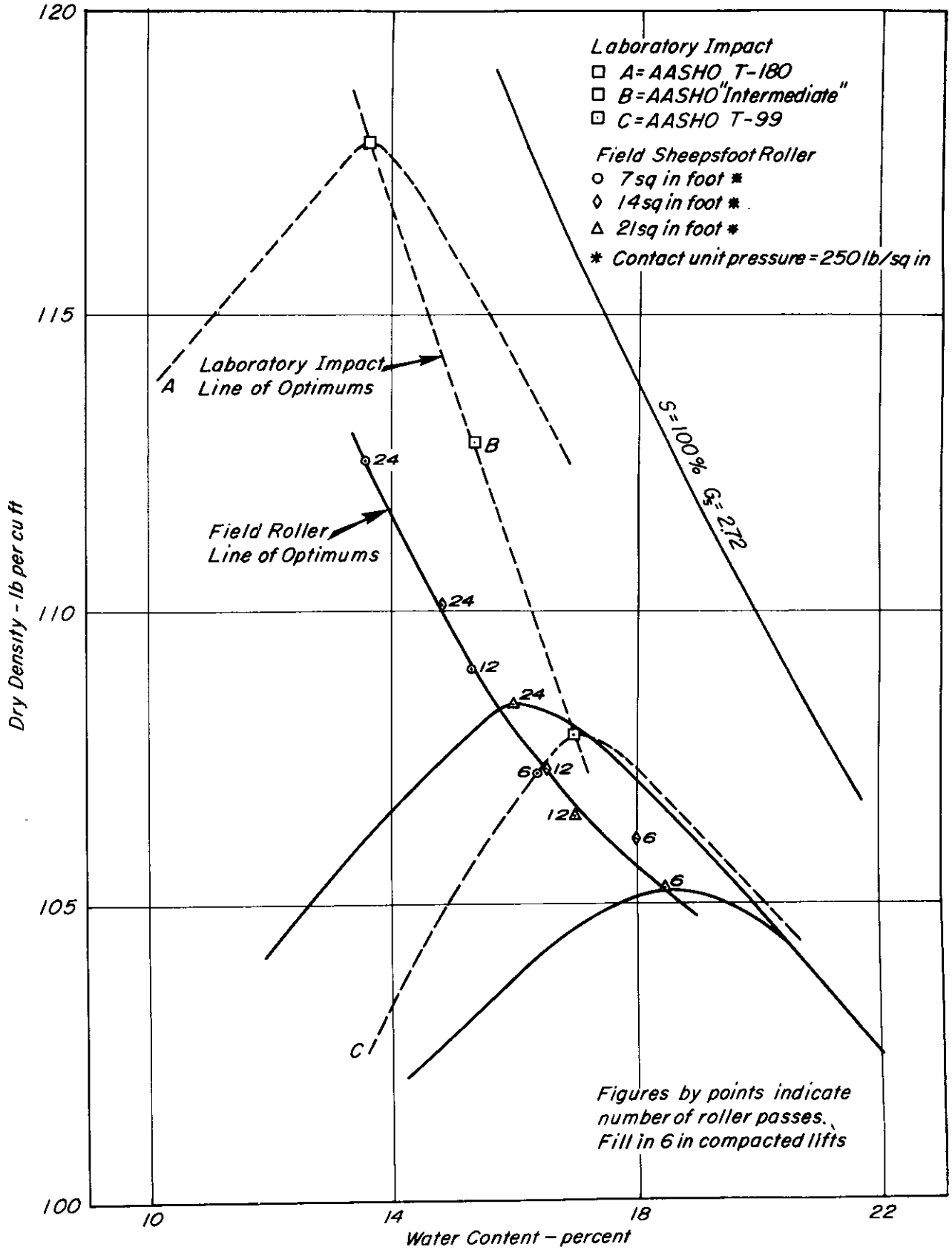


Fig. 26 — Relationship between line of optimums for laboratory impact compaction and field sheepsfoot compaction for a silty clay and the relationship between constant compactive effort curves for the same conditions. (Ref. 19.)

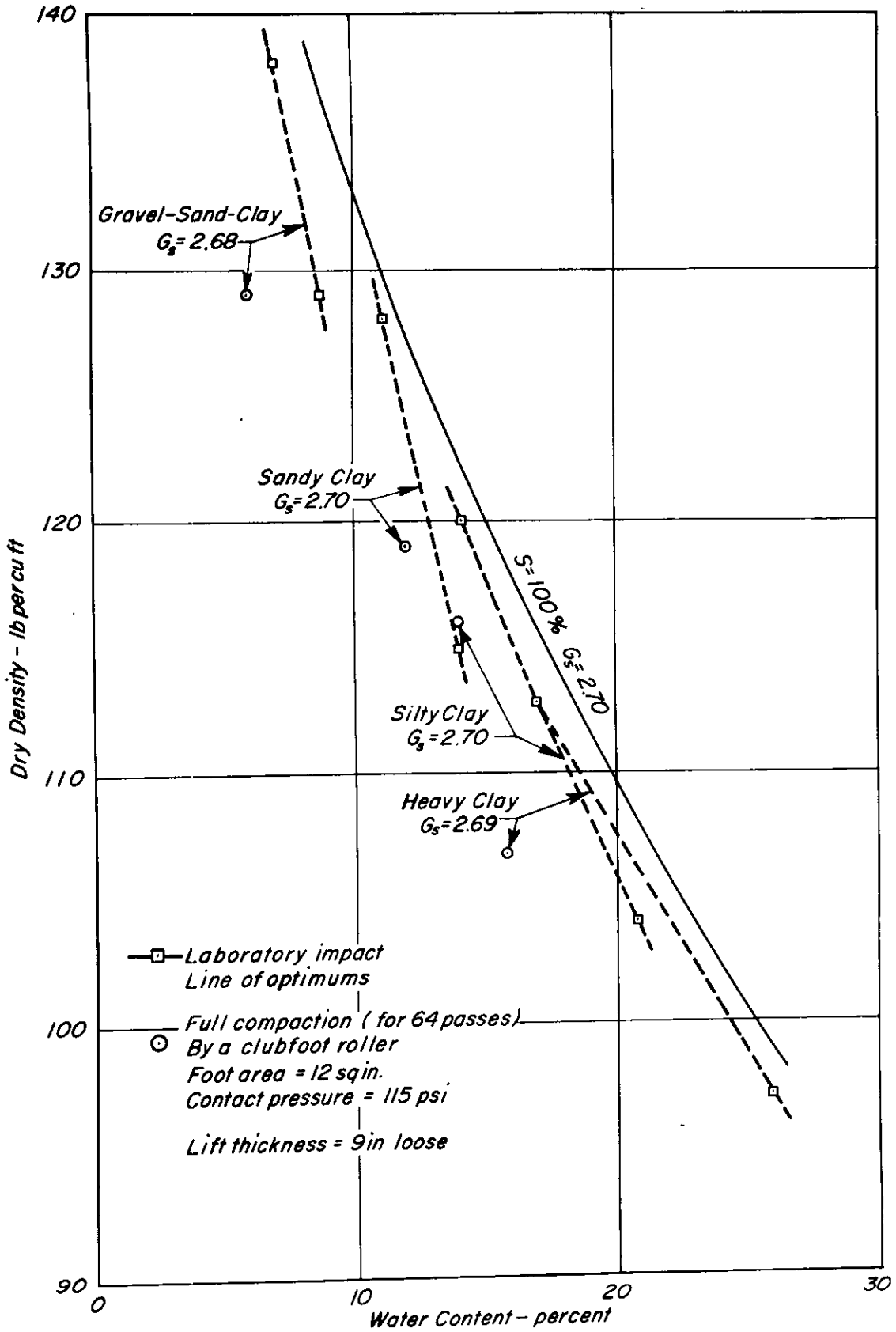


Fig. 27 – Comparisons of sheepsfoot-type roller maximum dry density and water contents with laboratory impact compaction line of optimums for four different soils. (Ref. 20.)

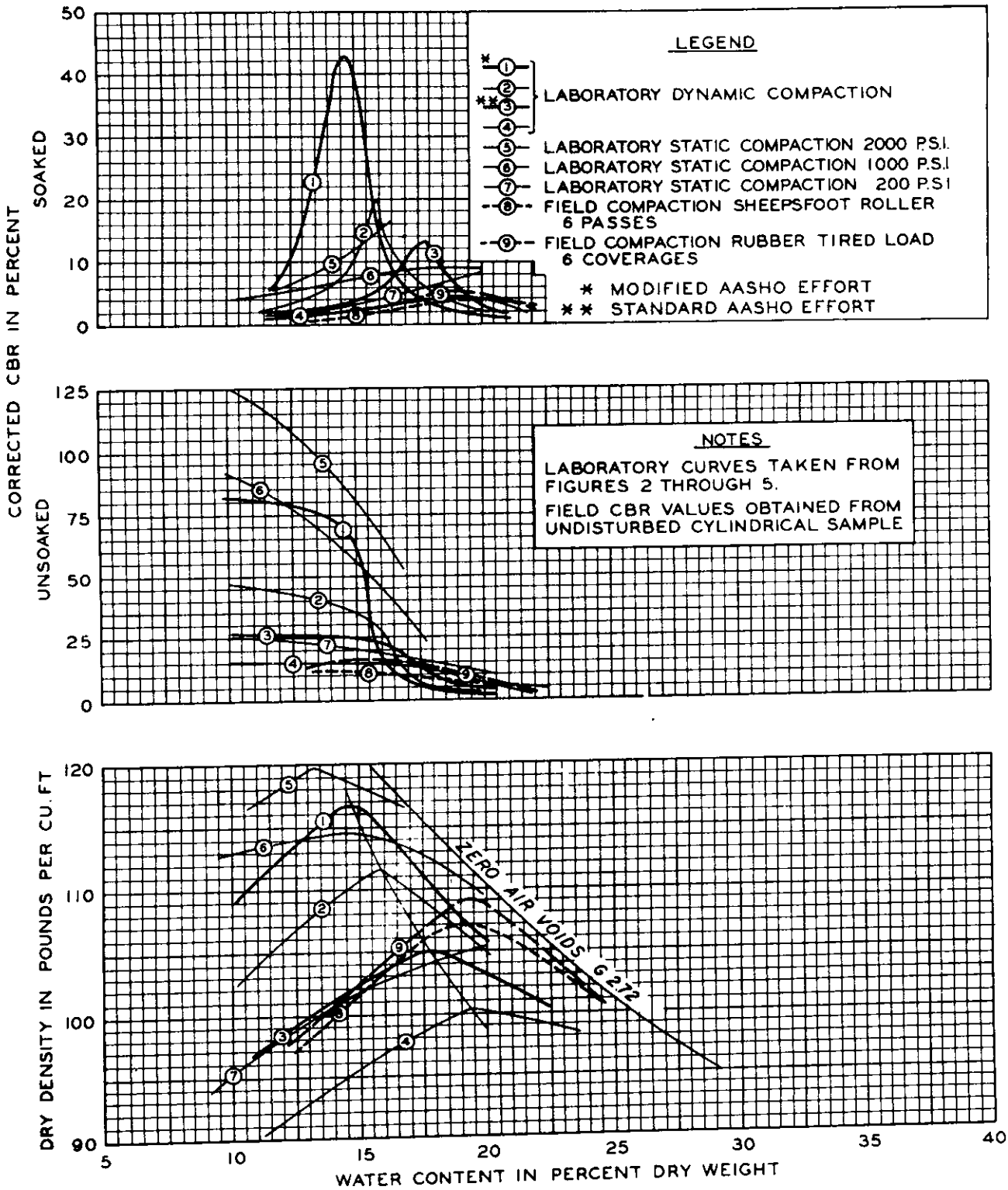


Fig. 28 — Comparison of field and laboratory compaction and CBR.

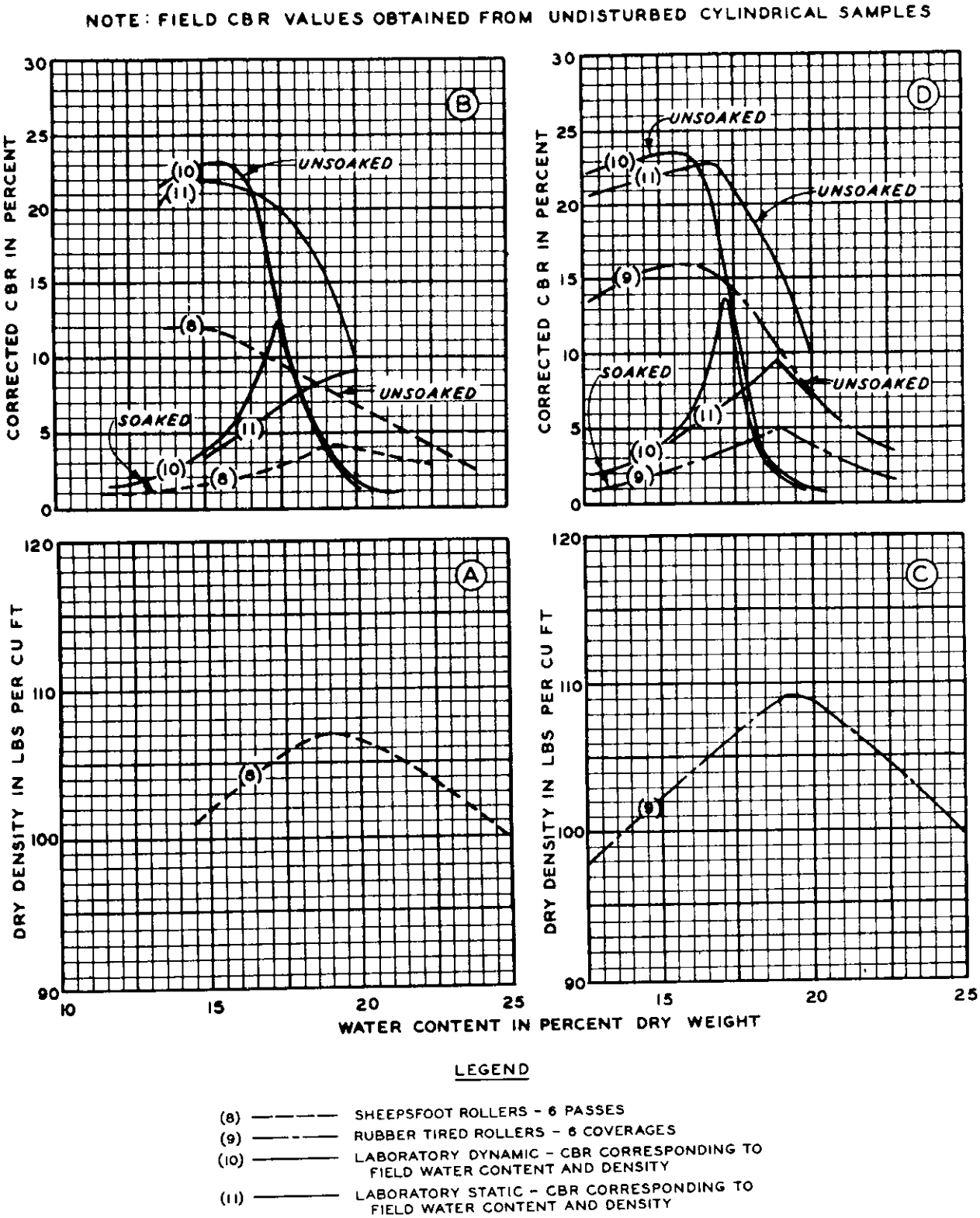


Fig. 29 — Comparison of CBR-field compacted samples vs. laboratory compacted samples. (Equal water contents and densities.)

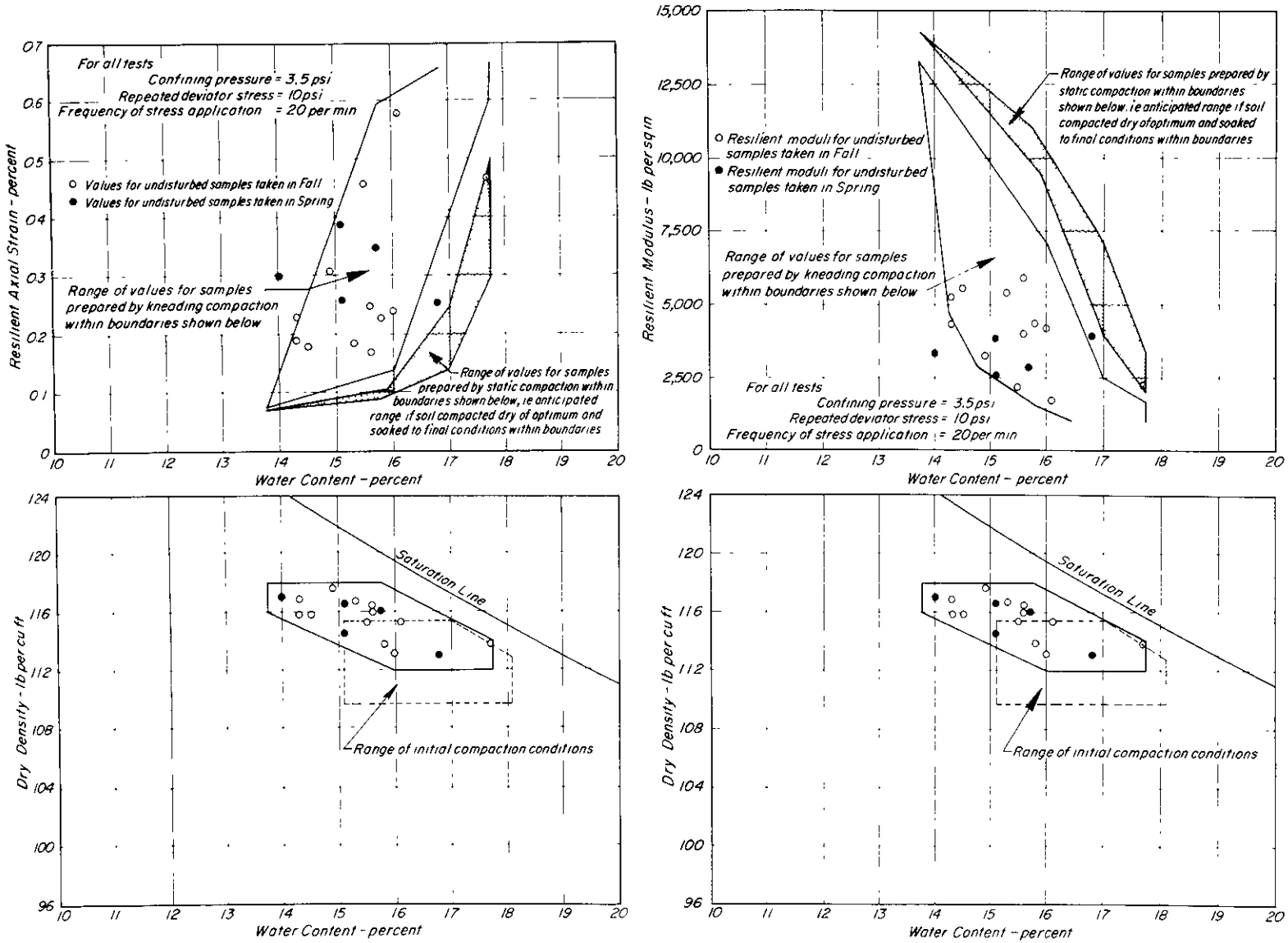


Fig. 30 — Resilient characteristics of undisturbed samples from untrafficked loops of AASHO Test Road. (After Seed et al.)

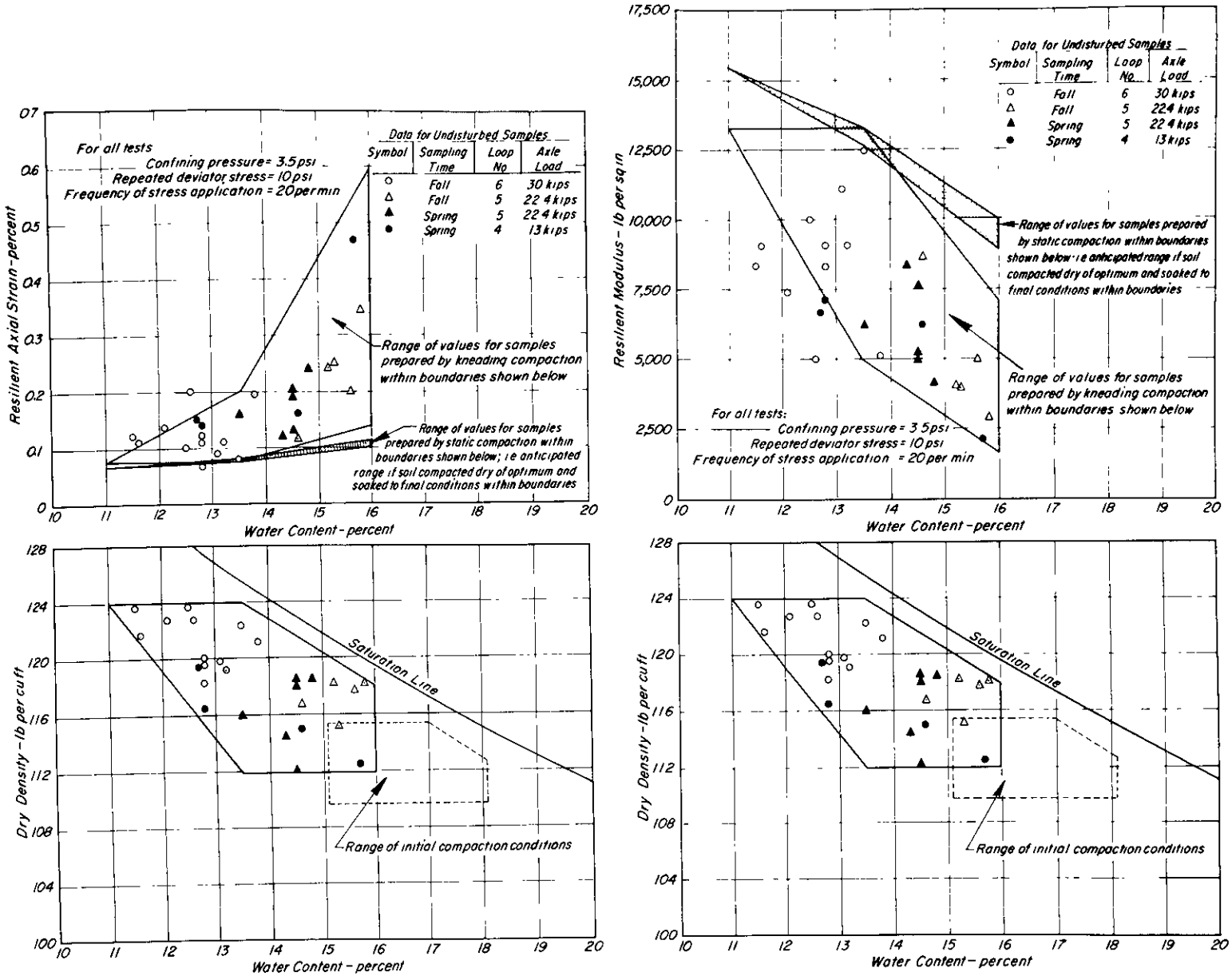


Fig. 31 — Resilient characteristics of undisturbed samples from trafficked loops of AASHO Test Road. (After Seed et al.)

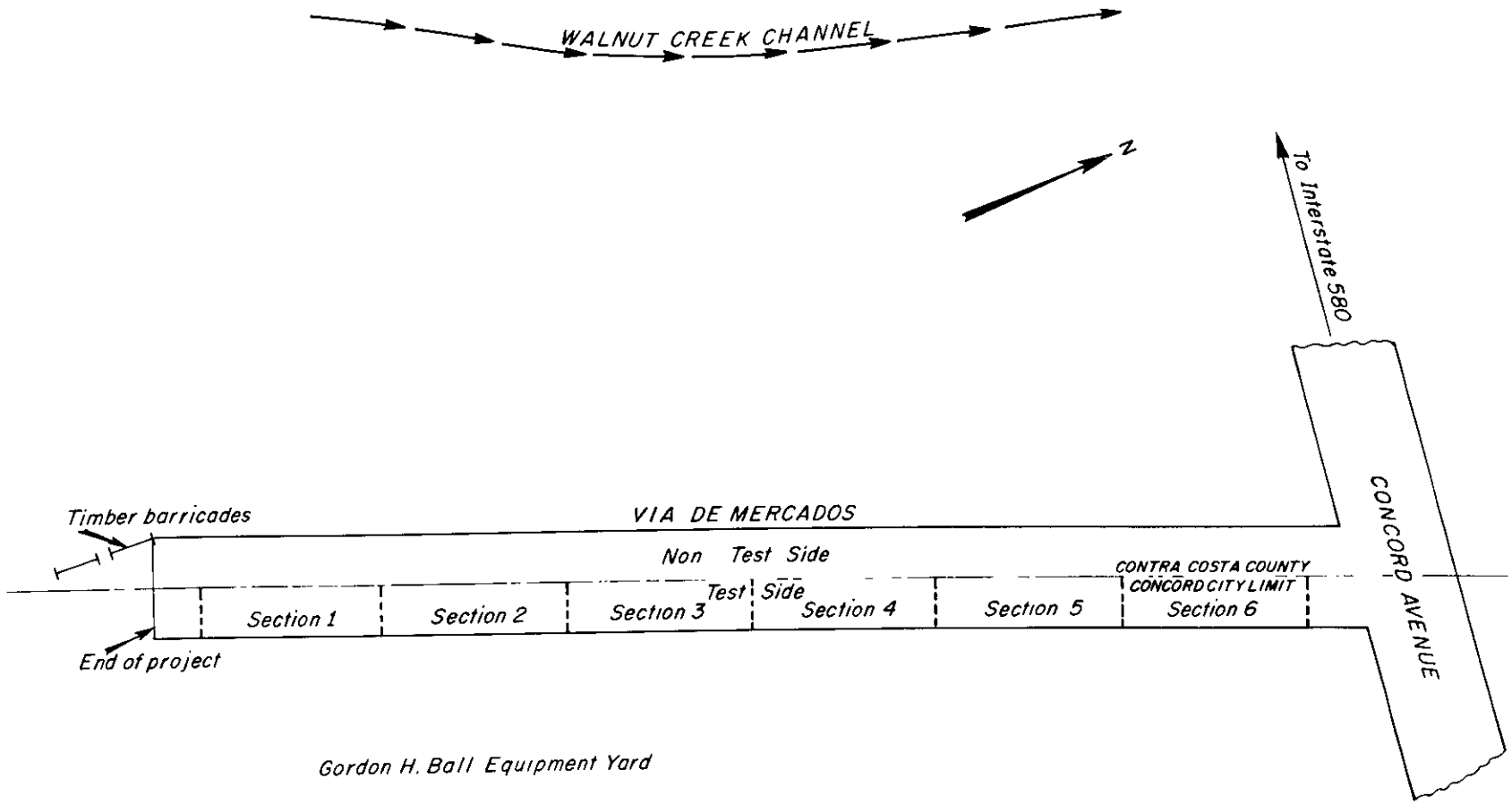


Fig. 32 — Sketch of Test Road site.

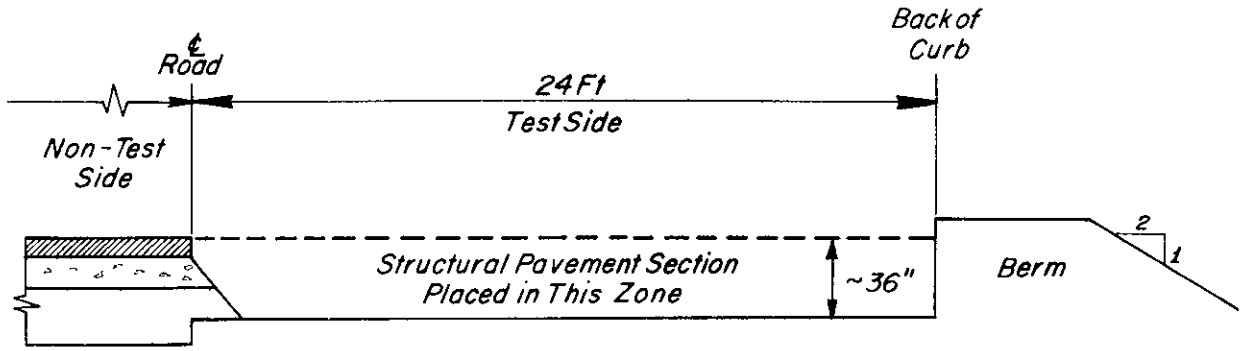


Fig. 33 — Cross-section of test section before placement of subgrade, base, and asphalt concrete.

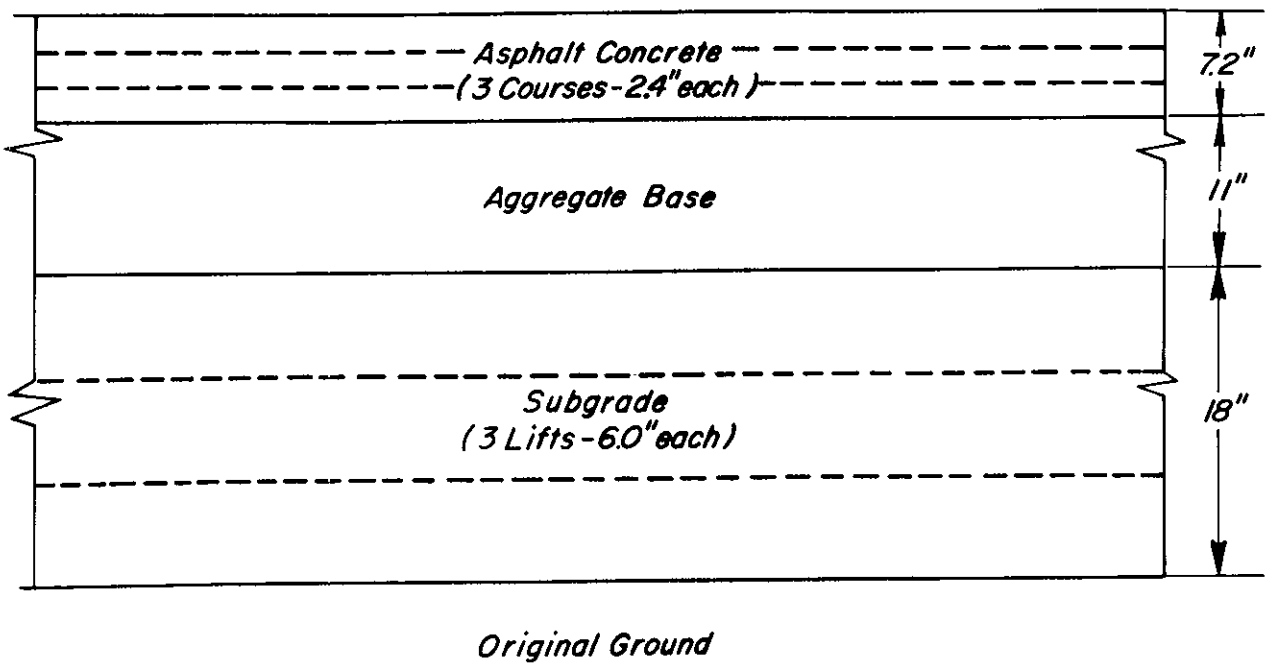


Fig. 34. Structural pavement section of test road.

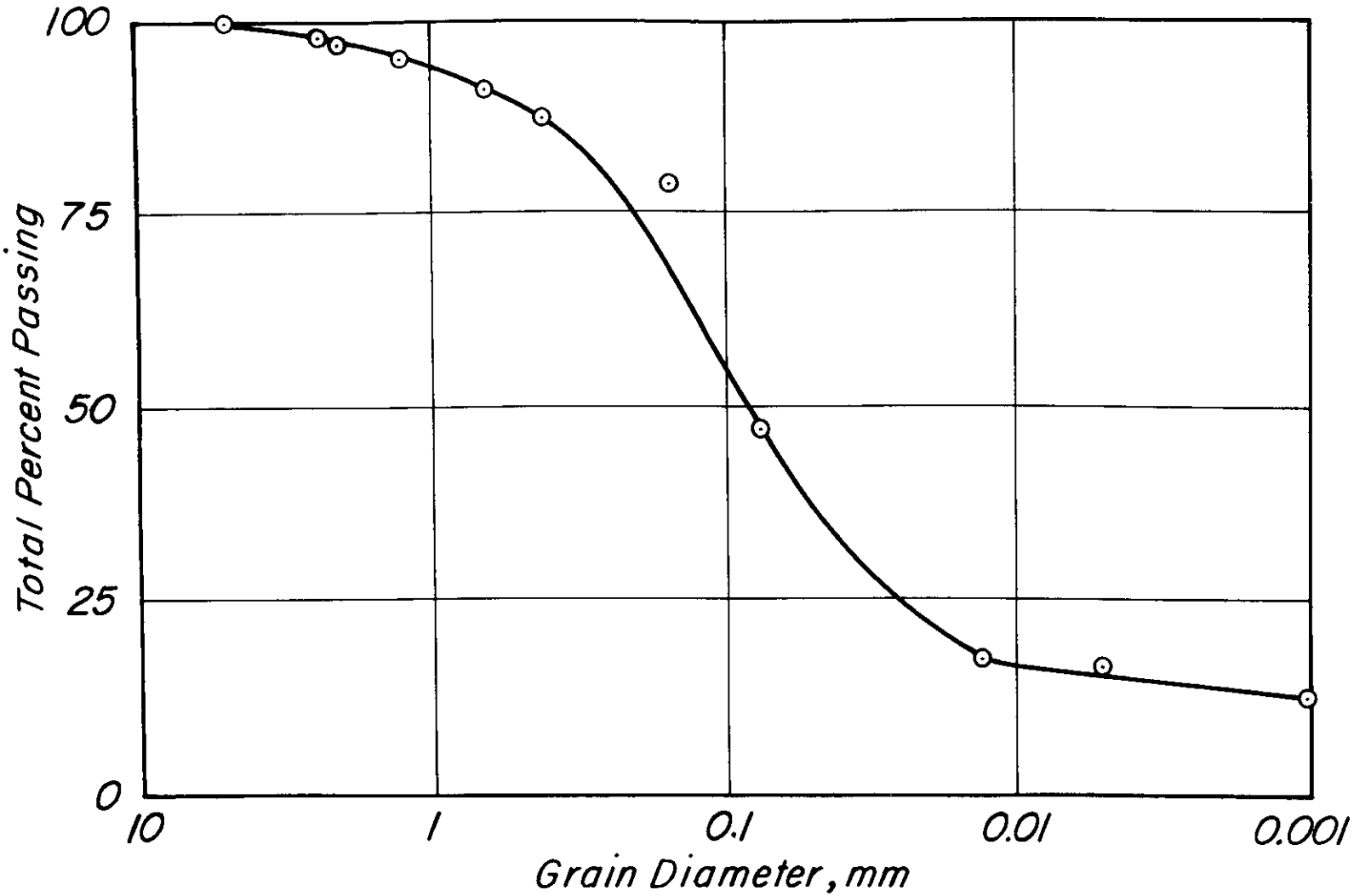


Fig. 35 -- Grain size distribution for silty-clay subgrade soil.

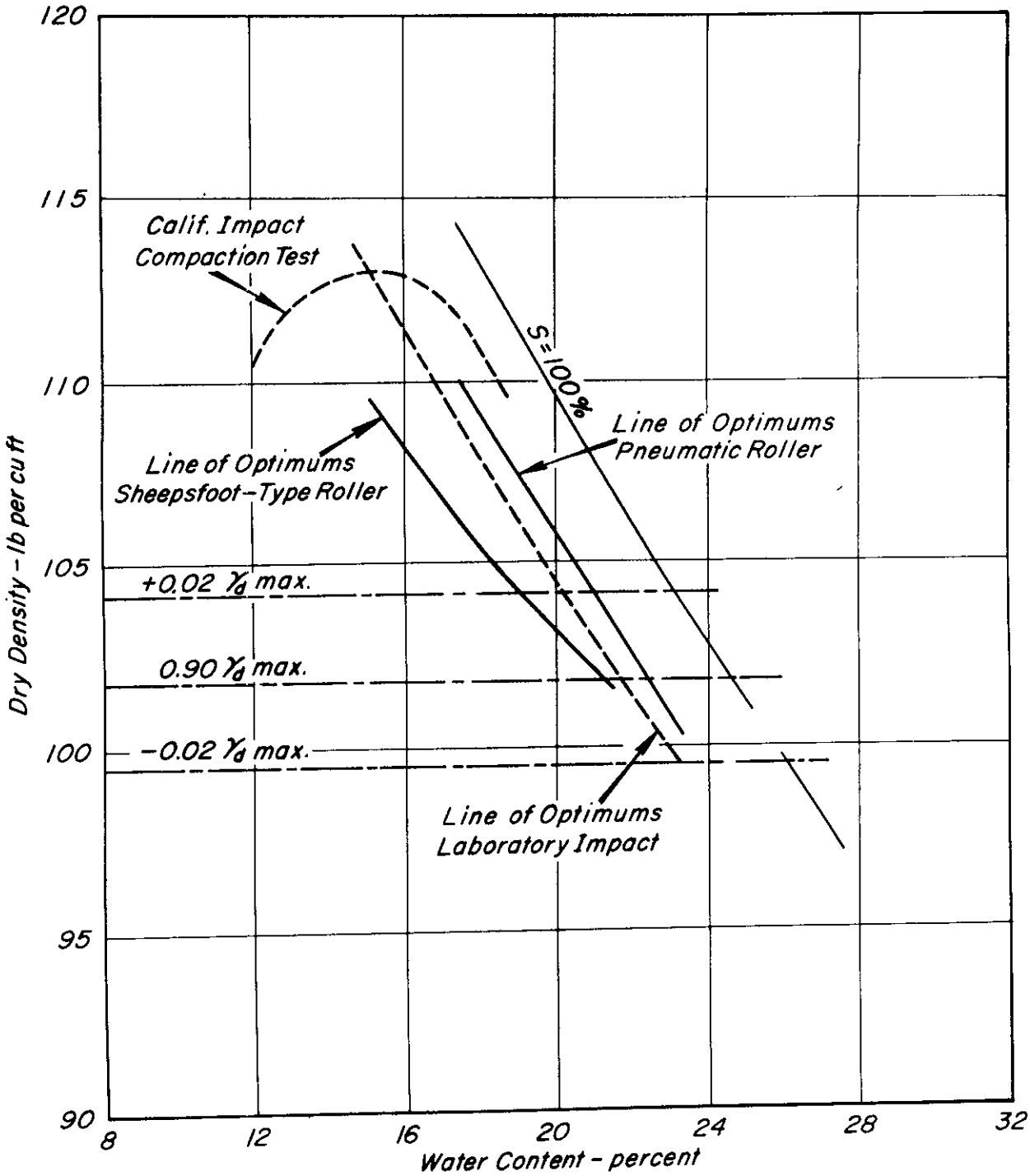


Fig. 36 — Compaction data silty-clay subgrade soil.

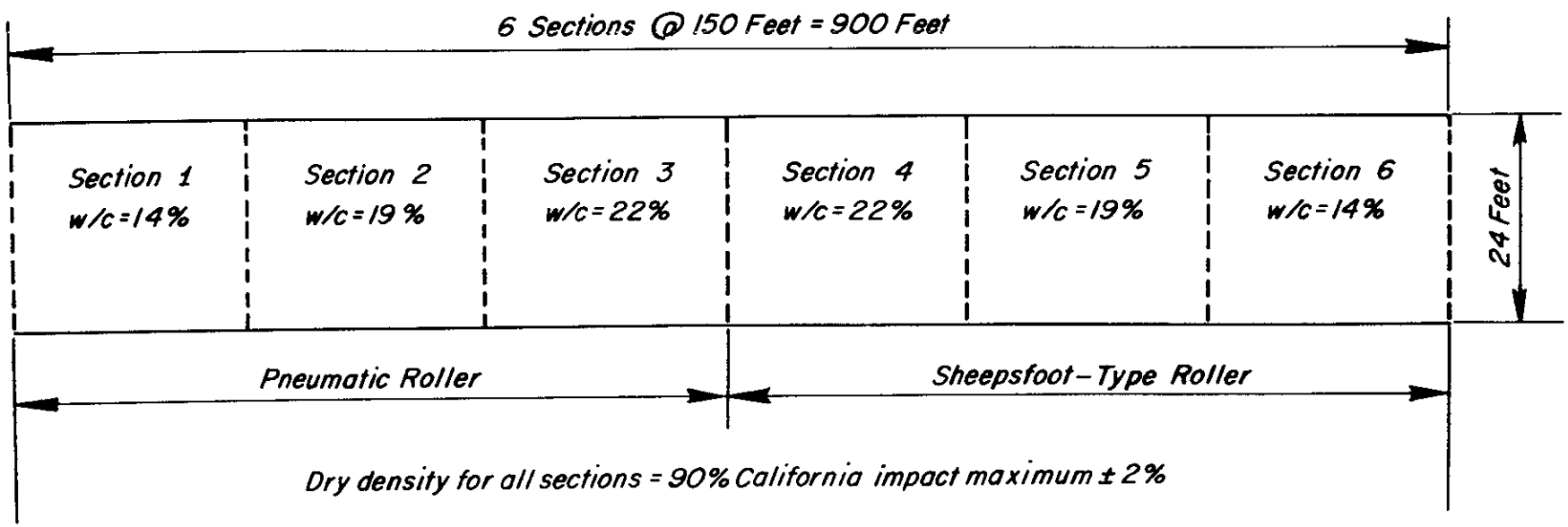


Fig. 37 — Schematic layout of test road.

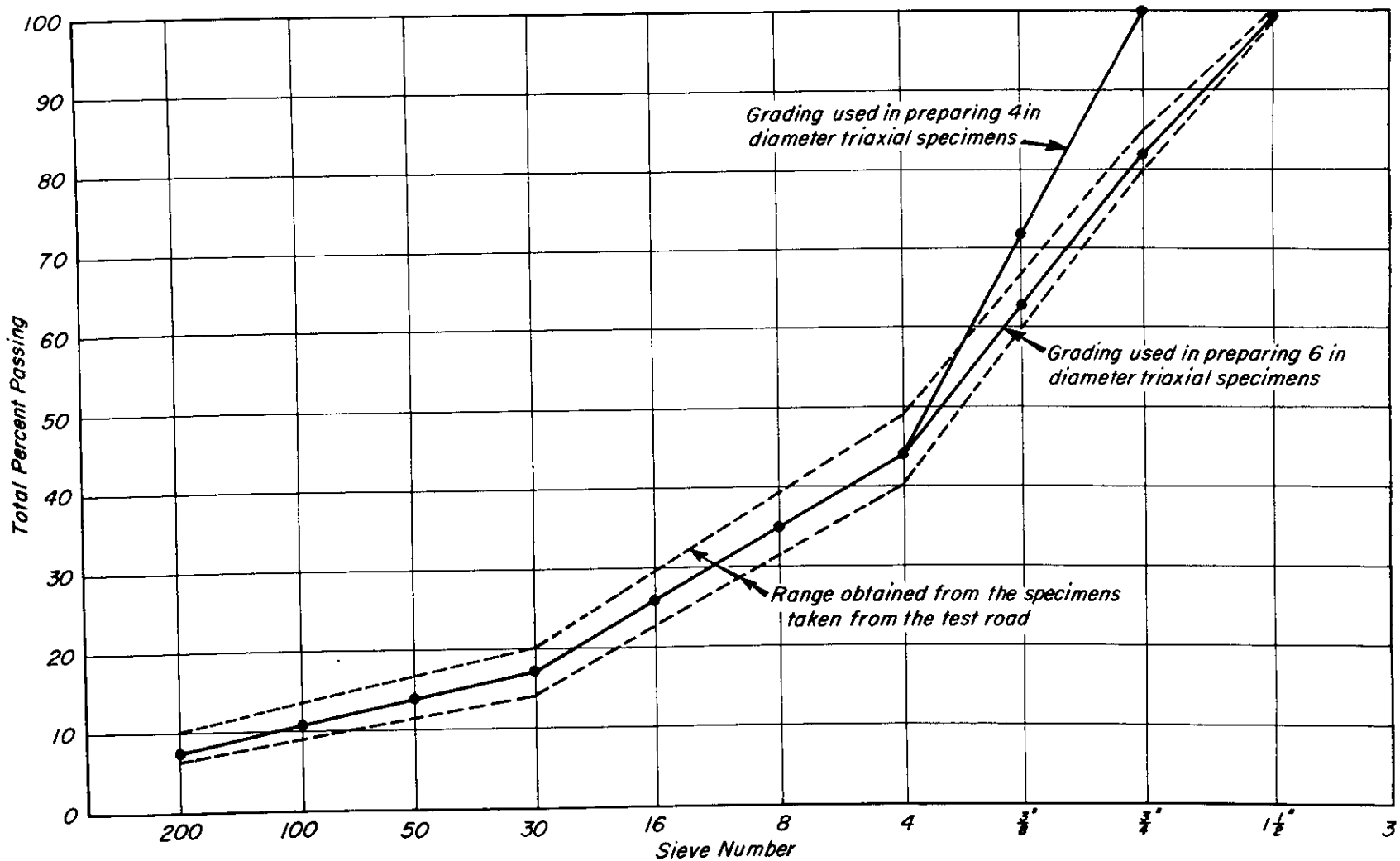


Fig. 38 — Grading curve of aggregate base.

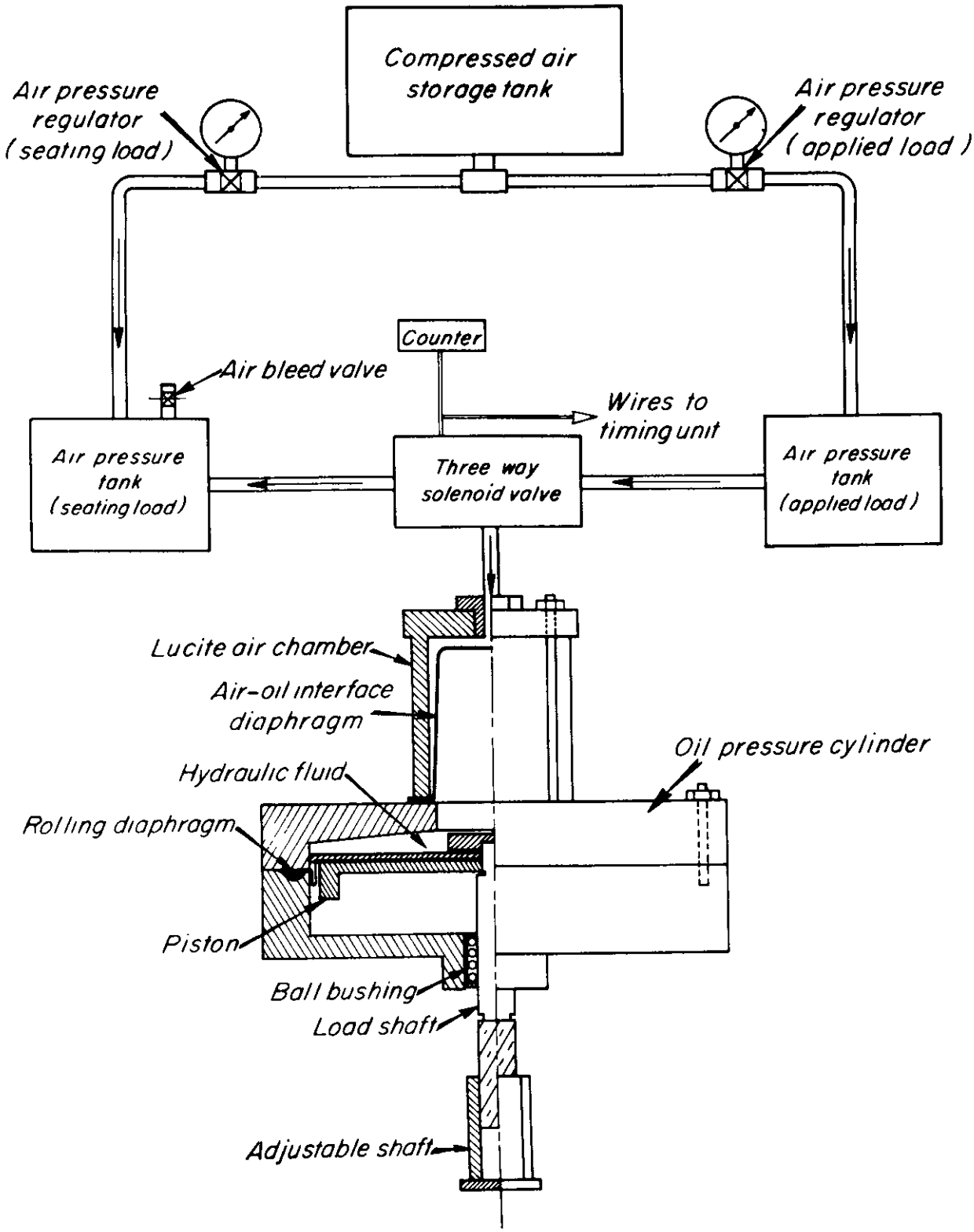


Fig. 39 — Large loading piston and control mechanism.

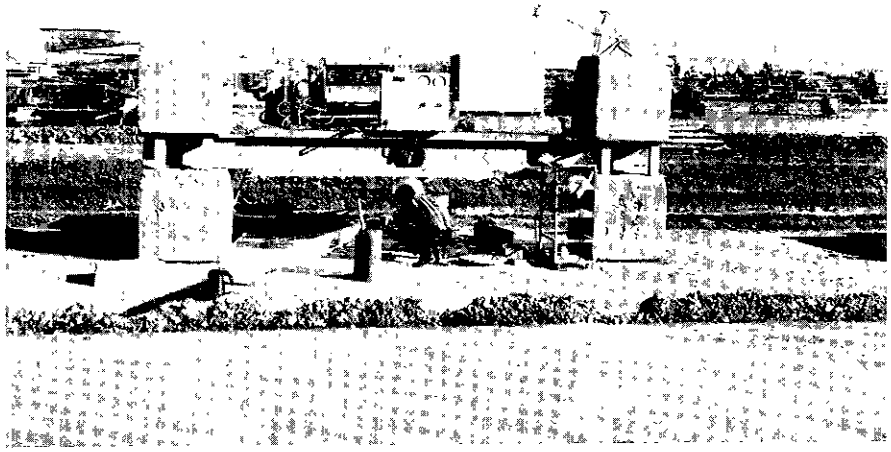


Fig. 40 — Photograph of field test installation for tests on subgrade.

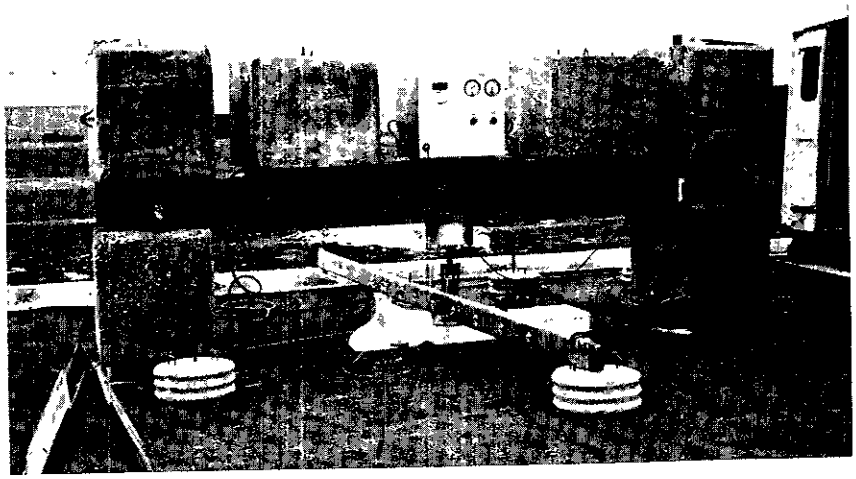


Fig. 41 — Photograph of field test installation for tests on asphalt concrete surfacing.

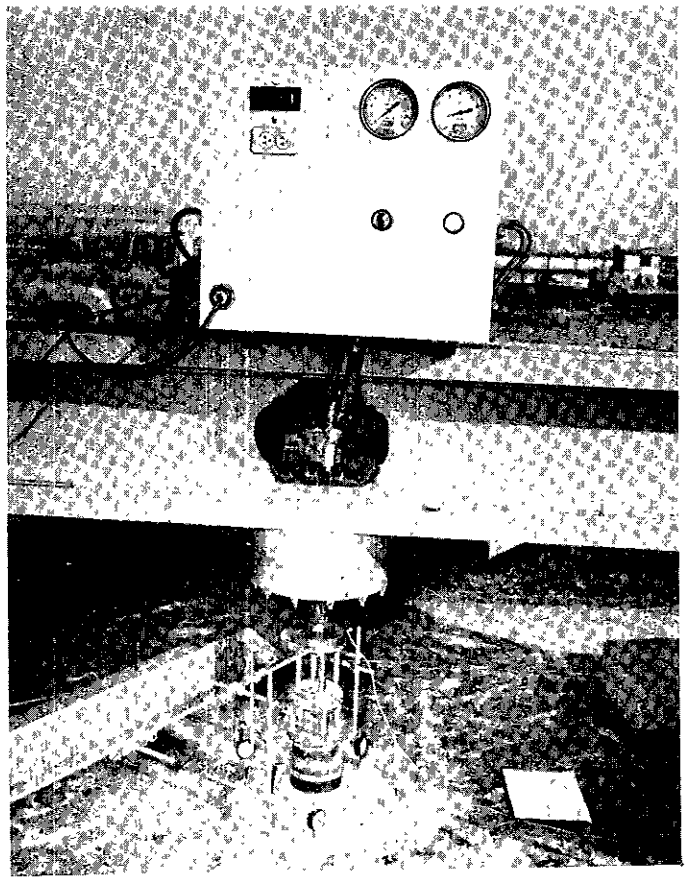


Fig. 42 — Photograph of an 18-in. plate test on the subgrade.

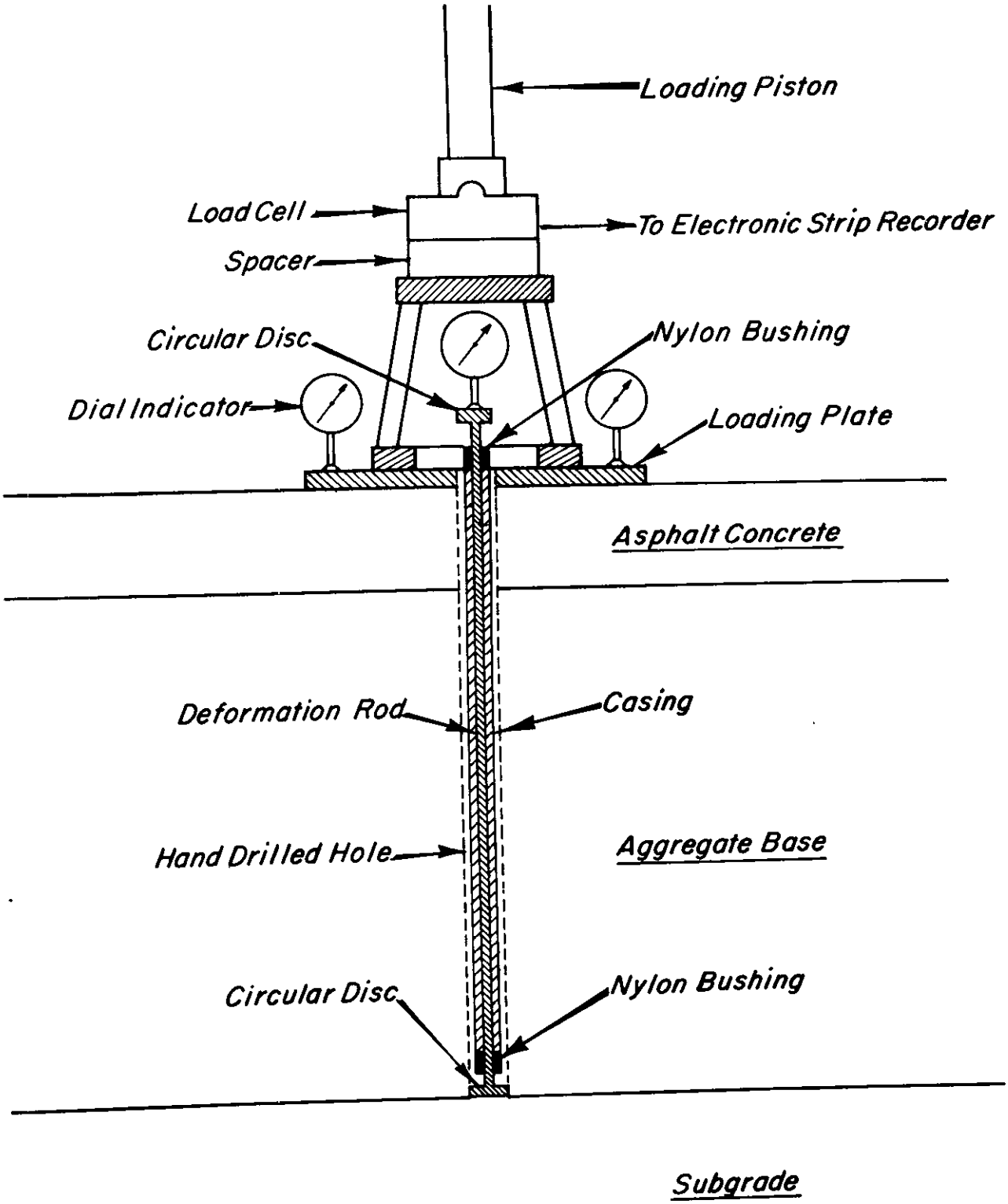


Fig. 43 — Schematic section of the system used to measure deflections of base-subgrade interface in layered pavement sections.

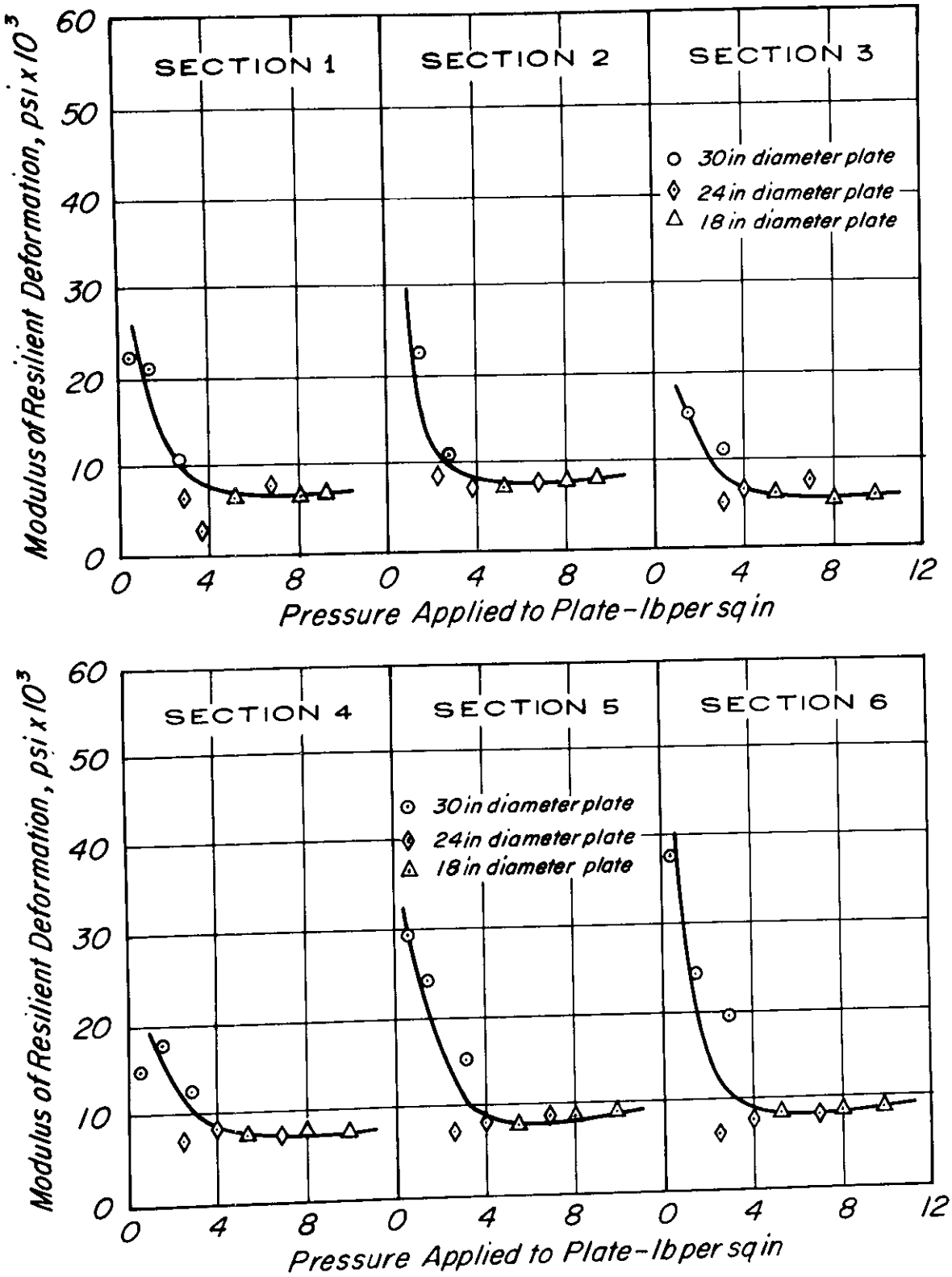


Fig. 44 — Results of repeated plate load tests at surface of subgrade for six test sections.

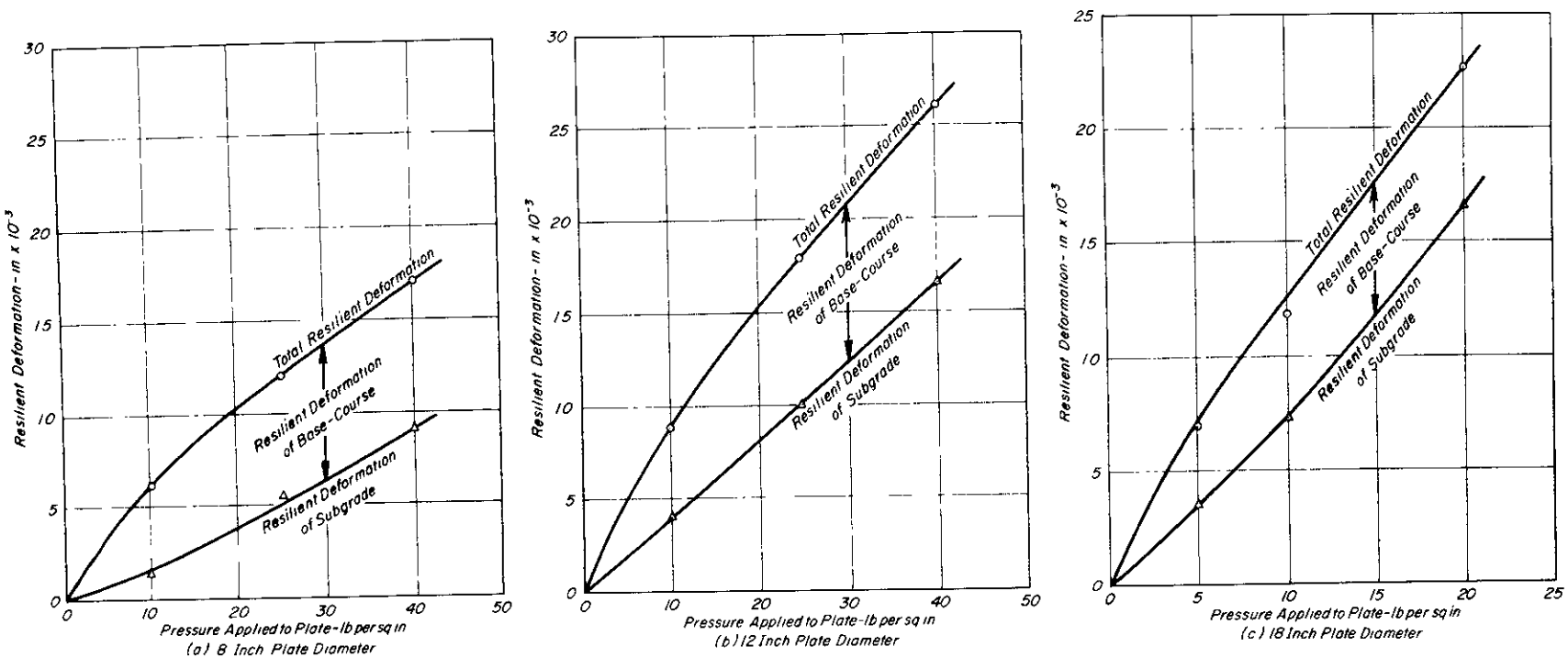


Fig. 45 — Results of plate load tests at surface of two-layer system consisting of 11-in. of untreated aggregate base and subgrade — Section 1.

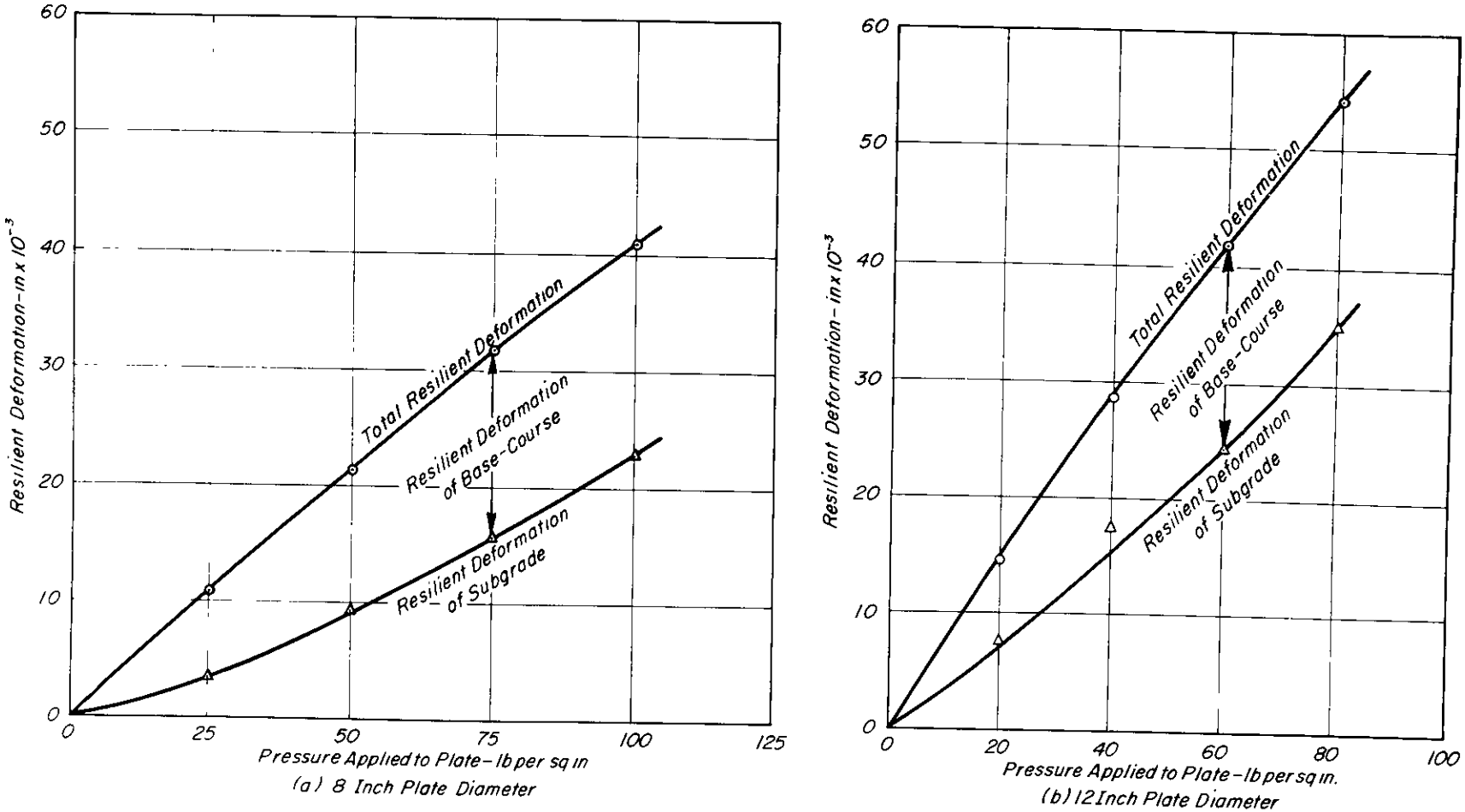


Fig. 46 — Relationship between applied pressure and resilient deformation of components of three-layer system consisting of subgrade, 11-in. of base and 2.4-in. of asphalt concrete for Section 5.

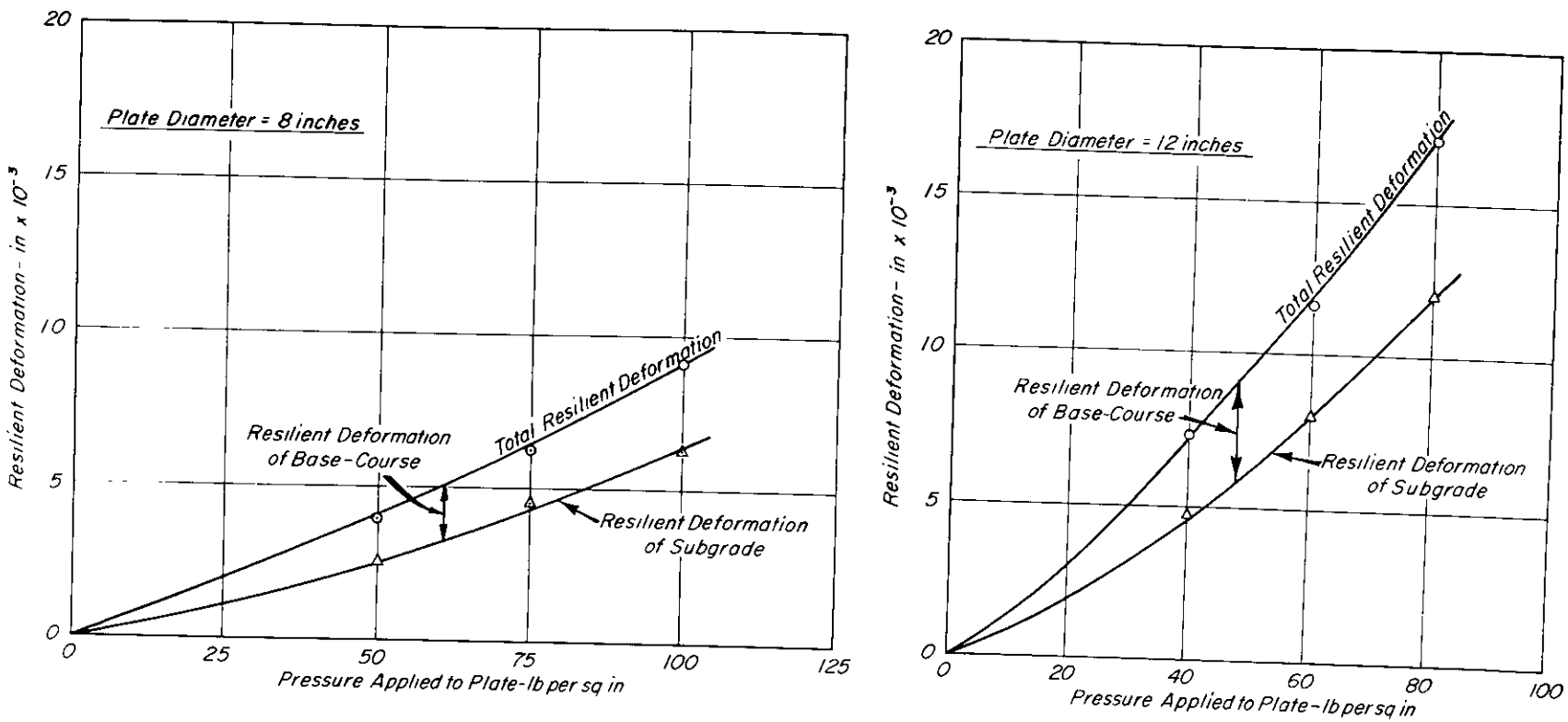


Fig. 47 — Relationship between applied pressure and resilient deformation of components of three-layer system of subgrade, 11-in. of base and 7.2-in. of asphalt concrete for Section 5.

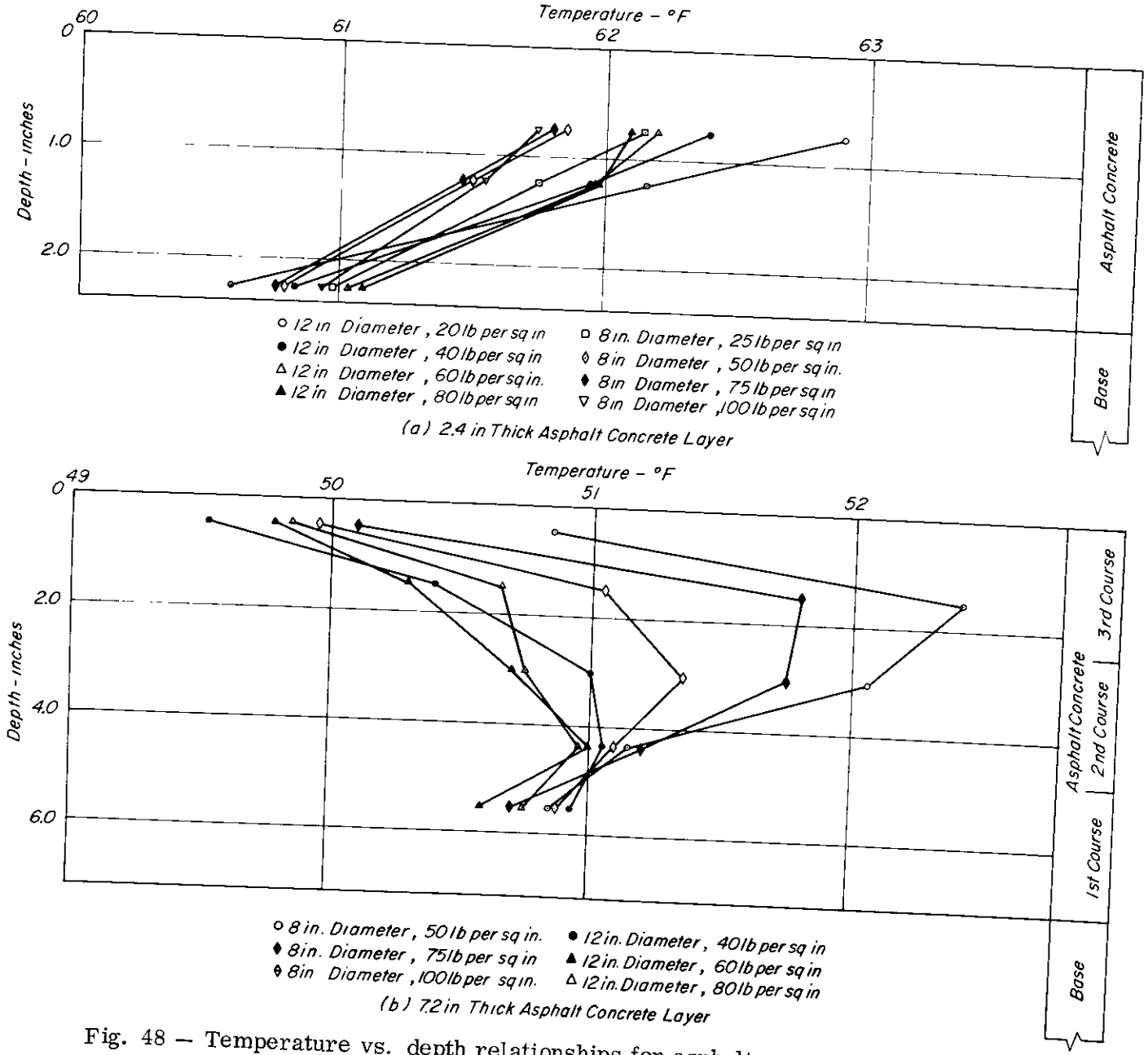


Fig. 48 — Temperature vs. depth relationships for asphalt concrete — Section 5.

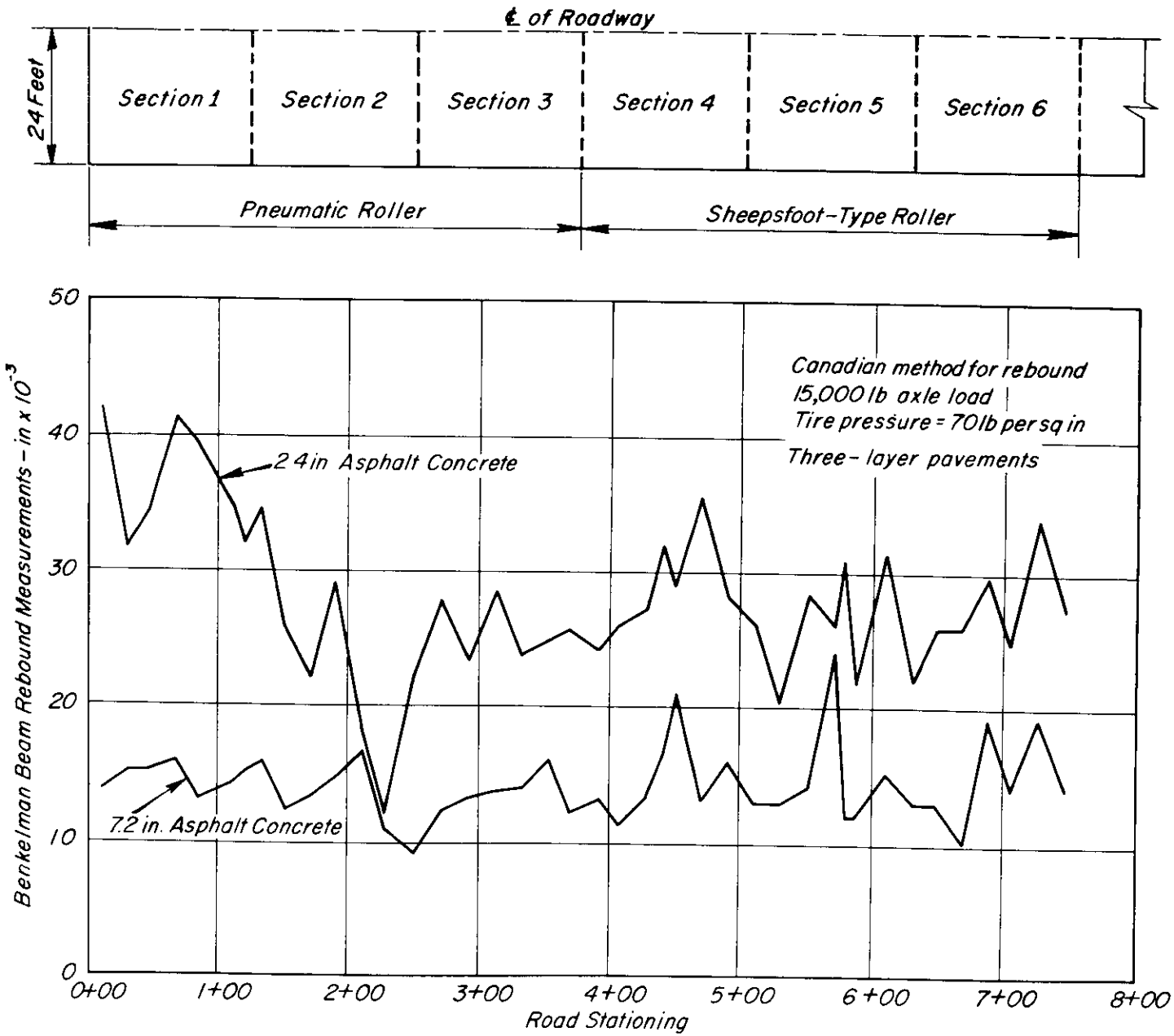


Fig. 49 — Benkelman beam rebound measurements for the test road.

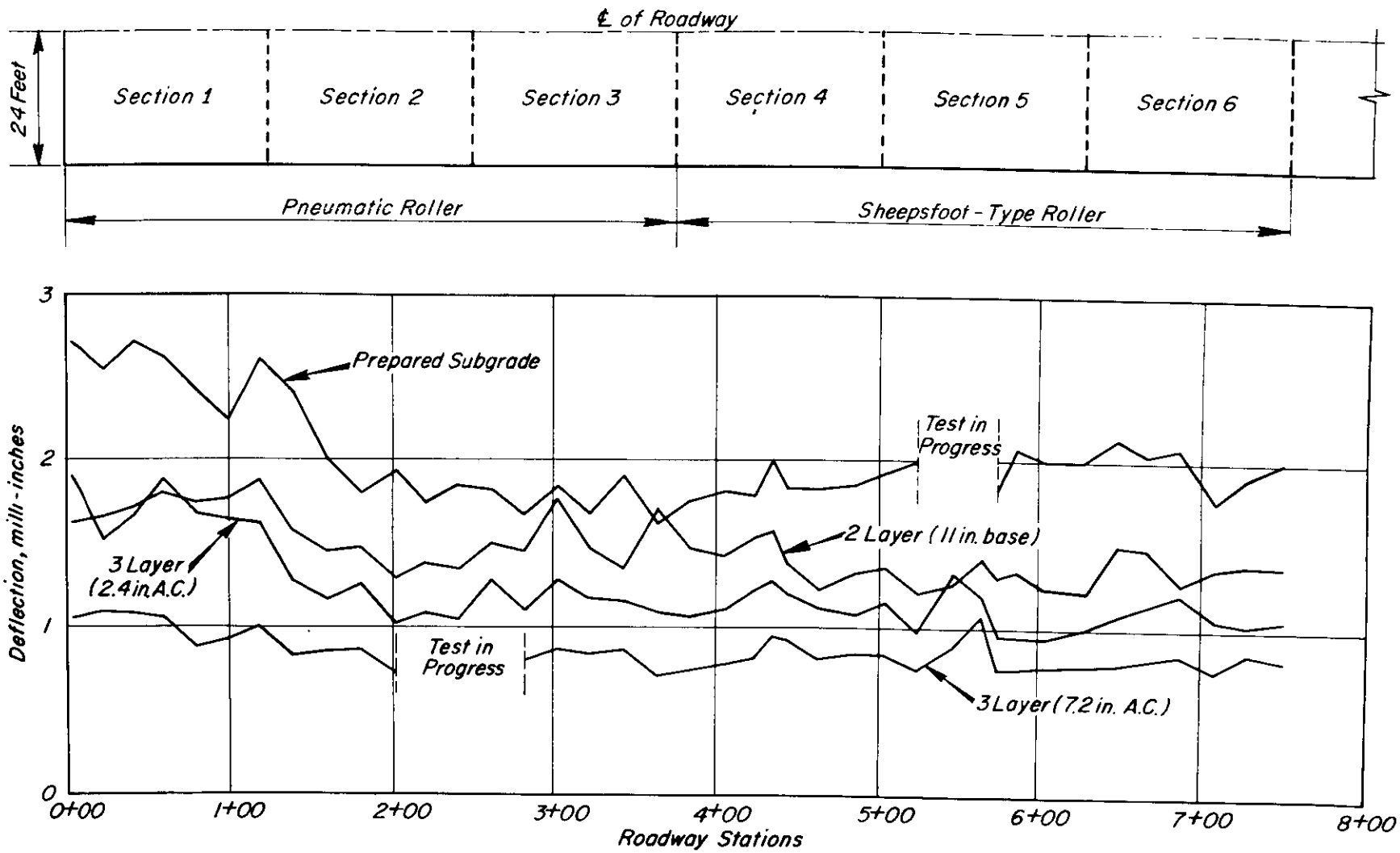
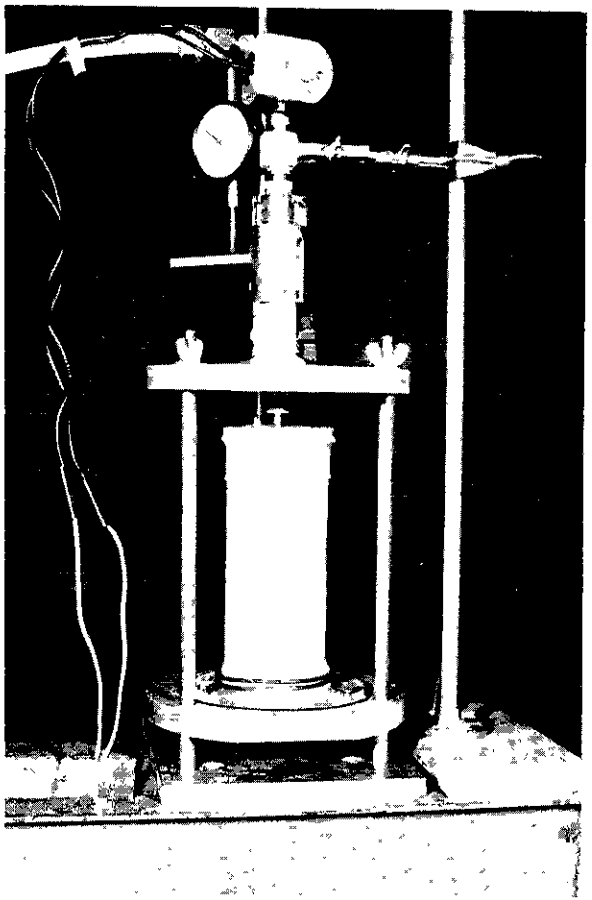
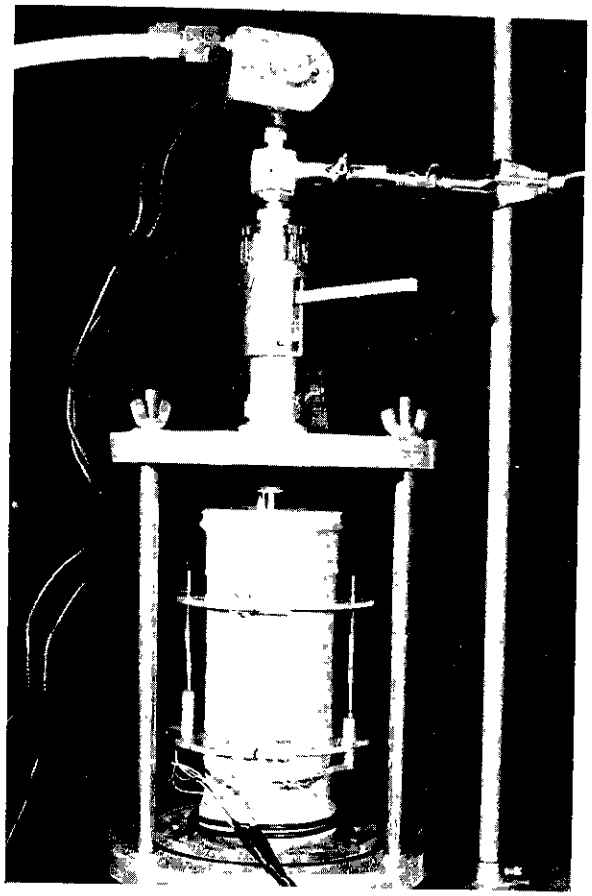


Fig. 50 — Dynaflect deflection measurements for test road.



a. Dial indicator used to measure deformation.



b. Dual LVDT system used to measure deformation.

Fig. 51 — Photographs of low stress repeated loading apparatus.

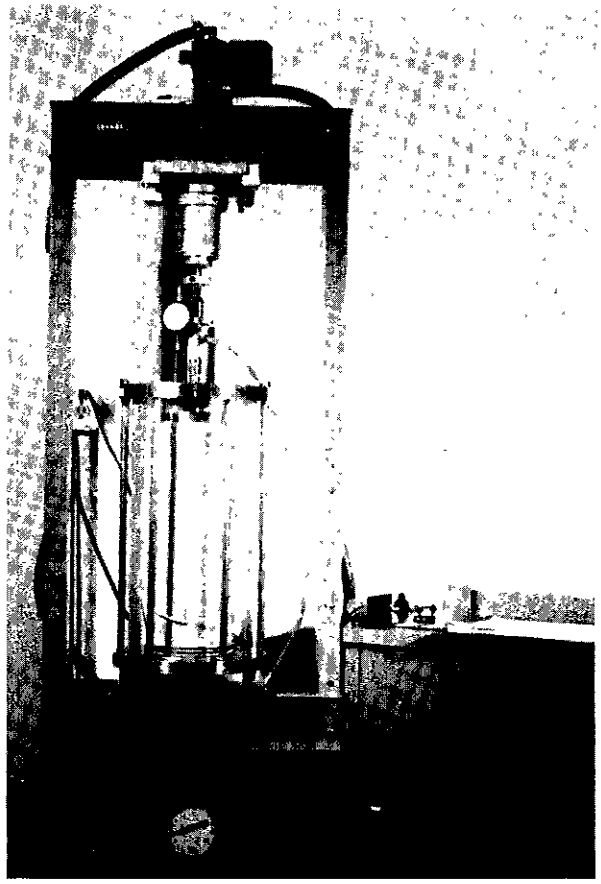


Fig. 52 — Photograph of apparatus for laboratory repeated load tests on aggregate base.

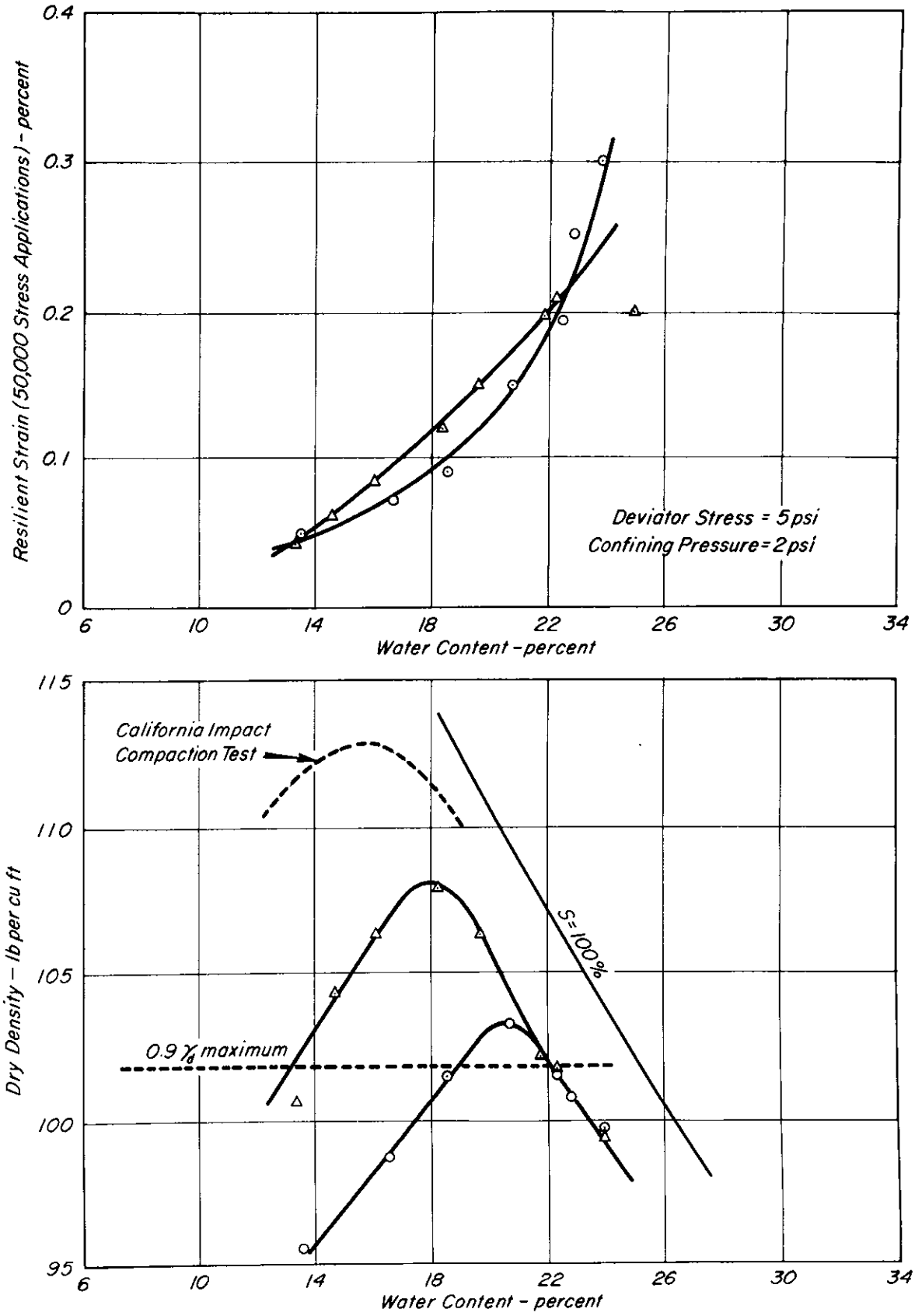


Fig. 53 — Relationship between dry density, water content and resilient strain for laboratory prepared specimens of subgrade soil.

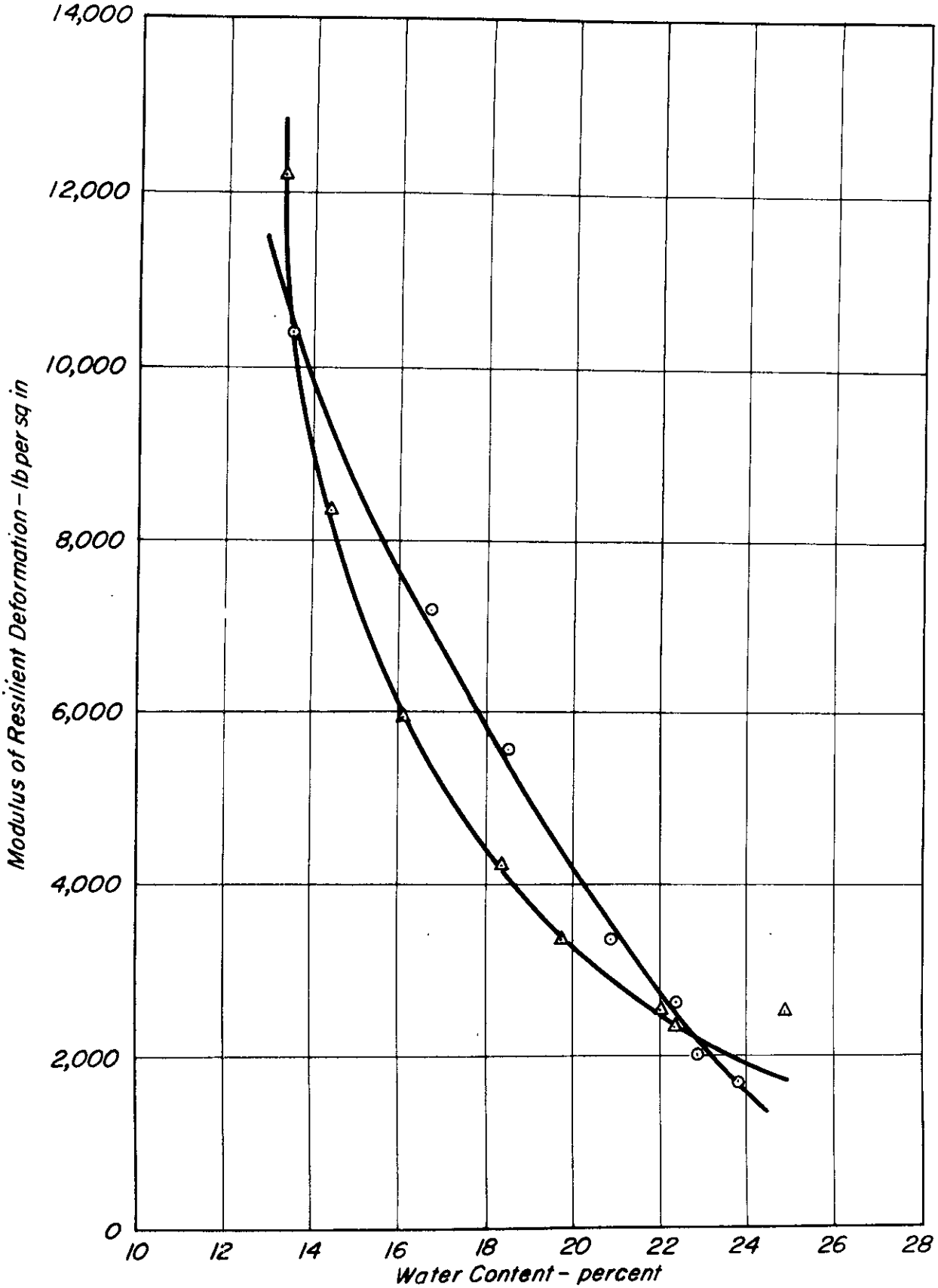


Fig. 54 — Relationship between water content and modulus of resilient deformation for laboratory prepared specimens of subgrade soil.

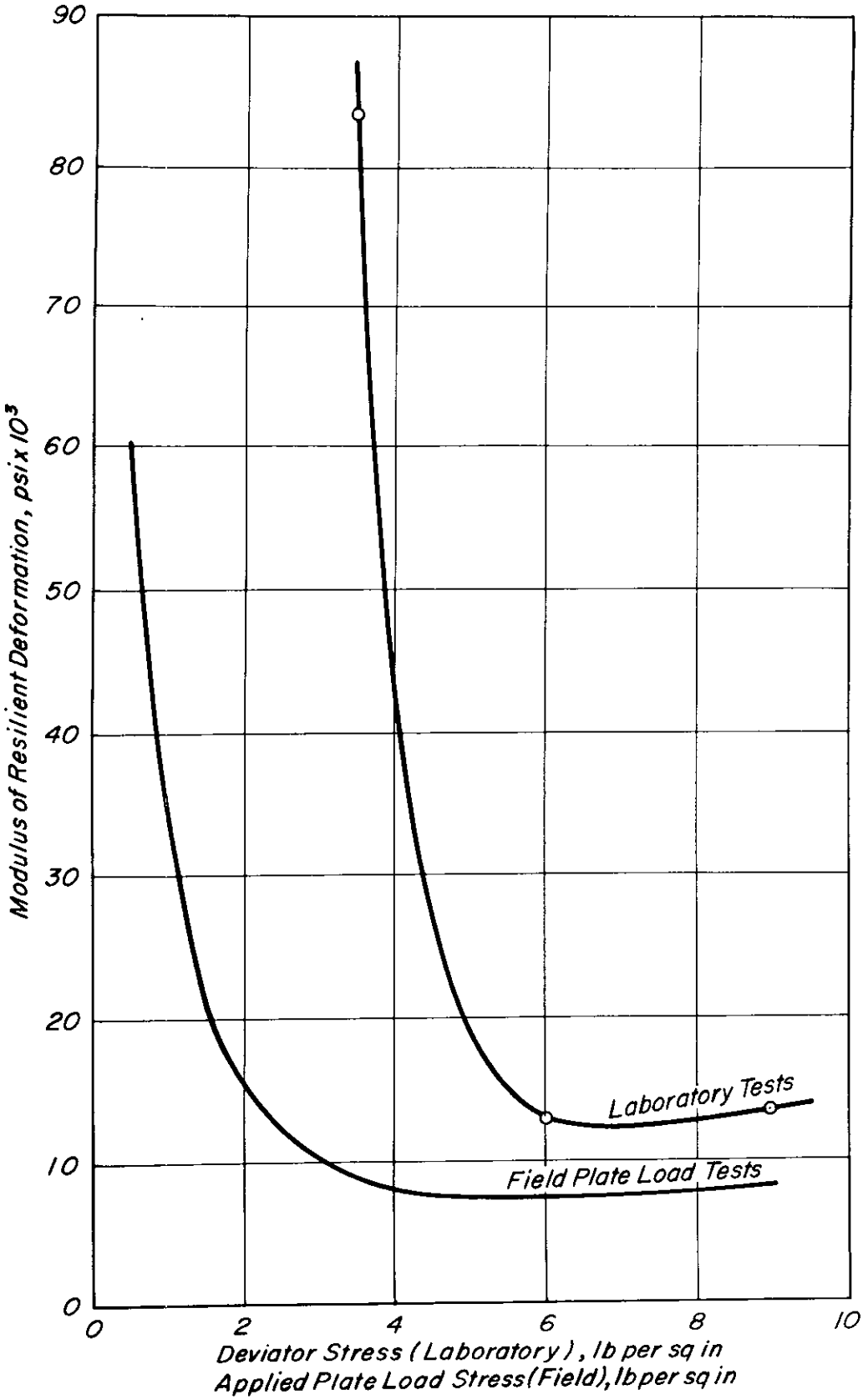


Fig. 55 — Relationship between applied stress and modulus of resilient deformation for laboratory and field tests on the subgrade material of Section 2.

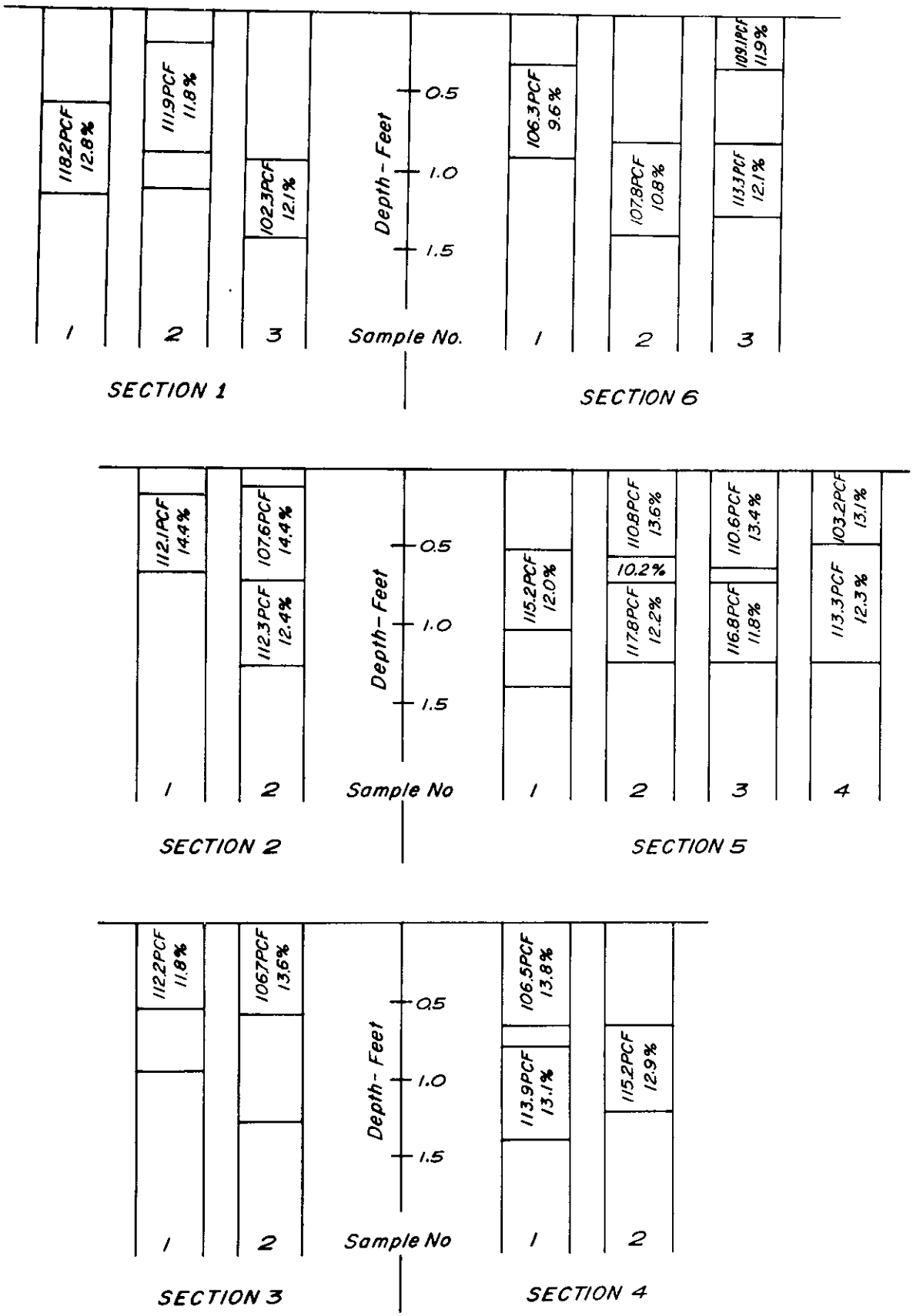


Fig. 56 — Water content and dry density as determined for each test section from 2.8-in. diameter samples.

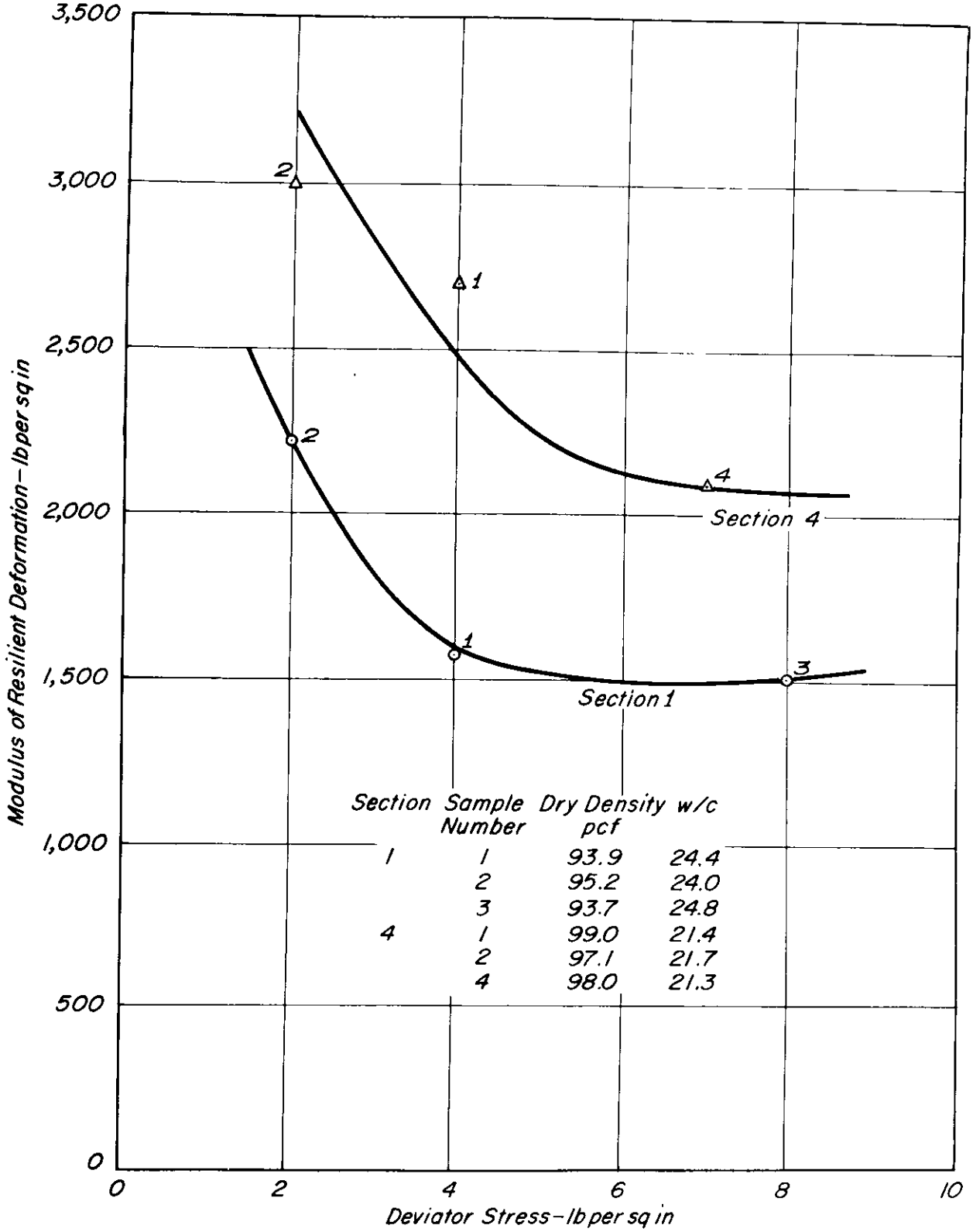


Fig. 57 — Relationship for deviator stress and modulus of resilient deformation for laboratory tests on specimens from highly saturated undisturbed samples.

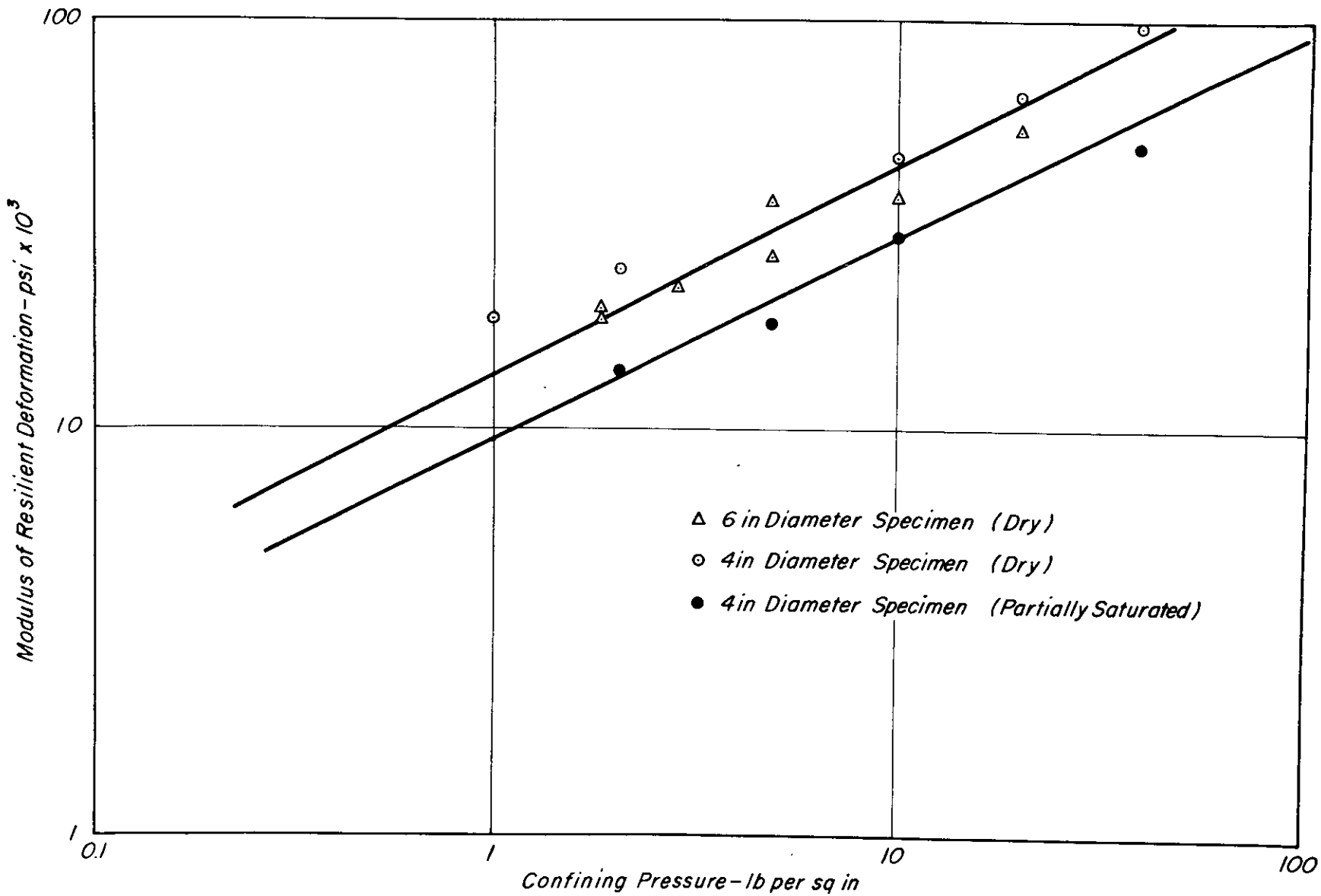


Fig. 58 — Relationship between modulus of resilient deformation and confining pressure for aggregate base (log-log plot).

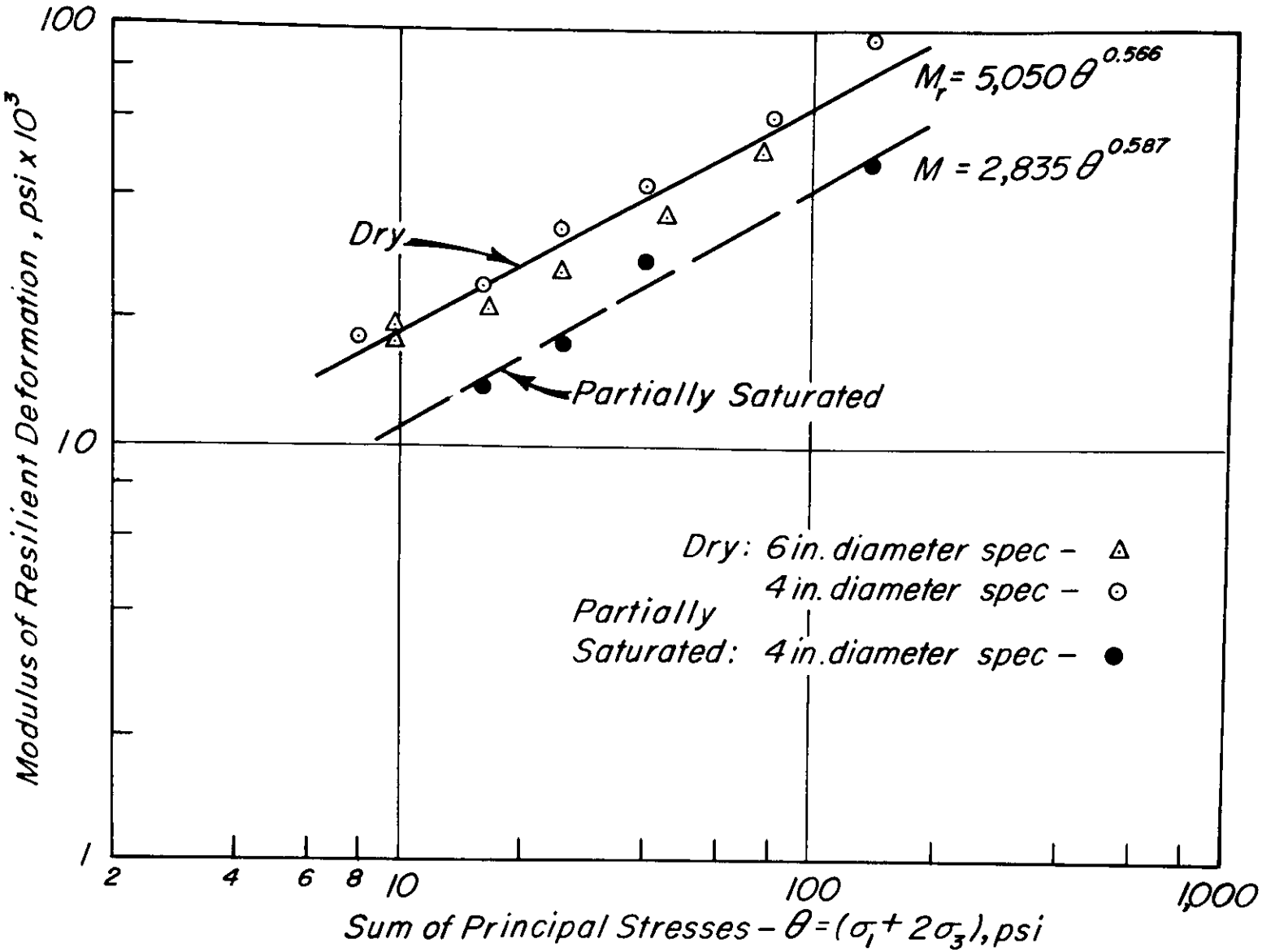


Fig. 59 — Relationship between resilient modulus and sum of principal stresses for untreated aggregate base.

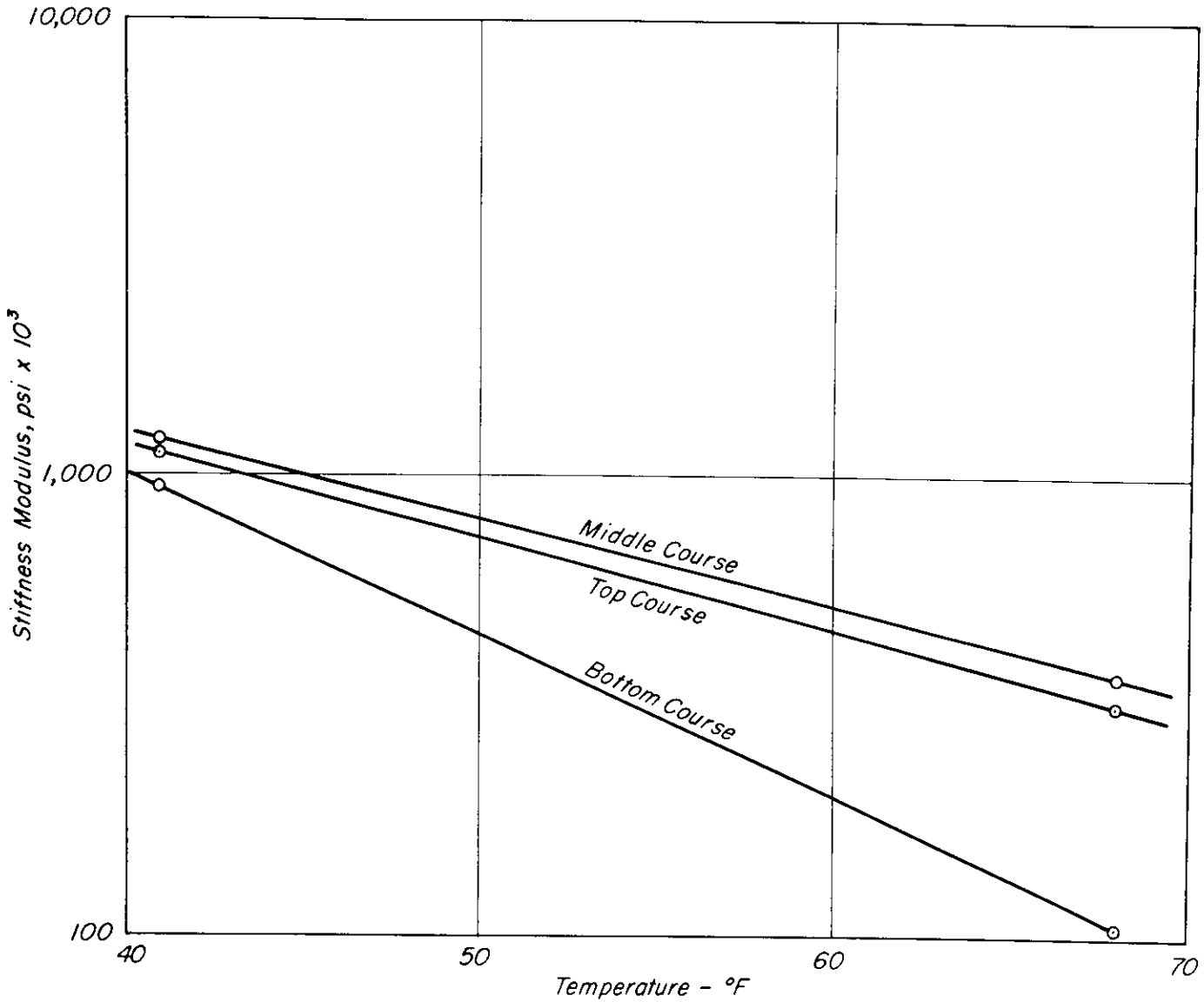


Fig. 60 – Influence of temperature on the stiffness modulus of specimens of asphalt concrete taken from the surfacing of the test road and tested in repeated flexure at a stress duration of 0.1 sec.

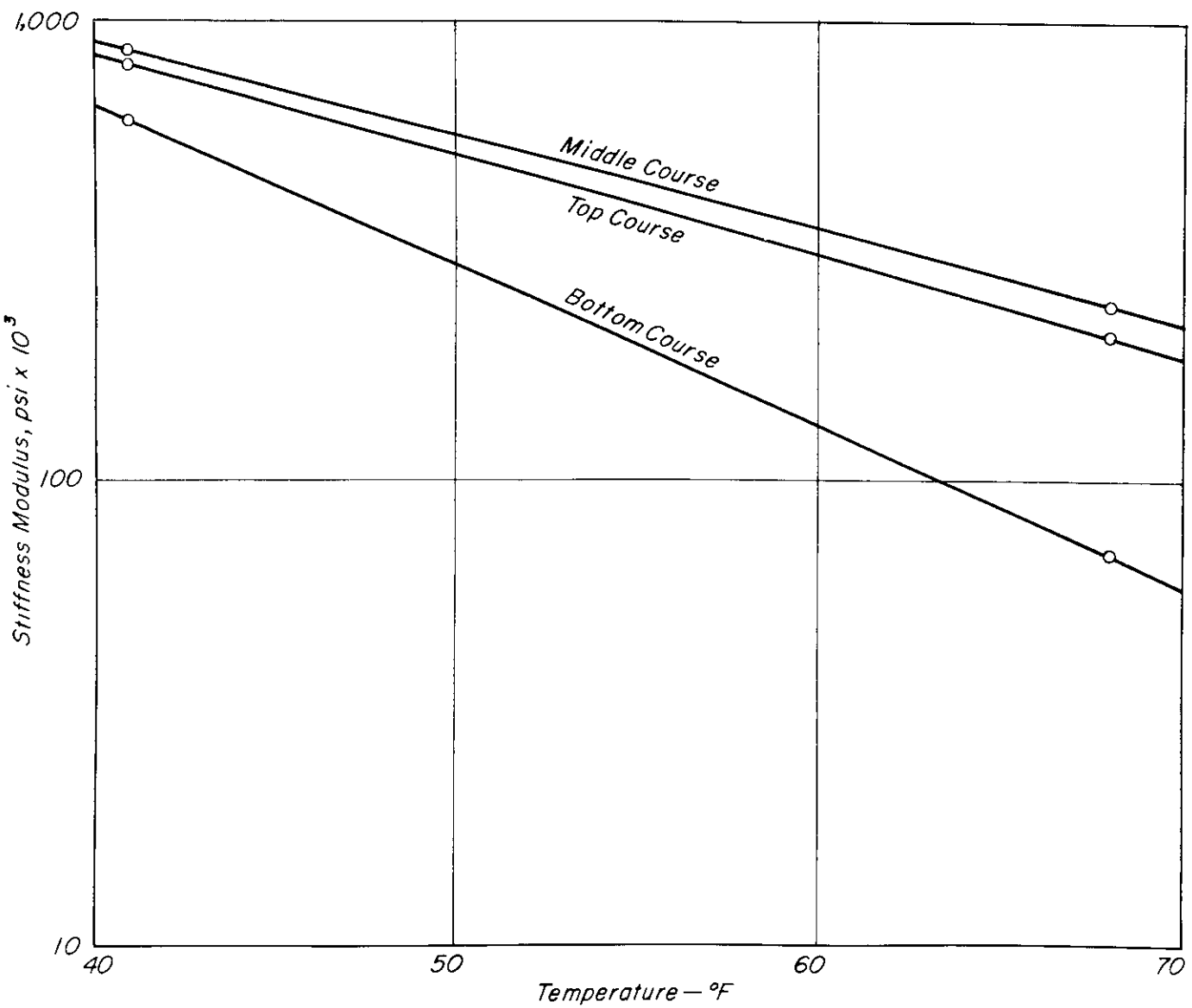


Fig. 61 – Influence of temperature on the stiffness modulus of specimens of asphalt concrete taken from the surfacing of the test road, tested in repeated flexure, and adjusted using a modified Van der Poel stiffness calculation for a stress duration of 0.25 sec.

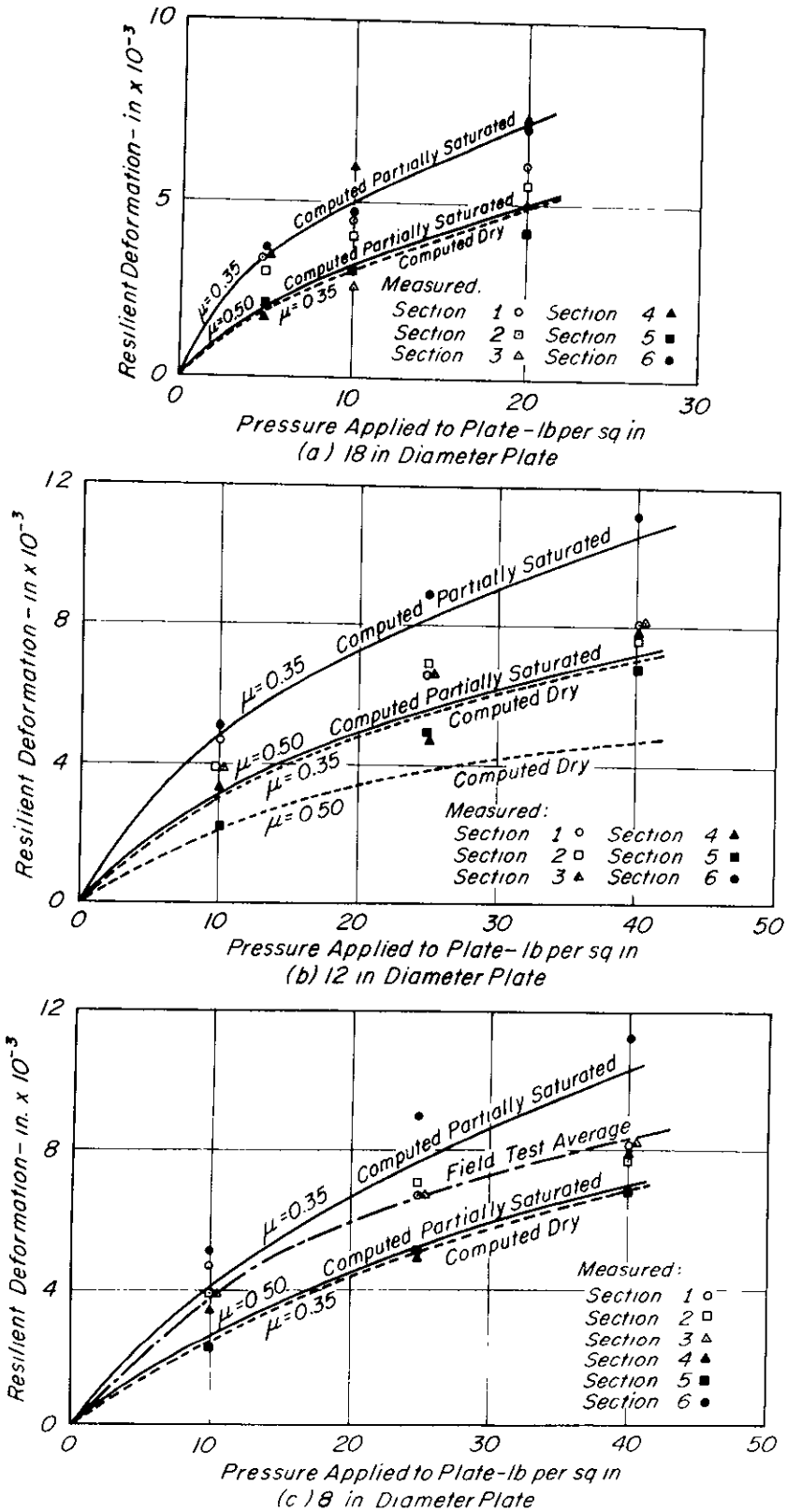


Fig. 62 – Comparisons between computed and measured resilient deformations of the base course in a two-layer system consisting of 11-in. of base and the subgrade.

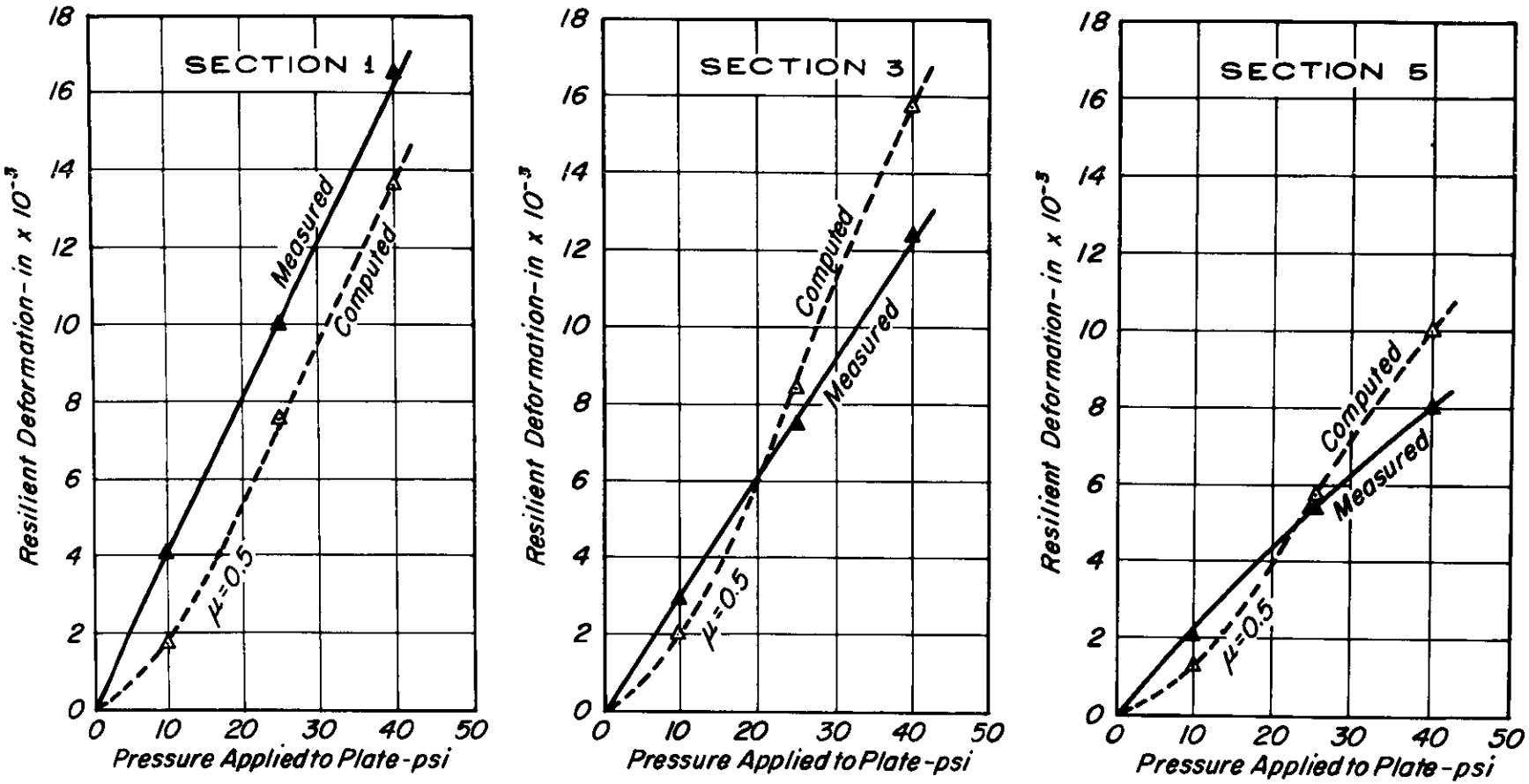


Fig. 63 — Comparison between measured and computed resilient deformations of the subgrade for a two-layer system consisting of 11-in. of base and the subgrade for a 12-in. plate.

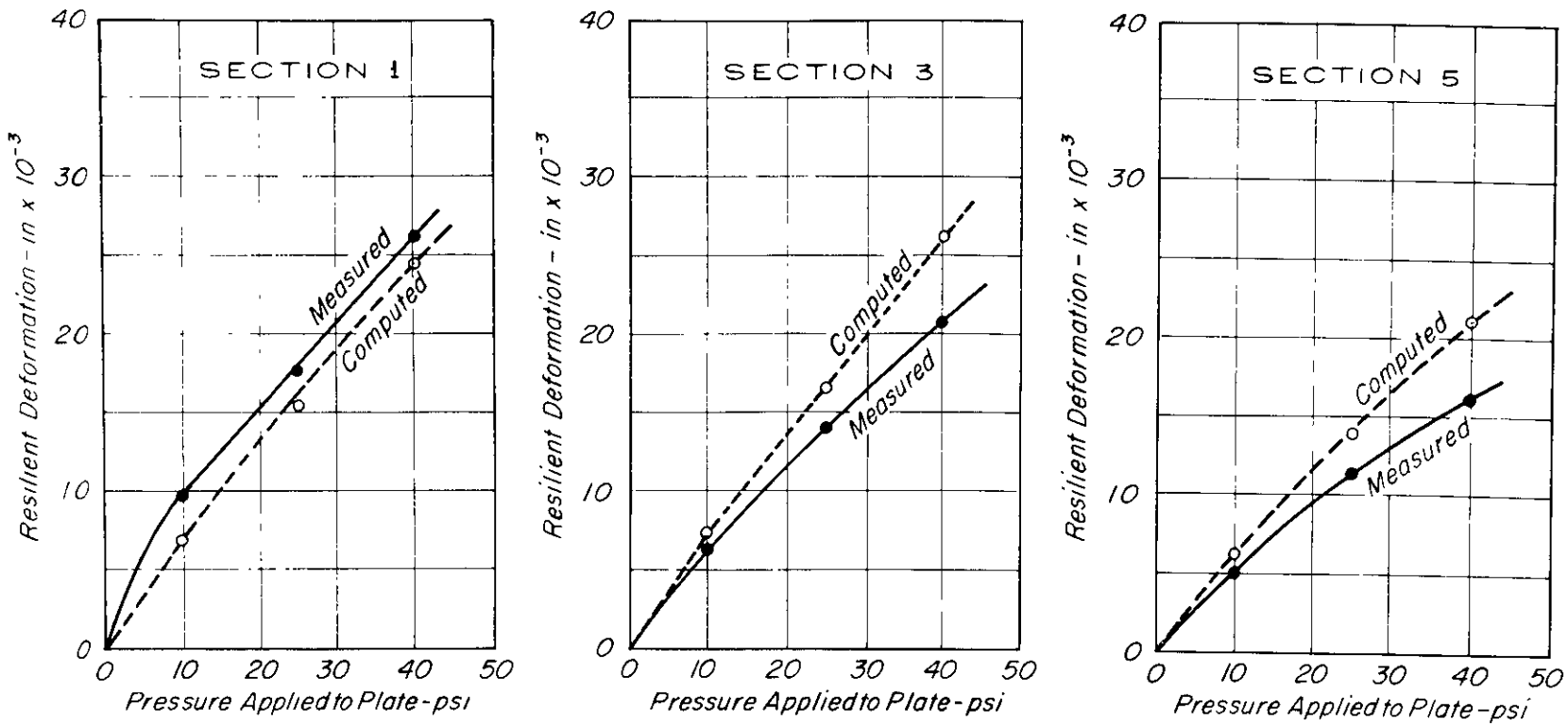


Fig. 64 — Comparison between measured and computed total resilient deformations for two-layer system consisting of 11-in. of base (μ = 0.35) and the subgrade (μ = 0.5) for a 12-in. plate.

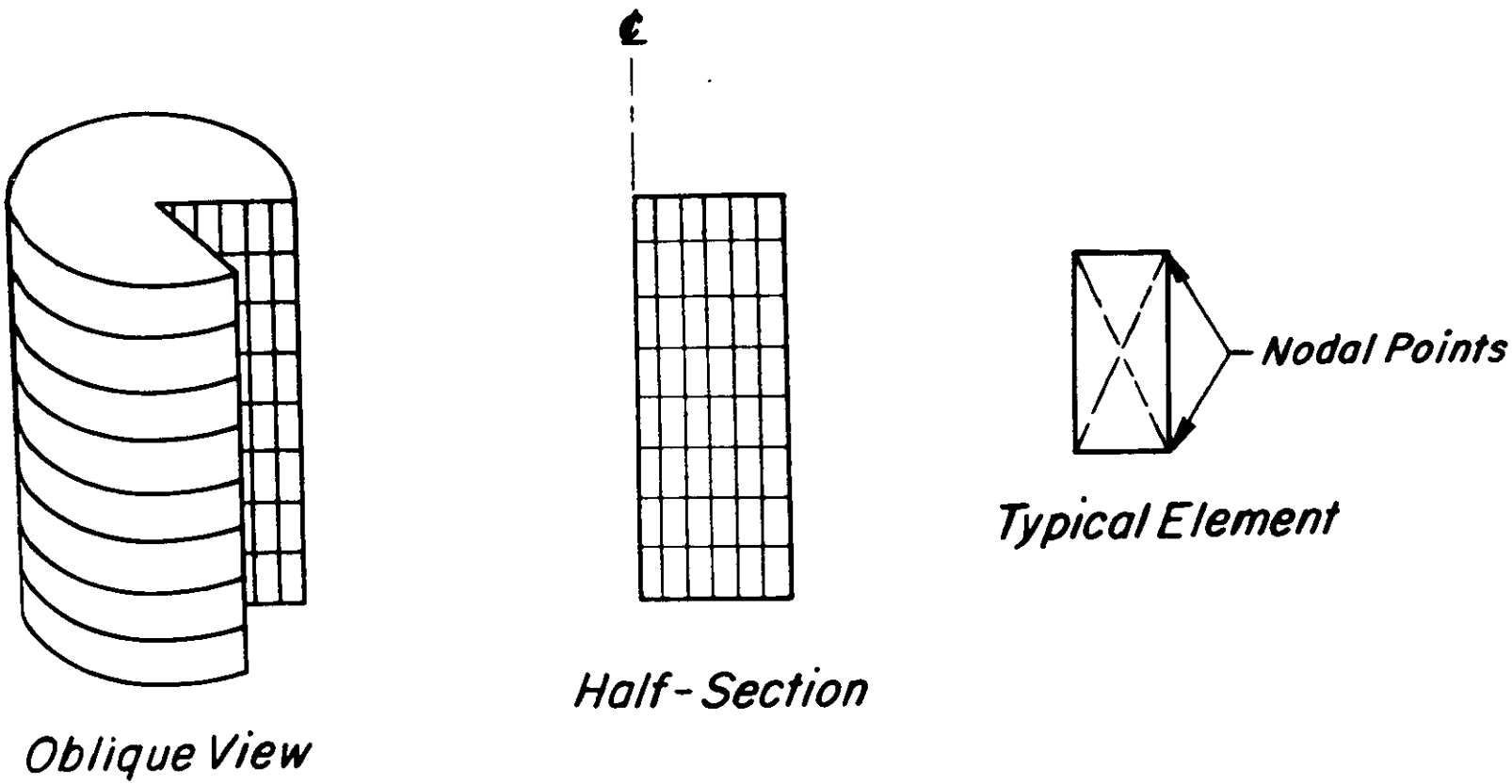


Fig. 65 — Finite element idealization of a cylinder

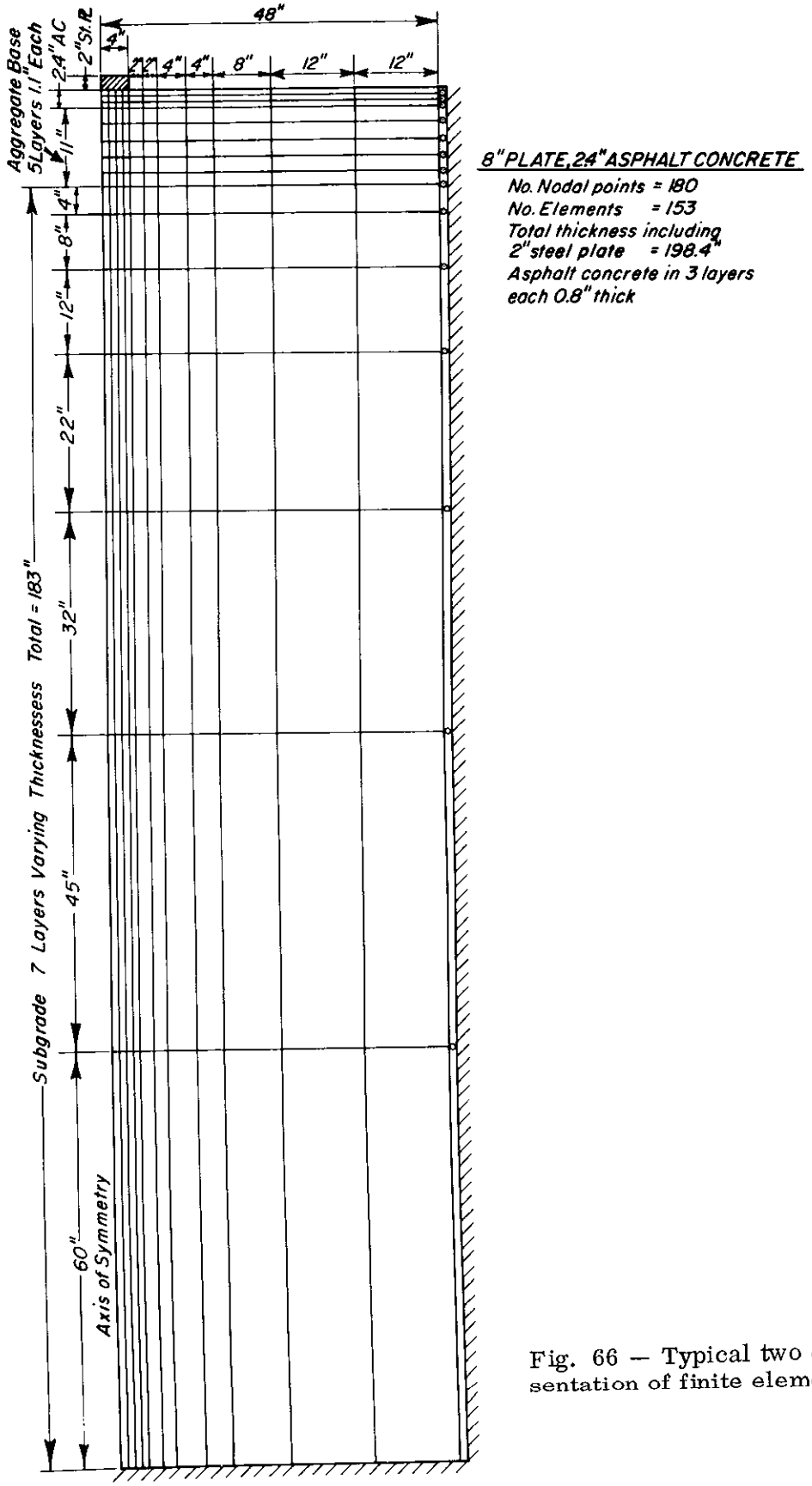


Fig. 66 — Typical two dimentional representation of finite element configuration.

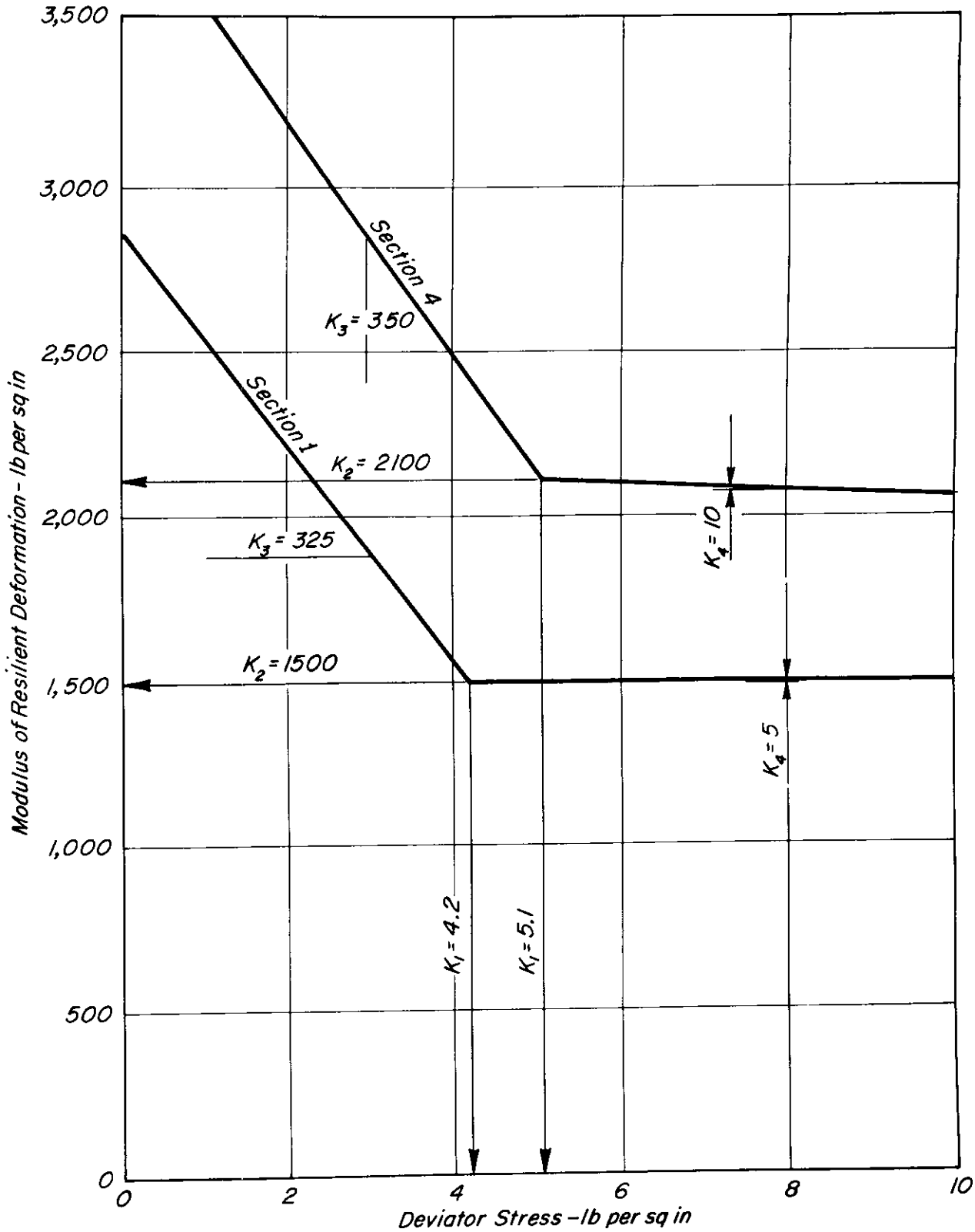


Fig. 67 — Bilinear material representation for subgrade materials of Sections 1 and 4 from Fig. 57.

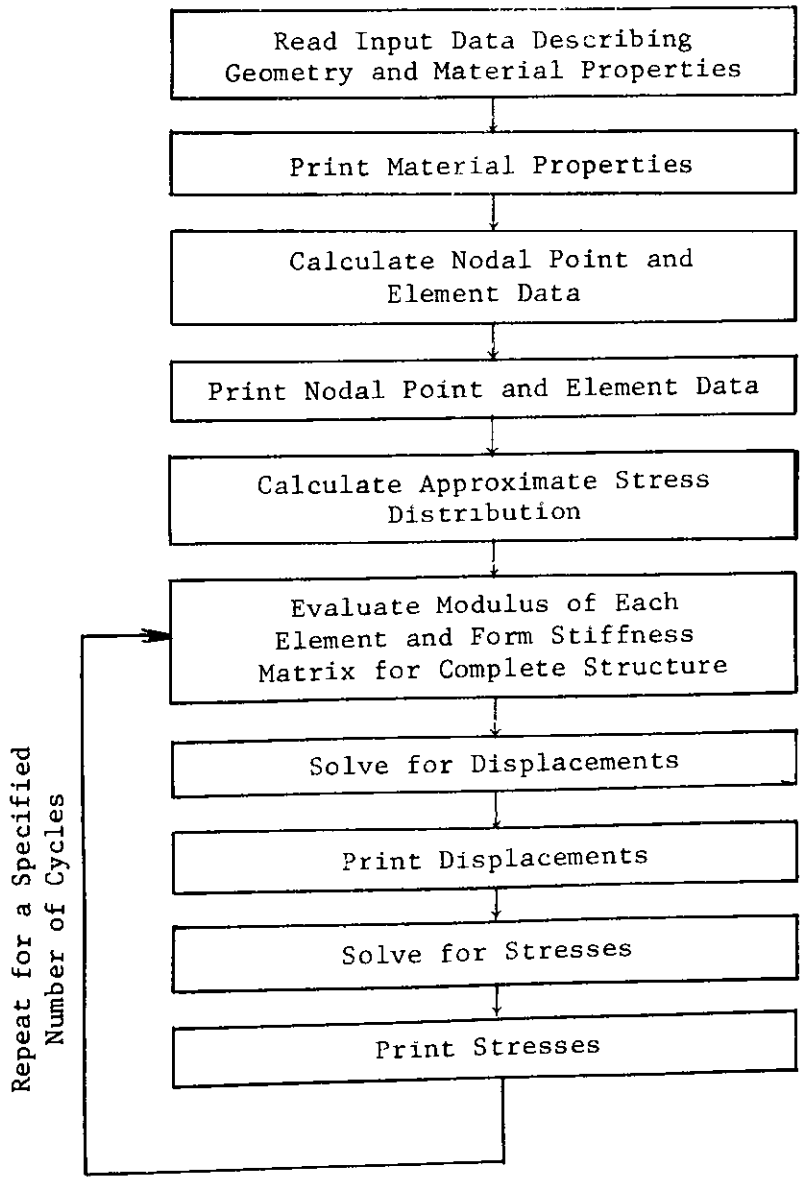


Fig. 68 – Simplified flow diagram.

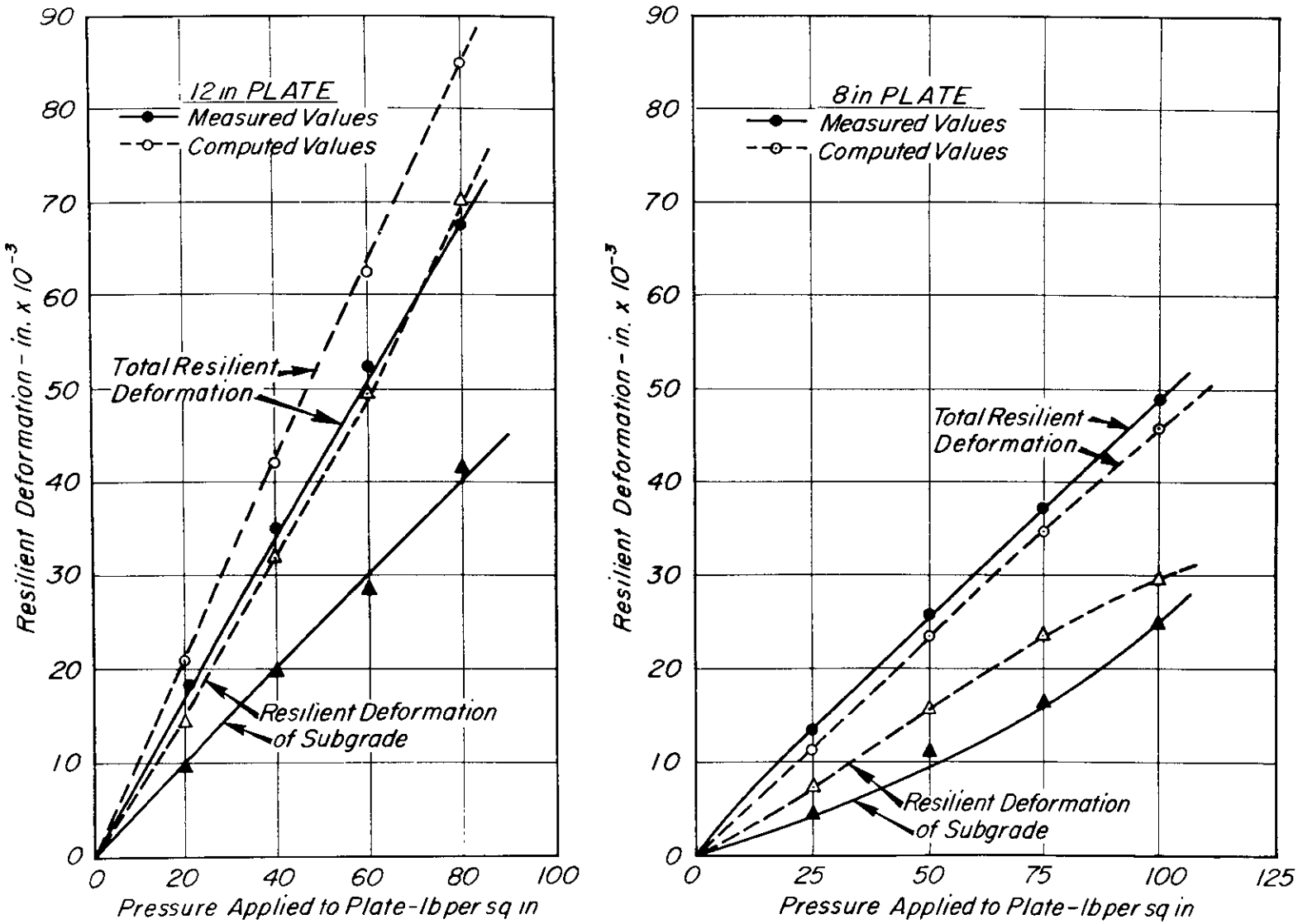


Fig. 69 — Comparison between measured and computed resilient deformations for three-layer pavement of Test Section 1 of the CCC Test Road (2.4-in. asphalt concrete surface).

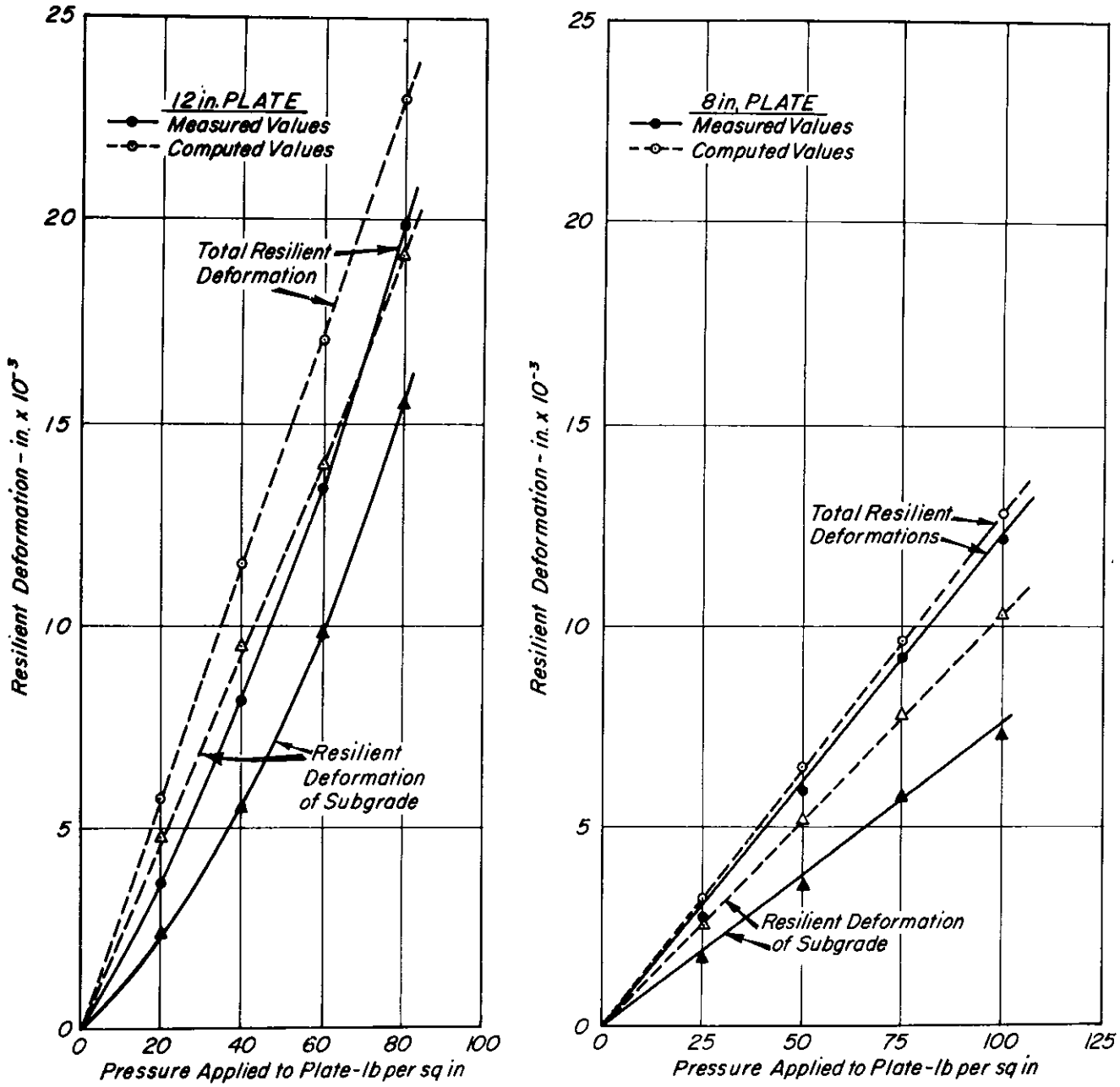


Fig. 70 — Comparison between measured and computed resilient deformations for three-layer pavement of Test Section 1 of the CCC Test Road (7.2-in. asphalt concrete surface).

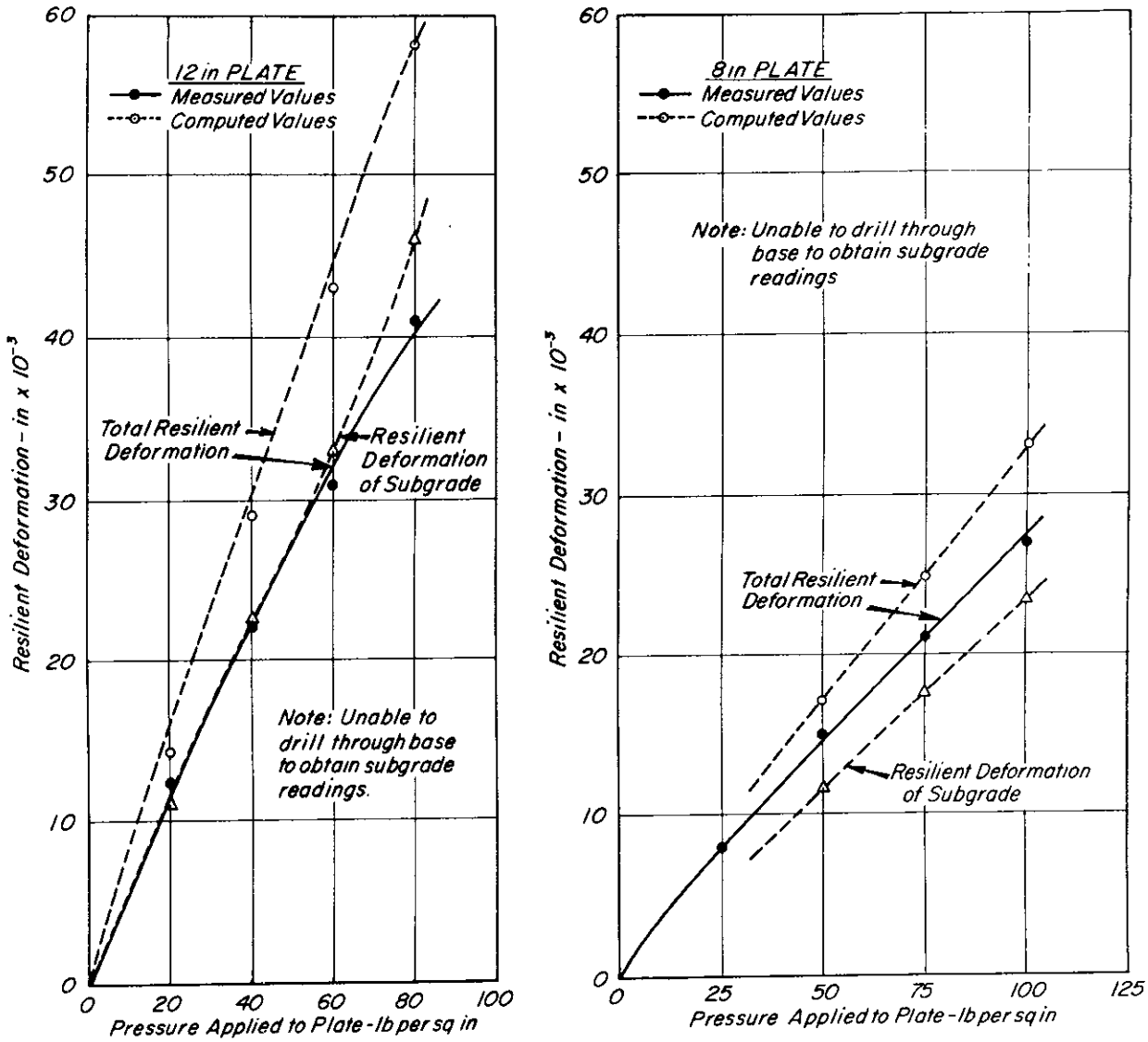


Fig. 71 – Comparison between measured and computed resilient deformations for three-layer pavement of Test Section 4 of the CCC Test Road (2.4-in. asphalt concrete surface).

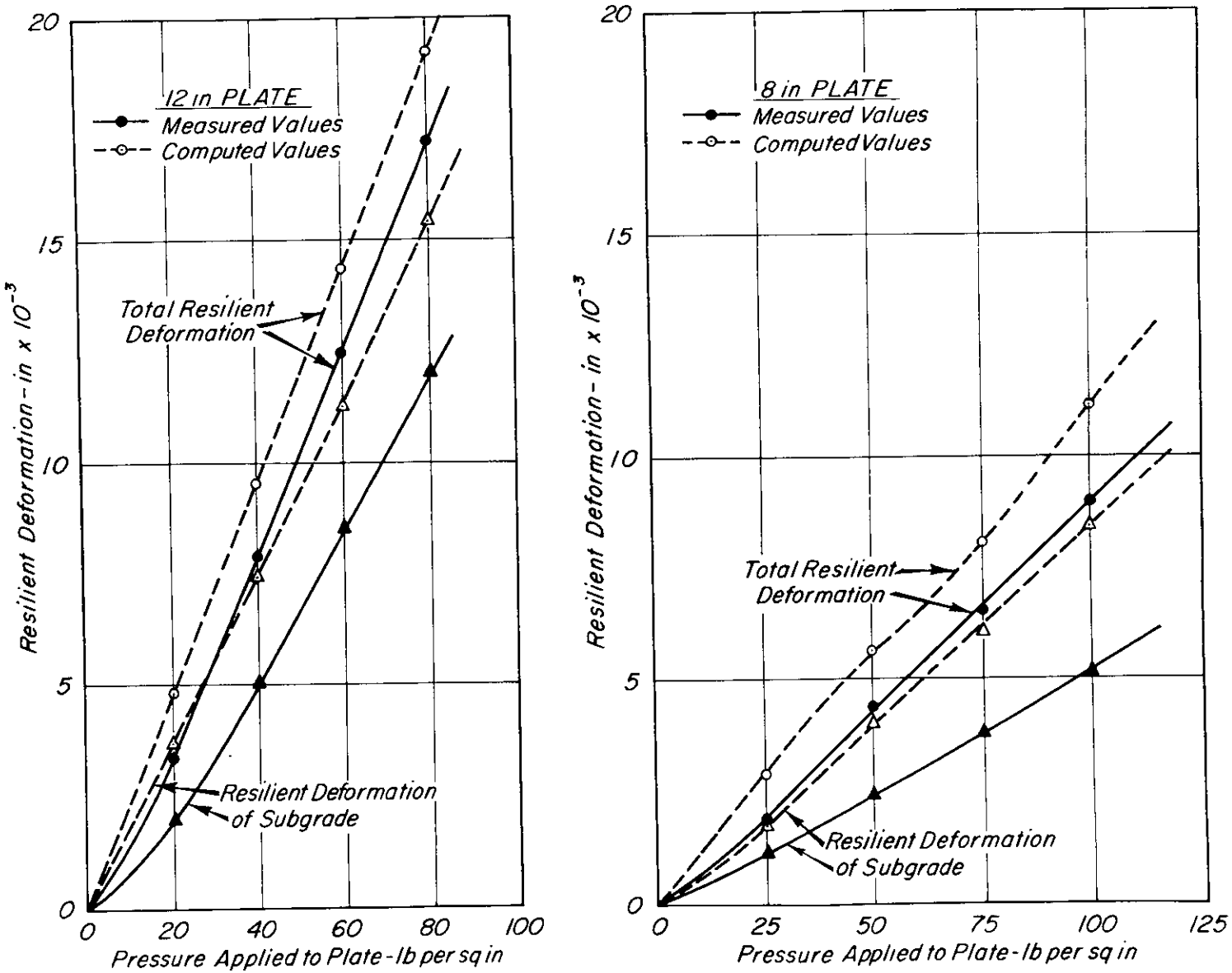


Fig. 72 -- Comparison between measured and computed resilient deformations for three-layer pavement of Test Section 4 of the CCC Test Road (7.2-in. asphalt concrete surface).

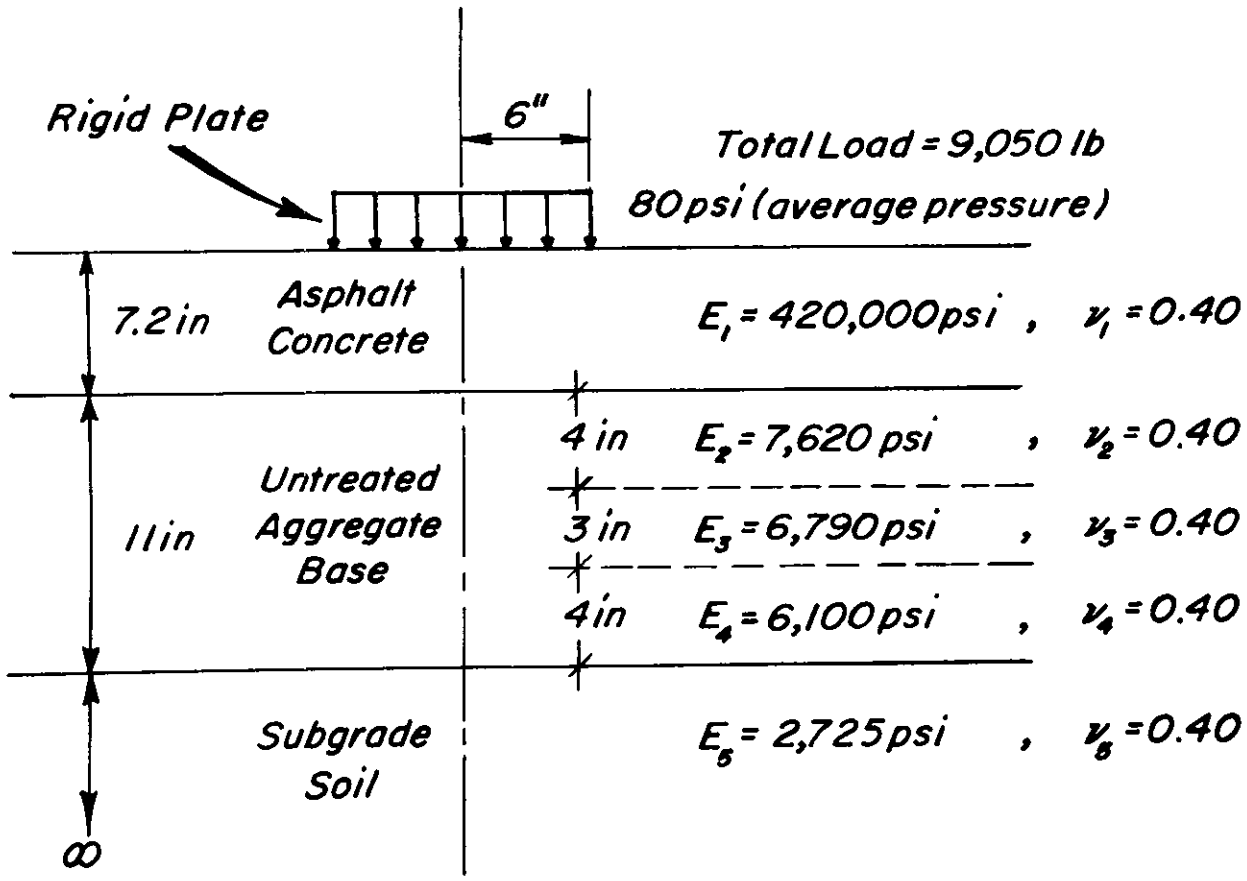


Fig. 73 — Characterization of 7.2-in. thick asphalt concrete pavement section (Section 4) as a five-layer elastic system.

APPENDIX A

TABLE A-1 — SUMMARY OF RELATIVE COMPACTION TESTS[1] BY NUCLEAR GAGE[2]

Test Section	Lift No.	Dry Density pcf	Water Content %	Relative Compaction %
1	1	94.1	14.2	83
	2	-	-	-
	3	93.3	16.3	83
2	1	94.7	17.2	84
	2	96.3	18.4	86
	3	93.2	16.6	83
3	1	103.1	17.3	91
	2	98.9	22.1	88
	3	93.4	16.5	83
4	1	86.9	17.7	77
	2	95.5	21.8	85
	3	104.3	18.2	92
5	1	95.4	17.5	85
	2	104.7	16.2	93
	3	99.2	14.8	88
6	1	-	-	-
	2	-	-	-
	3	103.2	14.3	92

1. Tests performed during construction by California Division of Highways Personnel.
2. Hidrodensimeter HDM-4.

TABLE A-2 — SUMMARY OF RELATIVE COMPACTION TESTS[1] BY SAND VOLUME AND OVEN DRYING[2]

Test Section	Lift No.	Dry Density pcf	Water Content %	Relative Compaction %
1	1	92.5	11.4	81.9
4	1	91.8	14.4	81.4
5	1	97.2	13.9	86.1

1. Tests performed during construction by Contra Costa County Personnel.
2. 400°F oven temperature, drying for 30 minutes.

APPENDIX B

TABLE B-1 — SUMMARY OF RESULTS OF FIELD TESTS (2 LAYER SECTION)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. x 10^{-3}	Subgrade in. x 10^{-3}	Base Course in. x 10^{-3}
1	18	5	7.0	3.5	3.5
	18	10	11.8	7.3	4.5
	18	20	22.6	16.5	6.1
	12	10	8.8	4.0	4.8
	12	25	17.9	10.0	7.9
	12	40	26.3	16.5	9.8
	8	10	6.2	1.5	4.7
	8	25	12.2	5.5	6.7
	8	40	17.1	9.0	8.1
2	18	5	6.2	3.0	3.2
	18	10	10.3	6.0	4.3
	18	20	18.2	12.5	5.7
	12	10	7.9	3.2	4.7
	12	25	15.5	8.5	7.0
	12	40	21.8	13.5	8.3
	8	10	5.1	1.2	3.9
	8	25	10.6	3.5	7.1
	8	40	14.3	5.7	8.6
3	18	5	4.3	2.4	1.9
	18	10	7.7	4.0	3.7
	18	20	16.0	13.0	5.0
	12	10	6.3	2.8	3.5
	12	25	13.8	7.5	6.3
	12	40	20.8	12.5	8.3
	8	10	5.2	1.2	4.0
	8	25	10.1	3.5	6.6
	8	40	13.9	5.7	8.2
4	18	5	5.7	3.0	2.7
	18	10	9.3	5.4	3.9
	18	20	16.5	10.0	6.5
	12	10	8.3	3.7	4.6
	12	25	14.5	8.4	6.1
	12	40	20.3	12.8	7.5
	8	10	5.1	1.8	3.3
	8	25	10.3	5.4	4.9
	8	40	13.9	6.0	7.9

TABLE B-1 (continued)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. × 10^{-3}	Subgrade in. × 10^{-3}	Base Course in. × 10^{-3}
5	18	5	3.8	1.5	2.3
	18	10	6.6	3.4	3.2
	18	20	12.6	8.3	4.3
	12	10	5.3	2.0	3.3
	12	25	11.3	5.5	5.8
	12	40	15.9	8.0	7.9
	8	10	3.1	0.8	2.3
	8	25	7.6	2.5	5.1
	8	40	10.8	9.0	6.8
6	18	5	6.8	3.0	3.8
	18	10	10.3	5.5	4.8
	18	20	17.4	10.2	7.2
	12	10	8.7	3.0	5.7
	12	25	15.8	7.0	8.8
	12	40	21.8	10.5	11.3
	8	10	6.3	1.0	5.3
	8	25	12.3	3.2	9.1
	8	40	16.6	5.4	11.2

TABLE B-2 — SUMMARY OF RESULTS OF FIELD TESTS
(3 LAYERED PAVEMENT) (2.4-IN. ASPHALT CONCRETE)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. × 10^{-3}	Subgrade in. × 10^{-3}	Base Course in. × 10^{-3}
1	12	20	17.0	9.3	7.7
Note 1	12	40	33.9	19.0	14.9
	12	60	52.2	27.5	24.7
	12	80	67.0	46.0	21.0
	8	25	13.7	4.5	9.2
	8	50	25.2	10.5	14.7
	8	75	37.2	18.0	19.2
	8	100	48.9	25.0	23.9
2	12	20	9.4	4.0	5.4
	12	40	16.2	8.3	7.9
	12	60	23.0	13.0	10.0
Note 2	12	80	29.2	17.8	11.4
	8	25	6.7	2.0	4.7
	8	50	12.3	4.5	7.8
	8	75	17.0	7.0	10.0
	8	100	21.5	10.0	11.5
3	12	20	9.3	4.3	5.0
	12	40	17.0	8.5	8.5
Note 1	12	60	24.8	13.8	11.0
	12	80	34.3	19.5	14.8
	8	25	5.4	1.5	6.9
	8	50	13.3	4.5	8.8
	8	75	19.4	8.8	10.6
	8	100	25.8	11.8	14.0
4	12	20	11.8	See note 3	
	12	40	21.7		
Note 1	12	60	31.3		
	12	80	40.7		
	8	25	7.8		
	8	50	14.8		
	8	75	20.7		
	8	100	27.2		

TABLE B-2 (continued)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. × 10^{-3}	Subgrade in. × 10^{-3}	Base Course in. × 10^{-3}
5	12	20	14.9	7.5	7.4
	12	40	28.8	17.5	11.3
	12	60	41.7	24.5	17.2
	12	80	54.3	35.0	19.3
	8	25	10.8	3.5	7.3
	8	50	21.2	9.5	11.7
	8	75	31.5	15.5	16.0
	8	100	40.7	22.5	18.2
6	12	20	12.1	6.8	5.3
	12	40	25.5	14.5	11.0
Note 1	12	60	39.3	22.8	16.5
	12	80	52.5	32.0	20.5
	8	25	9.1	3.5	5.6
	8	50	19.1	9.0	10.1
	8	75	29.0	14.0	15.0
	8	100	37.8	20.0	17.8

1. Upon drilling the hole for the subgrade-base interface deflection reading, the base at sections 1, 3, 4, and 6 appeared to contain free water.
2. 12-in. plate 80 psi pressure test section 2 discontinued after 80 applications as loading moved reaction beam out of position.
3. Unable to drill hole through base - apparently due to water in the base.

TABLE B-3 — SUMMARY OF RESULTS OF FIELD TEST
(3 LAYERED PAVEMENT) (7.2-IN. ASPHALT CONCRETE)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. × 10^{-3}	Subgrade in. × 10^{-3}	Base Course in. × 10^{-3}
1	12	20	3.6	2.4	1.2
	12	40	8.1	5.5	2.6
	12	60	13.3	9.8	3.5
	12	80	19.9	15.5	4.4
	8	25	2.7	1.6	1.1
	8	50	5.8	3.5	2.3
	8	75	9.3	5.8	3.5
	8	100	12.1	7.3	4.8
2	12	20	2.4	1.5	0.9
	12	40	5.3	3.3	2.0
	12	60	9.0	5.5	3.5
	12	80	12.5	8.5	4.0
	8	25	1.5	0.6	0.9
	8	50	3.3	1.7	1.6
	8	75	5.6	2.8	2.8
	8	100	7.4	3.8	3.6
3	12	20	3.2	2.0	1.2
	12	40	7.2	4.8	2.4
	12	60	11.3	8.0	3.3
	12	80	17.2	11.8	5.4
	8	25	1.5	0.8	0.7
	8	50	3.9	2.3	1.6
	8	75	6.0	3.3	2.7
	8	100	8.7	5.5	3.2
4	12	20	3.3	2.0	1.3
	12	40	7.6	5.0	1.6
	12	60	12.2	8.5	3.7
	12	80	17.2	11.5	5.7
	8	25	2.0	1.3	0.7
	8	50	4.3	2.3	2.0
	8	75	6.4	3.8	2.6
	8	100	8.9	5.0	3.9

TABLE B-3 (continued)

Section Tested	Plate Diameter in.	Pressure Applied on Plate psi	Resilient Deformation		
			Total in. × 10^{-3}	Subgrade in. × 10^{-3}	Base Course in. × 10^{-3}
5	12	20	-	-	-
	12	40	7.3	4.8	2.5
	12	60	11.5	8.0	3.5
	12	80	17.3	12.0	5.3
	8	25	-	-	-
	8	50	4.0	2.5	1.5
	8	75	6.2	4.5	1.7
	8	100	9.0	6.0	3.0
6	12	20	2.4	1.5	0.9
	12	40	5.4	3.5	1.9
	12	60	8.4	5.8	2.6
	12	80	12.4	8.3	4.1
	8	25	1.2	0.6	0.6
	8	50	2.7	1.5	1.2
	8	75	4.3	2.5	1.8
	8	100	6.4	3.5	2.9

TABLE B-4 — SUMMARY OF TEMPERATURE GRADIENT FOR REPEATED PLATE LOAD TEST ON 2.4-IN. ASPHALT CONCRETE

Section	Plate Size Diameter in.	Pressure psi	Temperature Location			
			Air	0.5 in.	1.0 in.	1.75 in.
1	12	20	70.7	73.2	72.2	72.6
	12	40	70.8	73.0	73.0	72.0
	12	60	69.7	72.3	72.4	71.5
	12	80	69.1	71.6	72.1	71.8
	8	25	68.4	71.4	71.7	71.7
	8	50	66.4	70.5	70.5	71.0
	8	75	64.1	69.2	69.6	70.4
	8	100	61.5	67.1	68.4	69.2
			Air	0.375 in.	1.0 in.	1.75 in.
2	12	20	67.2	69.9	69.5	68.5
	12	40	68.9	69.1	68.6	68.2
	12	60	69.0	68.8	68.4	67.9
	12	80	68.6	69.6	69.4	69.4
	8	25	68.4	69.8	69.9	69.2
	8	50	68.1	69.1	69.3	69.1
	8	75	68.4	69.2	68.9	68.6
	8	100	68.0	68.8	68.5	68.5
			Air	0.5 in.	1.0 in.	2.0 in.
3	12	20	52.3	55.1	55.1	54.6
	12	40	53.6	54.5	54.3	54.2
	12	60	54.6	54.7	54.4	54.1
	12	80	55.8	55.5	54.9	54.6
	8	25	56.3	56.3	55.5	55.1
	8	50	55.4	55.7	55.5	55.2
	8	75	55.8	55.4	55.4	55.3
	8	100	57.2	55.9	55.8	55.4
			Air	0.625 in.	1.0 in.	2.0 in.
4	12	20	55.8	57.9	58.1	57.3
	12	40	56.8	57.4	57.3	57.3
	12	60	57.1	57.3	57.2	57.2
	12	80	57.2	58.0	57.9	57.5
	8	25	58.2	58.3	58.2	57.8
	8	50	58.2	57.8	57.6	57.5
	8	75	57.7	57.6	57.3	57.4
	8	100	57.8	57.5	57.2	57.3

TABLE B-4 (continued)

Section	Plate Size Diameter in.	Pressure psi	Temperature Location			
			Air	0.75 in.	1.25 in.	2.25 in.
5	12	20	62.7	62.9	62.2	60.6
	12	40	62.0	62.4	62.0	60.9
	12	60	62.1	62.2	62.0	61.1
	12	80	62.4	62.1	62.0	61.1
	8	25	62.6	62.2	61.8	61.0
	8	50	62.1	61.9	61.5	60.8
	8	75	62.6	61.8	61.5	60.8
	8	100	63.0	61.8	61.5	61.0
			Air	0.625 in.	1.0 in.	2.0 in.
6	12	20	58.5	60.8	60.8	60.2
	12	40	58.8	60.4	60.5	60.0
	12	60	59.1	60.4	60.6	60.1
	12	80	59.3	61.0	60.3	60.7
	8	25	59.4	61.3	61.6	61.1
	8	50	Broken Thermocouple			
	8	75				
	8	100				

TABLE B-5 — SUMMARY OF TEMPERATURE GRADIENT FOR REPEATED PLATE LOAD TESTS ON 7.2-IN. ASPHALT CONCRETE

Section	Plate Size Diameter in.	Pressure psi	Temperature Location						
			Air	0.5 in.	1.0 in.	1.75 in.	3.0 in.	4.5 in.	5.5 in.
1	12	20	52.9	53.5	53.7	53.8	52.7	51.6	51.6
	12	40	53.8	53.7	53.5	53.2	52.1	51.3	50.9
	12	60	54.7	54.6	54.3	53.3	52.2	51.3	50.9
	12	80	54.2	54.5	54.4	53.7	52.5	51.7	51.3
	8	25	54.0	54.6	54.5	53.7	52.8	52.1	51.7
	8	50	53.5	54.2	53.3	53.9	53.2	52.4	52.0
	8	75	52.6	53.2	53.3	53.3	52.9	52.4	52.1
	8	100	52.0	52.4	52.5	52.7	52.5	52.3	52.0
			Air	0.564 in.	1.25 in.	2.25 in.	4.375 in.	5.75 in.	
2	12	20	46.4	47.4	47.3	47.0	46.5	46.8	
	12	40	46.3	46.9	46.2	47.1	46.8	47.2	
	12	60	47.3	47.4	47.3	47.1	47.0	47.2	
	12	80	48.1	48.5	48.2	47.6	47.4	47.6	
	8	25	48.4	49.1	48.9	48.3	47.8	47.9	
	8	50	49.1	49.1	48.9	48.5	48.2	48.2	
	8	75	49.9	49.7	49.3	48.8	48.5	48.4	
	8	100	50.0	50.0	49.5	49.1	48.7	48.5	
			Air	0.60 in.	1.5 in.	2.75 in.	4.437 in.	6.0 in.	
3	12	20	53.2	51.8	49.7	48.4	47.8	47.9	
	12	40	54.6	50.7	49.2	48.2	48.0	48.3	
	12	60	55.1	51.2	49.5	48.5	48.2	48.6	
	12	80	55.5	53.9	52.3	50.3	49.1	49.0	
	8	25	56.1	55.5	54.4	52.0	50.1	49.6	
	8	50	56.4	56.0	54.4	52.6	50.7	50.0	
	8	75	56.1	55.8	54.5	52.9	51.1	50.4	
	8	100	55.3	55.5	54.7	53.2	51.5	50.9	
			Air	0.60 in.	1.50 in.	2.75 in.	4.5 in.	5.75 in.	
4	12	20	57.3	53.2	52.6	50.5	49.0	49.1	
	12	40	58.7	54.2	52.2	50.3	48.9	48.8	
	12	60	59.9	55.1	53.2	50.8	49.4	48.6	
	12	80	61.8	56.5	59.3	51.4	50.3	49.5	
	8	25	62.5	57.4	54.9	52.5	51.2	50.7	
	8	50	62.3	57.1	55.2	53.7	51.8	51.3	
	8	75	62.3	56.9	55.0	54.0	52.1	51.7	
	8	100	62.0	57.3	55.3	54.1	52.1	51.8	

TABLE B-5 (continued)

Section	Plate Size Diameter in.	Pressure psi	Temperature Location					
			Air	0.5 in.	1.5 in.	3.0 in.	4.375 in.	5.5 in.
5	12	20						
	12	40	46.8	49.6	50.4	51.0	51.1	51.0
	12	60	46.7	49.8	50.3	50.9	51.0	50.6
	12	80	46.8	49.9	50.7	50.8	51.0	50.7
	8	25						
	8	50	47.4	50.9	52.4	52.1	51.2	51.4
	8	75	47.0	50.1	51.8	51.8	51.2	50.7
	8	100	47.0	49.6	51.1	51.4	51.1	50.9
			Air	0.5 in	1.5 in.	2.875 in.	4.375 in.	5.375 in.
6	12	20	44.8	48.4	49.2	49.2	48.9	48.7
	12	40	44.7	48.4	49.1	49.1	49.0	48.9
	12	60	45.2	47.8	48.7	48.8	48.9	49.2
	12	80	44.2	47.2	48.4	48.8	49.0	49.1
	8	25	42.5	45.6	47.3	47.4	47.8	48.0
	8	50	43.1	46.8	47.5	47.8	48.1	48.1
	8	75	43.1	46.8	47.7	48.0	48.3	48.3
	8	100	44.6	47.4	48.6	48.6	48.5	48.5