EXTENSION OF ROAD TEST PERFORMANCE

CONCEPTS--FLEXIBLE PAVEMENTS

By

ALEKSANDAR S. VESIĆ

And

LEONARD DOMASCHUK

Project No. A-728

Prepared For
National Academy of Sciences,
National Cooperative Highway Research Program

Project 1 - 4

Engineering Experiment Station
Georgia Institute of Technology
Atlanta, Georgia
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CONTENTS

Quarterly Progress Report.

No. 1-2 by Vesić, Aleksandar B.

Final Report.

Vesić, Aleksandar Sedmak and Domaschuk, Leonard.
August 31, 1964.
Mr. Earl Campbell  
Program Engineer  
National Academy of Sciences  
Highway Research Board  
2101 Constitution Avenue  
Washington 25, D. C.

Subject: Quarterly Progress Report No. 1  
Project No. A-728 (1-4)  
"Extension of Road Test Performance Concepts"

Dear Mr. Campbell:

This progress report pertains to the period from October 1 to December 31, 1963. During this time work was concentrated on studies and evaluations of the flexible pavement sections of the AASHO test road.

First, the data on embankment pressures under flexible pavement were analyzed. These data are considered to be most significant for the understanding of behavior of pavements under load. Unfortunately, the embankment pressure measurements constituted only a small portion of the main Road Test objectives. Valuable additional information on these pressures was collected from Special Studies dealing with vehicle characteristics.

The analysis of pressure data was centered around four principal points:

(1) Rate of decrease of vertical stress with depth; effect of variation of pavement thickness on that rate.

(2) Rate of decrease of vertical stress with distance from the center of the loaded area.

(3) Effect of variation of tire pressure and wheel loads on vertical stress distribution.

(4) Seasonal variations of embankment pressures.

The pertinent findings to date are as follows:

(1) It was found that the rate of decrease of peak vertical stresses with depth was in close agreement with that predicted by the Boussinesq theory of stress distribution. The variation of pavement thickness did not seem to influence the mentioned rate.
(2) The rate of decrease of peak vertical stress with distance from the center of the loaded area was also found to fairly follow the pattern predicted by the Boussinesq theory.

(3) The good agreement of computed and observed peak stress distribution was not affected by variations of tire pressure and wheel loads.

(4) There was a wide fluctuation of vertical stresses during the course of a year. The two most significant factors contributing to that fluctuation appear to be the temperature and the moisture content of the subgrade soil.

At present studies are continued by investigating pavement deflections, with particular emphasis on the effects of deflection on pavement performance and failure.

The project personnel consists of the project director and one research assistant, Mr. Leonard Domaschuk, M.S., who is at the same time pursuing work toward a Ph.D. degree at our institute. A second research assistant may be hired during the coming quarter.

Considering the entire research plan for the first phase of this project, scheduled for completion by May 31, 1964, it may be stated that good progress was achieved so far and that the execution of the work follows the existing plan. Total funds expended through December 31, 1963 are, approximately, $2,700.

Respectfully submitted,

Aleksandar B. Vesić
Project Director

ABV/c
Mr. Earl Campbell  
Program Engineer  
National Academy of Sciences  
Highway Research Board  
2101 Constitution Avenue  
Washington 25, D. C.  

Subject: Quarterly Progress Report No. 2  
Project No. A-728 (14)  
"Extension of Road Test Performance Concepts"

Dear Mr. Campbell:

This progress report pertains to the period from January 1 to March 31, 1964. In this period, most of the work was devoted to a rational analysis of the AASHO road test data dealing with the deflections, failures and general performance of the flexible pavements. Test data from the Hybla Valley road test was also incorporated in this study.

The following is a brief outline of some of the areas of investigation:

(a) **Permanent Deflections**

The magnitude of permanent surface deflections was investigated in relation to such factors as:

1. change in thickness of the pavement structure
2. consolidation of the pavement structure and the subgrade
3. stress intensity at subgrade level
4. stress repetitions

(b) **Conditions Under Which Pavements Failed**

Pavement failures were examined on the basis of such factors as rut depth, stress intensity, and stress repetition.

(c) **Deflection Bowl Studies**

The limited data available on the pattern of surface deflections in the vicinity of a loaded area, was analyzed on the basis of theoretical stress distribution, and elastic deflection.

(d) **Analysis of Hybla Valley Data**

The Hybla Valley data consists of numerous peak bearing tests conducted on pavements of varying thicknesses, as well as on a prepared subgrade. An attempt is being made to isolate and analyze the influencing factors involved in the determination of the modulus
of deformation of soil by means of plate bearing tests. This information is pertinent to the analysis of the behavior of flexible pavements subjected to stress repetitions.

A discussion of the findings of all these investigations has not been included since any conclusions would have to be properly qualified which is beyond the scope of this report. Most phases mentioned will require additional analysis.

Considering the entire research plan for the first phase of the project, it may be stated that in spite of good progress achieved so far, there exists some delay with respect to the originally scheduled completion date of May 31, 1964. A request for two months extension of the closing date was submitted.

It is intended to spend the remainder of the contract period on extending the work described in this report and on preparing the final report.

Total funds expended through March 31, 1964, are approximately $5,200.

Respectfully submitted,

Aleksandar S. Vesić
Project Director
FINAL REPORT

PROJECT NO. A-728

EXTENSION OF ROAD TEST PERFORMANCE CONCEPTS - FLEXIBLE PAVEMENTS

Aleksandar Sedmak Vesić
Leonard Domaschuk

August 31, 1964

Prepared for
National Academy of Sciences,
National Cooperative Highway Research Program
Project 1-4

Engineering Experiment Station
GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia
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FINAL REPORT
PROJECT NO. A-728

EXTENSION OF ROAD TEST PERFORMANCE CONCEPTS - FLEXIBLE PAVEMENTS

By
ALEKSANDAR SEDMAK VESIC

and

LEONARD DOMASCHUK

August 31, 1964

Prepared for

NATIONAL ACADEMY OF SCIENCES
NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
(Project 1-4)
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SUMMARY

This is a study of structural behavior of flexible pavements of the AASHO Road Test. Data on deflections and stresses measured during the test as well as data on mechanisms of pavement failure are assembled and analyzed.

It is shown that the load spreading abilities of flexible pavements with conventional, untreated bases are very limited. The stresses and deflections vary with pavement temperature and the degree of saturation of the subgrade, as well as with the vehicle speed.

At creeping vehicle speeds and over the major part of the year, excepting frost periods, the stress and deflection patterns are generally similar to those predicted by the Boussinesq theory for a homogeneous solid.

The analyses of structural failures show that, depending on relative resilience or compressibility of the subgrade soil with respect to the shear strength of the pavement structure, different failure mechanisms may take place.

Very strong and thin pavements over compressible subgrades fail in punching shear. Thick pavements as well as pavements over firm subgrades fail in general shear. In this condition rutting is caused primarily by distortion of the pavement structure.

It appears that there exists, for a given subgrade, a critical subgrade stress beyond which the rutting gets extended into the subgrade. This finding justifies the selection of the limiting subgrade stress as a design criterion.

Recommendations for needed research are given. It is suggested that a general design method for flexible pavements must include considerations of both elastic and plastic phenomena.
INTRODUCTION

This study is undertaken with the purpose of furnishing a rational, mechanistic interpretation of measurements and observation made on flexible pavements in the AASHO Road Test and other similar experimental investigations. The work was initiated by the National Cooperative Highway Research Program in the desire of relating the wealth of information assembled in the Road Test to other ambient conditions.

While the study treats all the major aspects of structural behavior of flexible pavements it is centered around two most frequently used indicators of pavement performance, namely stresses and deflections. At the same time, particular attention is devoted to mechanisms of failure under critical loads. The data analysis is made primarily in the light of existing theories; however, some new concepts and approaches are proposed as well.

The first chapter of the report contains a comprehensive critical review of the existing theories of structural behavior. Data on measured stresses and deflections, as well as on structural failure conditions observed, are assembled in Chapter 2. Chapter 3 presents available data on strength and deformation characteristics of the pavement materials and the subgrade. Finally Chapters 4 and 5 contain analyses of assembled information, appraisals of the meaning and value of the findings of the present study and recommendations for future research.
CHAPTER 1

EXISTING THEORIES OF STRUCTURAL BEHAVIOR OF FLEXIBLE PAVEMENTS

Developed from their crushed-stone historical prototypes, flexible highway pavements were designed as late as 1920 exclusively by rule-of-thumb procedures based on past experience. Standard cross-sections and thicknesses of pavements for all possible soil and traffic conditions were generally practiced. While highway engineers recognized the importance of subgrade properties for pavement behavior, the pavement itself was still not considered as a structural system that serves to transmit the vehicle loads to the subgrade soil. No analysis or observations of pavement stresses and displacements were even attempted.

Subsequent years brought radical changes in the overall approach to pavement design. The development of soil mechanics and extensive studies of soil properties made it possible to formulate first empirical relations between the pavement performance on one side and the vehicle load intensity and the soil type on the other side (1) (2). Intensified airport pavement studies initiated during World War II lead ultimately to a semi-empirical extension of the existing relationships into more general criteria, including variables such as tire pressure and number of load applications (13). In the same period several theoretical and semi-theoretical methods for pavement design were proposed, (4) through (11), which tried to incorporate into design also other variables such as deformation moduli or strength characteristics of the pavement and subgrade materials.
Existing Theoretical Methods for Pavement Design

All the theoretical methods proposed can be classified into two major groups:

(1) Theoretical methods based on considerations of ultimate strength of the pavement components.

(2) Theoretical methods based on considerations of pavement and subgrade stresses and deflections in the range of working loads.

The methods of the first group or ultimate strength methods are concerned with pavement behavior at failure. Their basic design criterion is that a pavement must possess a defined safety factor against shear failure or the pavement materials. The two best known representatives of this group are the Glossop-Golder method (3) and the McLeod ultimate strength method (10). Both methods assume that a pavement system fails in general shear, similarly to bearing capacity failure of shallow footings on dense soils (Figure 1).

When using this ultimate strength approach the pavement materials and subgrade are assumed to behave as rigid-plastic solids defined by their shear strength characteristics: cohesion or strength intercept, \( c \), and angle of shearing resistance, \( \varphi \). No formal considerations of strains and deflections are introduced.

The methods of the second group or elasticity methods consider the pavement behavior under working conditions, when deflections, by assumption, are proportional to applied loads. Their basic design criteria require evaluations of stresses and strains in the pavement materials. For such evaluations, in all instances, the theory of elasticity is used.
Figure 1. General Shear Failure of a Flexible Pavement.
Among the known methods of this group, the following ones have been more widely used or show a substantial promise for development:

1) Kansas Highway Department or Palmer-Barber method (4) (5);
2) U. S. Navy or Burmister method (6) (7);
3) Odemark method (8);
4) Peattie method (11).

All these methods consider the pavement system to be a layered solid, in which individual layers are homogeneous, isotropic and linearly deformable or elastic. The behavior of these layers under load is defined by their deformation moduli $E$ and Poisson's ratios $v$. The methods differ, however, in their formal treatment of the upper layers, and, particularly, in their design criteria.

In the Kansas method the stresses and displacements are evaluated by using the Boussinesq solution for a homogeneous solid with a "stiffness factor" derived from considerations of the slab action of the upper layers. In this way the assumed better load spreading ability of the apparently stiffer layers is taken into account. The deformation moduli of pavement layers and of the subgrade are determined by triaxial tests. The design criterion used is to limit the theoretical deflection of the surface under load to 0.1 in.

The U. S. Navy method uses essentially the same design criterion of limiting deflection which is set equal to 0.2 in. However, the stresses and displacements are evaluated by using the Burmister solution for a two-layer solid. The deformation moduli of pavement layers and of the subgrade are determined by plate load tests, which are interpreted by means of the same Burmister solution.
In the Odemark method the stresses and displacements are evaluated by considering the pavement layers to behave as a slab resting on subgrade soil. The deformation moduli are determined by plate load tests. The design criterion used is to limit the maximum curvature of the deflected pavement surface.

Finally the new Peattie method, which is still in development, uses two design criteria. The vertical stresses on the subgrade as well as the radial tensile strain in the surfacing layer should be kept within certain allowable limits. The deformation moduli of pavement layers are determined in the field by vibrational techniques.

It should be mentioned that the methods of the second group do not include investigations of safety factors against structural failure. Also, by the nature of approach used, they do not allow a direct evaluation of the effects of load frequency and duration. Such effects are generally included in design indirectly, usually by some empirical estimates of behavior of pavements under repeated loading.

In the last few years serious efforts have been made toward development of visco-elastic theories of pavement behavior (12) (13), which, potentially, would allow some rational considerations of the variable time in stress and displacement analyses. However, these theories are still in the basic research stage and have not yielded a consistent design method.
Critical Appraisal of the Ultimate Strength Methods

The approach used in the ultimate strength methods of pavement design undoubtedly possesses several advantages common to all plastic design methods:

1) It is simpler in principle and in formal presentation. It involves fewer assumptions about the behavior of pavement components and deals with well defined and familiar physical characteristics of the materials involved.

2) It allows a pavement design with predetermined safety factor, the magnitude of which can, in principle, be selected by following a consequent design philosophy.

The only disadvantage of general character, that this approach shares with other plastic design methods, is that it formally does not furnish information about pavement displacements.

In spite of the potential merits of this design approach, the two known methods which are based thereupon have not been widely used. This is to a great extent due to the fact that these methods were never thoroughly developed. No basic research was done to justify their fundamental assumptions. Thus, they contain, among others, an arbitrary assumption of general shear failure of the pavement along curved rupture surfaces extending from the tire edge back to the pavement surface (Figure 1). Experience shows, however, that pavements more often fail by punching failures, similar to those observed under dynamically loaded footings as well as under ordinary footings on soft, loose and layered soils (14) (15) (Figure 2). It should be added that the amount of investigations done to correlate design findings
Figure 2. Punching Shear Failure of a Flexible Pavement.
of these methods with behavior of actual pavements has been very limited and inadequate.

In conclusion, the ultimate strength methods of pavement design are not usable in their present form. Nevertheless their general approach has great potential merits. Methods of this kind should be developed along with elastic or viscoelastic methods, over which they may possess certain advantages.

Critical Appraisal of the Elasticity Methods

As mentioned earlier, the methods of the second group or elasticity methods are based on considerations of stress and deflections of pavements predetermined analytically with the help of the theory of elasticity. In contrast to the ultimate strength methods, some of the methods of this group, notably the U. S. Navy method and the Kansas Highway Department method, have been widely used. This, however, does not mean that they are free of arbitrary assumptions. On the contrary, it might be said that, paradoxically, their more general use had contributed toward neglecting the task of verification of some of the very fundamental assumptions on which they are based.

To illustrate this argument, it should be recalled that the assumption of constant deformation moduli $E$ of individual pavement layers implies an unrestricted transmission of both compressive and tensile stresses, therefore an unrestricted slab action of the upper rigid layers. This action would cause reduced vertical stresses on the subgrade and relatively large deflection basins.
It has been shown recently (16) that practically all the vertical stress measurement data on pavements with conventional, untreated bases show significantly higher vertical subgrade stress than indicated by the layered solid theories. Newer experiments in the USSR (17) and at the Georgia Institute of Technology (18) (19) further confirm this fact*.

This is best evident from Figures 3 and 4, partly reproduced from paper (16). These figures show the measured vertical stresses \( \sigma_z \) directly under the loads applied at the surface of flexible pavements, as measured in five different full-scale and model investigations, namely:

1) In-situ tests performed by the Corps of Engineers, U. S. Army, with actual airplane loads on an airfield pavement section at Marietta, Georgia (22).

2) In-situ tests performed by the Road Research Laboratory of Great Britain (23).

3) Model tests performed at Purdue University with rigid plates on 8 x 8 ft. pavement sections (24).

*It has been suggested by Burmister (20) and Schiffman (21) that the stress measurements presented are in error because of the so-called pressure cell inclusion effect. If this effect were of any significance, the measured vertical stresses in homogeneous masses of soil would also be higher than analogous stresses under the pavements. Experiments at the Georgia Institute of Technology show definitely that they are not.
Figure 3. Vertical Stresses Under Flexible Pavements.
Figure 4. Vertical Stresses Under Flexible Pavements.
4) Model tests performed at the Georgia Institute of Technology
(25) (26) (18) (19) with truck tire loads on 12 x 8 ft. pavement sections.

5) Model tests performed at the Transportation Institute of the
USSR Academy of Sciences with rigid plates on 8 x 5 ft. pavement sections
(17).

Plotted on both figures by a full line is the theoretical stress
distribution for a homogeneous isotropic solid (Boussinesq), loaded at
the surface by a load uniformly distributed over a circular area of
radius a. Also plotted on Figure 4 are the theoretical stress distribu-
tions for a two-layer homogeneous isotropic solid, which are used in
analysis in the existing elasticity methods of pavement design.

Together with observations of sizes of deflection basins, which all
appear to be more confined to the vicinity of the loaded area than in-
dicated by the layered solid theories, the stress data presented leave
no doubt about the fact that the slab action of the upper layers of
conventional flexible pavements is very limited.

The above remarks do not discredit all evaluations of stresses and
deflections based on the layered solid theories. They merely point out
to one of the uncertainties of the known methods of the second group in
their present form. More basic research is needed to shed light into
the actual behavior of flexible pavements under working conditions.
Some ideas about necessary theoretical investigations in this direction
have been expressed in the conclusions of the reference (16).

Concerning the design criteria forming the basis of methods of
this group it should be stated that the criterion of a unique limiting
deflection, such as that used in the Kansas and U. S. Navy methods cannot withstand serious criticism. Obviously, quantities such as ratio of deflection to the size of the loaded area, or such as the curvature of the deflected surface, express better the ability of a loaded pavement to support additional load as well as its ability to offer a smooth riding surface. It is not difficult to show that the selection of either of the two last mentioned criteria or of the criterion of limiting stress on the subgrade leads to design curves that follow more closely the pattern of the CBR design curves, which have generally shown agreement with observations in a greater variety of design conditions.

In conclusion, the principal weakness of the existing elasticity methods of design of flexible pavements lies in uncertainty of some basic assumptions that lead to analysis of stresses and deflections, as well as in the well-known ambiguities in determining the deformation moduli of pavement layers. It is hoped that the present study will contribute toward better understanding of actual behavior of flexible pavements under load.
CHAPTER 2

DATA ON STRUCTURAL BEHAVIOR OF THE AASHO TEST ROAD FLEXIBLE PAVEMENTS

The performance of a flexible pavement system* can be evaluated in different ways depending on the object of the evaluation. There may be many concepts as to what constitutes adequate or inadequate pavement performance. For example, a roadway that is structurally adequate insofar as load carrying capacity is concerned may be considered to be inadequate in its riding qualities. In contrast to this, the structural behavior of a flexible pavement system is evaluated by its response to various load applications in all possible environmental conditions.

In the present investigation an attempt will be made to examine the structural behavior of AASHO Road Test flexible pavements strictly in a quantitative manner, with attention on pavement stresses and deflections. This method of approach is being pursued in an attempt to determine whether some of the existing, workable, fundamental laws governing the behavior of materials can be applied to simulate the response of flexible pavements to applied loads. Particular emphasis will be placed on assessing the applicability of the theory of elasticity to some phases of pavement performance.

It is felt that the structural behavior can best be examined by separate consideration of the following phases:

*Hereinafter pavement system refers to the surface, base, subbase and subgrade components of the roadway cross-section. Pavement structure refers to those components lying above the natural basement soil.
1. The nature of the load distribution throughout the depth of pavement system.

2. The resilient deformation of the roadway surface due to the imposed wheel load.

3. The cumulative plastic deformation and structural failure of the pavement system.

Information for the analysis will be taken primarily from the results of the AASHO Road Test and will be supplemented by results from other road tests and investigations where possible.

In this chapter, the test road data pertinent to the analysis will be assembled in accordance with the above categories. The actual analysis of the data will be presented in Chapter 4.

**Stress Distribution Data**

As part of the main factorial experiment of the AASHO Road Test, a limited study of embankment pressure or vertical stress on the subgrade was conducted. Pressure cells were installed at the embankment level in various sections of Loop 4. The study included the effects of variation in vehicle speed, wheel load, pavement structure thickness, and environment, on the embankment pressure.

In addition, vertical stress data was obtained in conjunction with the special studies conducted on sections of Loop 4 that had survived the duration of the main factorial experiment.

In subsequent paragraphs the vertical stress data has been categorized in accordance with the different variables which constitute
separate analysis. It should be pointed out that all vertical stresses presented correspond to creep speed (2 mph.) of the vehicle unless otherwise designated.

1. Effect of pavement structure thickness.--Only very limited data showing the influence of pavement structure thickness on vertical stress distribution is available. The data which is given in Table 1 represents the mean of 7 weekly observations taken during the summer of 1959, at the designated sections* of Loop 4, as reported in reference (27), Report 5.

2. Effect of wheel load and tire pressure.--The data showing the influence of wheel load and tire pressure on distribution of vertical stresses was obtained from the special studies, (27) Report 6, and is presented in Table 2. A pressure cell was installed at the embankment level in the design section 5-6-12. The vertical stress was measured for various vehicle types having wheel loads ranging from 2 to approximately 34 kips and tire pressures ranging from 8 to 100 pounds per square inch. It will be noted that in many instances there is a duplication of wheel loads and tire pressures. These correspond to separate studies and are therefore presented independently. The data is also separated in accordance with wheel and axle configurations. This was done for the convenience of analysis.

*The sections are designated by three numbers which refer to the respective thicknesses of surfacing, base course and subbase.
3. Effect of distance from point of load application.--In conjunction with the special studies, vertical stress contours were obtained for various wheel loads, through the use of a variety of vehicles. The contours were developed from vertical stress readings taken with the wheel load placed at varying radial distances from the pressure cells. Typical stress contours are presented in reference (27), Report 5. For the purpose of comparing the theoretical and the observed pattern of vertical stress variation with distance from point of load application, data obtained for small and medium scraper was selected. This particular data was selected because it included a wide range in wheel loads and tire pressures. The data corresponding to stresses along longitudinal and transverse reference axes is presented in Tables 3 and 4 respectively.

4. Seasonal variations in vertical stresses.--Routine vertical stress readings were taken periodically at design sections 5-6-12 and 3-6-12 throughout the duration of the road test. From these data, an attempt will be made to determine the most significant environmental factors that cause appreciable variations in vertical stresses during the course of a year. The data is presented graphically in Figure 16, Chapter 4.

Deflection Studies

A considerable portion of the AASHO Test Road studies was dedicated to deflection studies and the development of empirical relationships between deflection and such factors as design thickness, vehicle speed, wheel load, and pavement temperature. Ultimately this led to the develop-
ment of a relationship between deflection and pavement performance. In
the present investigation, it is proposed to investigate the applicability
of the different methods of deflection analysis to describe the behav­
iment of flexible pavements.

The methods of analysis to be examined are all based on the theory
of elasticity. Thus, insofar as pertinent test road data is concerned,
the deflection basin study will constitute the major portion of the
data to be analyzed. In the AASHO Road Test studies, the configuration
of the deflection basin was determined by influence line techniques.
Deflection readings were obtained from vehicle placements to the left,
right, and directly over the deflection measuring point. Contours of
equal deflection were then constructed. Typical deflection contours
thus obtained are given in reference (27), Report 5.

It should be pointed out that the deflections were measured in one
of two ways. Firstly by means of electronic recording devices (LVDT)
which utilized settlement rods and hence the recorded deflections are
relative to some finite depth. Secondly, Benkleman beams were used
which provided deflections for essentially a semi-infinite soil mass.

In the present investigation, data obtained by means of the LVDT
devices were analyzed. The surface deflections were measured relative
to a point 6 feet below the surface of the embankment. The deflections
along the longitudinal and the transverse axis were of primary concern
to this investigation and hence only these data have been selected and
are presented in Table 5.
Structural Failure Studies

In the AASHO Road Test analysis, the pavement performance was evaluated in terms of roughness, extent of cracking, required patching, and rut depth. These factors were then incorporated, by a method of multiple regression analysis, into an index of performance known as the Present Serviceability Index. We have attempted to isolate and analyze the components of failure in an effort to establish any facts pertinent to the characterization of the behavior of flexible pavements.

Toward this end, the following aspects were investigated:

1. Contribution of the components of the pavement system to surface deflection. -- In the AASHO Road Test, the change in the transverse profile of the pavement surface was determined by periodic precise level and profilometer measurements. As well, layer thickness changes were measured by means of settlement rods located at different levels within the pavement structure. With these two sets of data it is possible to determine the contribution of the various components of the pavement system to the permanent deflection of the pavement surface. The data is reproduced in Figures 5 and 6.

2. Mechanics of change in thickness of pavement structure components. -- Under the action of a wheel load, the underlying material is subjected to radial, tangential, vertical, and shearing stresses. The response of the soil to this stress system can be both elastic and non-elastic and appears in the form of shear deformations and/or volume changes. The portions of the change in layer thickness caused by these two forms of deformation were determined from the trench studies. In these studies,
Figure 5. Permanent Deflections of AASHO Road Test Flexible Pavements.
Figure 6. Permanent Deflections of AASHO Road Test Flexible Pavements.
the changes in density and layer thickness of each pavement component were determined. A reproduction of the data dealing with the changes in thickness is given in Tables 6, 7, and 8 which were taken from reference (27), Report 5.

3. Factors influencing rut depth.--A convenient measure of non-elastic deformation is the depth of the ruts that develop in the wheel paths. Many factors contribute to the formation of these signs of pavement distress. An attempt will be made to correlate these factors with the depth of ruts developed in the AASHO Test Road. Considerable work has been done in this area and the results are presented in reference (27), Report 5. Some of the findings will be repeated here in order to present as complete a picture as possible. The influence of the following factors on rut depth will be examined.

a. Load repetition.--Typical plots of rut depth versus axle load repetitions are given in Figure 7 which was taken from reference (27), Report 5.

b. Axle load.--The number of axle load repetitions of different axle loads required to produce rut depths of 0.25, 0.50, and 0.75 inches was obtained from AASHO Test Road Data System 4199. The values are given in Table 9.

c. Vertical stress on subgrade.--A study of the effect of base thickness on the rut depth was carried out, the results of which are presented in the reference (27), Report 5. This data was reanalyzed so as to obtain a relationship between the vertical stress on the sub-grade and rut depth. The data is given in Table 10.
Figure 7. Effect of Repetitive Loading on Depth of Rut.
CHAPTER 3

DATA ON THE PHYSICAL PROPERTIES OF THE AASHO ROAD TEST MATERIALS

Of essence to the analysis of the structural behavior of a pavement system, is a thorough knowledge of the characteristics and properties of the materials making up the components of the system. Of particular significance, are those properties which characterize the response of the materials to static, dynamic, and repeated loads.

Since no evaluation of the properties of the materials was carried out as part of this particular investigation, the properties presented are based on existing published data on the subject.

The properties as categorized and presented herein represent average values obtained from the indicated references.

Index Properties

The index properties of the embankment soil, the subbase, the base, and the surfacing courses are listed in Tables 11 through 14. The values represent the average of numerous tests conducted by the AASHO Road staff prior to and during construction of the test road. The values were taken from reference (27), Report 5.

Strength Characteristics

The strength characteristics of significance for this investigation are the two parameters: cohesion, c, and angle of shearing resistance, \( \phi \). The results of the cooperative study, the Bureau of Public Roads study as well as other studies indicated a wide variation in strength
parameters among the various agencies. The parameters obtained in the aforementioned studies (obtained by triaxial compression tests on partially saturated samples) are given in Table 15. Because of the large variation in results reported they can be accepted only as representative average values.

**Deformation Characteristics**

The deformation characteristics pertinent to this investigation are the modulus of deformation $E$ and the Poisson's ratio $v$ of the soil. A review of published data on the modulus of deformation of the AASHO Road Test materials reveals firstly that very few such results are available and secondly that there is a wide range of values within any one investigation as well as between investigations. The latter of course is to be expected since the modulus of deformation varies with conditions of loading, soil properties, etc., and moreover, is not unique in its definition.

Of particular interest to this investigation is the modulus of deformation which characterizes the response of the soil to repeated stress applications. It has been demonstrated that under repeated loading soils will generally assume elastic behavior after a certain number of repetitions of a given stress intensity. Thus, it may be said that most flexible pavements will in time exhibit almost complete elastic response to load applications. The slope of the stress-strain curve defining this response is called, after Hveem (28), resilience modulus, $E_r$. Analysis of elastic deflections can, in principle, be based on
such modulus. Another modulus that is sometimes used in total deflection analysis is the initial tangent deformation modulus, $E_0$, which is defined as the initial slope of the stress-strain curve at the first loading.

The following text briefly reviews the studies of deformation characteristics performed for the AASHO Road Test. All the significant results of these studies are summarized in Table 16.

1. Cooperative materials testing program.--As outlined in reference (29) numerous agencies undertook evaluations of the properties of the materials used in the test road. Among those, the Kansas State Highway Department reported data on the deformation moduli obtained by standard triaxial compression tests.

2. University of California studies.--An investigation into the resilience characteristics of the AASHO Road Test subgrade soil was conducted at the University of California at Berkeley by Seed, Chan, and Lee (16). This study included effects of method of compaction, density, moisture content, stress level, and number of stress repetitions on the resilience modulus of deformation. Each of the aforementioned factors had a significant effect on the modulus of deformation, however the degree of saturation and the stress level caused perhaps the widest variation in the modulus values. This work not only emphasizes the difficulty of selecting a working modulus of deformation, but indicates that variations of several hundred percent from the selected value can be expected under varying loading and climatic conditions. The value presented for reference in Table 16 corresponds to 100,000 repetitions of a stress difference of 10 pounds per square inch.
3. Asphalt Institute studies.--In a study of the application of theoretical concepts to asphalt concrete pavement design (31) the modulus of deformation of the asphaltic concrete was computed by using the Van der Poel stiffness concept (32). The modulus of the surfacing layer so obtained varies with the rate of loading and the temperature. For typical environmental and loading conditions, a working modulus of 150,000 lb/in² was obtained.

It is of interest to note that moduli of 15,000 and 3,000 lb/in² were used in this study for the base and the subgrade respectively. These values were not obtained experimentally, and are shown in Table 16 only for reference.

4. Ohio State University studies.--In a study of deflections of the AASHO Test Road at the Ohio State University (33) use was made of the complex modulus $E^*$ as defined by Papazian (34). The complex modulus is based on the viscoelastic response of soils undergoing triaxial compression. The complex modulus varies with stress amplitude, frequency, temperature, moisture content and density. Typical values for the components of the pavement structure, as determined in these investigations are given in Table 16.

5. Georgia Institute of Technology studies.--Standard triaxial compression tests were conducted on samples recovered from the AASHO Road Test subgrade in conjunction with a satellite research project (35). An initial tangent modulus of 1,040 lb/in² was found.

6. Indirect methods.--Moduli of deformation have also been computed by the authors in the following ways:
a. From a relationship between the coefficient of subgrade reaction and modulus of deformation. Plate bearing tests were conducted on the different components of the structure from which the respective (elastic) coefficients of subgrade reaction were determined. These coefficients were converted to moduli of deformation on the assumption of a semi-infinite homogeneous mass. Only the value for the subgrade is given since the values obtained for the other components would in reality reflect composite moduli due to the nature of the plate load test.

b. From an empirical relationship between the CBR of a soil and the modulus of deformation (36).

\[ E (\text{kg/cm}^2) = 100 \times \text{CBR} \]

Field CBR values were used in the case of the embankment soil and the laboratory CBR was used for the subbase course.

c. From deflection analysis. By means of the LVDT devices, the deflection of the individual structural pavement components were obtained. This made it possible to compute the modulus of deformation of each component using the theory of elasticity. In these calculations, stress distribution according to the Boussinesq theory was assumed and a composite modulus was determined for the pavement structure.

As it can be seen from Table 16, there are significant differences in individual deformation moduli obtained by various procedures. This is quite normal in the present situation with the measurements of deformation characteristics of pavement components. However, if any real progress is to be made toward developing more rational methods of pave-
ment design, it is absolutely necessary to develop, first of all, more understanding of the fundamental laws that govern the behavior of all the types of pavement materials involved. This accomplished, it will be possible to reduce testing of pavement materials and soils to standardized procedures which will all give, irrespective of the testing equipment used, the same, well defined deformation characteristics.

In view of the existing situation and the lack of funds for any materials and soils testing as a part of this project, no attempt was made in this report to make evaluations of results with the absolute values of deformation moduli as presented in Table 16. Limited use was made of a ratio of a composite modulus $E_1$ of the pavement structure, and the modulus $E_2$ of the subgrade. On the basis of values in Table 16, a ratio of $E_1/E_2$ of 10 was assumed.
This chapter is devoted to the analysis of the results referred to in Chapter 2. The chapter is subdivided in a manner similar to that used in Chapter 2, making cross reference simple and convenient.

**Stress Distribution**

One of the primary functions of the pavement structure is to distribute the concentrated stress imposed on its surface, over a sufficiently large area of the subgrade, so as to prevent the inherently weaker subgrade from undergoing excessive deformation. The pattern of the stress distribution within the pavement structure, is of utmost importance to the rational analysis of flexible pavements.

To date, numerous analysis and investigations have been directed towards the establishment of the pattern of stress distribution in a soil due to a load at surface. It may be stated that from these investigations, two basic methods of stress analysis for flexible pavements have been adopted. The first method is based on well known Boussinesq solution for stress distribution in a homogeneous isotropic solid; the second on equally well known Burmister solution for layered solids. Since these methods in many instances yield vastly different results for identical conditions there exist a real need for assessing conditions under which the validity of one or the other is questionable.
The following analysis consists primarily of a comparison of vertical stresses measured in the AASHO Road Test with values computed on the basis of both the Boussinesq and Burmister theories.

In addition to well known basic assumptions on which the classical elasticity theory is based, the following simplifying assumptions are made in all the stress analyses:

1. The tire-pavement surface contact pressure is assumed to be uniform and equal to the tire pressure wherever contact pressures have not been experimentally determined. Thus, the contact area is computed by means of the tire pressure and the wheel load, and is assumed to be circular.

2. Stresses under dual tires and tandem axles are computed on the assumption of the validity of principle of superposition in stress analysis.

3. It is assumed that no surface shear stresses are induced at the contact between the tire and the pavement.

Figure 8 shows a few measured values of vertical stress on the subgrade under an 18 kip single axle load on pavements of different thickness. The theoretical stress distributions according to Boussinesq and Burmister theories are also shown. It can be stated that, here again, there exists a good agreement with the Boussinesq theory.

Figure 9 shows vertical stresses under variable single wheel loads and tire pressures, as measured in design section 5-6-12. A dimensionless plot was used to permit a generalized presentation of the results. It is obvious from this figure that the observed stresses are in close
Figure 8. Effect of Thickness of Pavement Structure on Vertical Subgrade Stresses.
Figure 9. Vertical Stresses Under Varying Conditions of Wheel Load and Tire Pressures (Single Wheel and Axle).
agreement with the theoretical stresses after Boussinesq, though the latter are generally somewhat higher. Plotted in the same figure are theoretical stresses for a two-layer solid after Burmister, for moduli ratios 10 and 100.

Although not indicated on the graph, it was found that the Burmister solution corresponding to a modular ratio of 5 formed the outer limit for the observed values in question. It was found that the layered solid solutions based on moduli ratios in excess of 2 would not yield a better fit curve than does the Boussinesq curve.

The data corresponding to vehicles with multiple axles and/or wheels could not be presented in a similar dimensionless plot due to a lack of common parameters. The data presented in Figure 10 is a direct comparison of observed values of embankment pressure with those based on the Boussinesq solution. The plot indicates a greater scattering of results and somewhat poorer agreement with the theoretical Boussinesq values than in the previous case. The observed values were generally lower than the Boussinesq values, but not to the extent that the use of the Boussinesq theory in these cases would be insensible. A good agreement with the layered solid theory can be obtained only if a moduli ratio $E_1/E_2$ of 2 is selected.

To show the distribution of vertical stresses at points other than directly beneath the load the stress contour data as given in Tables 3 and 4 is presented graphically in Figures 11 through 15. As in previous plots, theoretical curves based on Boussinesq stress distribution (solid line) and the layered solid theory (broken line) for a modular ratio of
Figure 10. Vertical Stresses Under Varying Conditions of Wheel Load and Tire Pressures (Multiple Wheel and Axle).
Figure 11. Observed Distribution of Vertical Stresses; Wheel Loads 7,700 to 16,450 lb; Tire Pressure 30 lb/in².
Figure 12. Observed Distribution of Vertical Stresses; Wheel Loads 7,700 to 16,450 lb; Tire Pressure 45 lb/in$^2$. 
Figure 13. Observed Distribution of Vertical Stresses; Wheel Loads 17,500 to 26,250 lb; Tire Pressure 30 lb/in².
Figure 14. Observed Distribution of Vertical Stresses; Wheel Loads 17,500 to 26,250 lb; Tire Pressure 45 lb/in$^2$. 
Figure 15. Observed Distribution of Vertical Stresses; Wheel Loads 20,450 to 33,750 lb; Tire Pressure 45 lb/in².
are included on each plot. From an examination of the plots, it can be seen that in general, the observed values of vertical stresses are in better agreement with the theoretical values based on the Boussinesq theory. The agreement is good insofar as actual magnitudes are concerned and also in the shape of the pressure bowl. The pressure bowl as based on the layered solid theory implies less stress concentration in the immediate vicinity of the loaded area, due to the assumed slab action of the upper layer. In some instances good agreement with the layered solid theory can be found by selecting a very low moduli ratio $E_1/E_2$.

To illustrate seasonal variations in vertical stresses Figure 16 is presented. This figure shows the variation in vertical stresses over the duration of the AASHO Road Test, as based on periodic observations. Included in the figure are variations of the environmental factors, temperature, moisture content of the soil and depth of frost penetration.

Considering first of all, the vertical stress and disregarding minor fluctuations, it can be seen that there are periods during which there are extreme deviations from the average. Generalizing on a seasonal basis it may be said that the low stress period corresponds to winter months, while the high stress periods correspond to the spring months, and continue through the rest of the year. It will be shown in subsequent discussion that this general trend of stress variation can be explained in a satisfactory manner.

A comparison of the temperature plot and the vertical stress plot indicates that in general, depressing temperatures are associated with lowered vertical stresses. In particular, the extremely low vertical
Figure 16. Variation of Vertical Stresses and Environmental Conditions During the Road Test.
stresses correspond, without any doubt, to periods during which the soil was in a frozen state. This is dramatically illustrated by the extreme variation in vertical stresses observed during the month of February 1960 during which time the soil passed from a frozen to an unfrozen and back to a frozen state.

One basis for disputing the validity of the layered solid theory is the inability of the macadam bases to withstand the tensile stresses accorded to them by the theory. This applies particularly to temperatures in excess of $70^\circ F$ when the asphalt loses its rigidity. It has been determined experimentally, that asphaltic concrete may exhibit a tensile strength in the order of several hundred pounds per square inch at sub-freezing temperatures (37) (38). It may also be expected that even granular water-bound bases will exhibit an appreciable tensile strength if in a frozen state. Under such conditions, the validity of the layered solid theory is undisputable. The asphaltic material need not be in a frozen state to acquire tensile strength, and hence may possess sufficient tensile strength at intermediate temperatures to behave essentially as a slab thus reducing the vertical stresses.

The AASHO Test Road studies showed that at temperatures in excess of $80^\circ F$, the behavior of the pavement is, for all practical purposes, independent of the temperature (27, Report 5). Correspondingly, as the temperature is decreased, the rigidity of the pavement structures is increased, thus permitting greater slab action to take place and resulting in a decrease of vertical stress.
In addition to the effects of temperature and frost there appears to be also a slight effect of moisture conditions in the subgrade and pavement structure. This is particularly evident in the period April through July 1960. In this period the vertical stress versus time curve, to a certain degree, shows the same trend as the moisture content versus time curve. This general trend would be expected by the layered solid theory, although the absolute magnitudes of stresses are still very high, actually higher than the Boussinesq stresses. On the other hand according to the Boussinesq theory, the vertical stresses should be independent of the deformation characteristics of the solid.

The mentioned findings point out to the fact that, perhaps, the pavement system acts as a very complex layered solid in which, due to lack of tensile strength and also due to anisotropy of some layers, vertical stresses are so high that they can be approximated by Boussinesq stresses for a homogeneous solid. It is also possible that the effect of moisture content on stress distribution has some connection with variations in shear resistance, therefore with plastic phenomena in pavement materials and soils.

It may be concluded that environmental conditions have a significant effect on vertical stress distribution within flexible pavements. There appears to exist a pronounced slab action of the asphaltic concrete surfacing and frozen macadam bases at subfreezing temperatures. In average temperature conditions the vertical stresses are approximately equal to Boussinesq stresses; slightly varying with variations of moisture content.
An understanding of the environmental changes that can be anticipated, and how these changes may affect the behavior of the pavement, will permit the designer to base his analysis on the prevalent or worst possible conditions in accordance with the economic and other aspects of the project.

**Deflection studies**

During the recent years, deflections of flexible pavements have been the object of increased interest to highway engineers. They are now quite widely used as a measure of performance of existing pavements. In some areas where the spring "break-up" has a detrimental effect on the load carrying capacity of the pavements, they are regularly controlled. Also, as mentioned earlier in Chapter 1, they are used by some agencies in their pavement thickness design criteria.

Numerous investigations have been directed towards establishing a rigorous solution to the problem of predicting the deflection of a flexible pavement system. As explained earlier in Chapter 1, most of the solutions have the elastic theory as their basis, although several of the more recent investigations treat the flexible pavement materials as viscoelastic. To date no reliable general method of analysis has evolved that would permit a sufficiently accurate prediction of deflections, although isolated instances of observed deflections have been successfully analyzed on a theoretical basis.

This investigation is confined to examining the applicability of the two known methods of analysis, namely the layered solid theory and the Boussinesq theory, to deflection problems. This will be done
by comparing observed and theoretical values, as was done in the case of vertical stresses. However, as shown in an earlier investigation (16), in attempting to assess the validity of the theories it is very important to examine the overall configuration of the deflection basin rather than just the deflection immediately under the loaded area.

In Figures 17 through 20 the measured and theoretical deflection basin profiles based on AASHO Road Test data from Table 5 are compared. To better illustrate the shape of the deflection bowl, relative deflections (expressed as a percentage of the maximum) are used rather than absolute values. In this way, discrepancies directly attributable to the use of inaccurate or non-representative values of the elastic constants are avoided.

The theoretical deflection curve referred to as the Boussinesq curve is based on stress distribution according to the Boussinesq theory, and settlement according to the elastic properties (E and v) of the component layers. For simplicity the pavement structure is treated as a single layer. To assist in the analysis, deflection factors were computed for points at different depths and horizontal distances from the center of the loaded areas. The deflection factors derived for this analysis are given in Tables 17 through 20.

The layered solid theoretical curve is based on the assumption of stress distribution and deflection after Hogg (39). As in the case of the vertical stress analysis, a moduli ratio of 10 for a two-layer solid is used.
Figure 17. Measured Deflection Basins, Sections 5-6-12 and 4-6-4, Single Wheel Loads.
Figure 18. Measured Deflection Basins, Sections 4-6-12 and 5-6-4, Single Wheel Loads.
Figure 19. Measured Deflection Basins, Sections 5-6-12 and 4-6-4, Dual Wheel Loads.
Figure 20. Measured Deflection Basins, Sections 4-6-12 and 5-6-4, Dual Wheel Loads.
As can be seen from the figures, the observed deflection profile has a characteristically shortened and an extended side indicating the transient character of the imposed loads. The overall configuration of the basin is generally in better agreement with that predicted by the Boussinesq theory than the layered solid theory.

This finding is very significant in view of the fact that recent research (40) stresses more and more the importance of strain and curvature in the development of fatigue failures of asphaltic concrete surfaces. The observed curvatures of deflection surfaces are definitely much greater than indicated by layered solid theories using a conventionally determined moduli ratio.

It should be pointed out at this time that although the prediction of the deflection basin profile by the Boussinesq theory appears to be more plausible, the problem of predicting the actual magnitude of deflection still remains. One reason for this is the difficulty of obtaining elastic constants of the various component materials that are truly representative of the behavior of the soil under varying loading and climatic conditions. Since according to the elastic theory, deflection varies directly with the modulus of deformation and the modulus of deformation has been demonstrated to vary by several hundred percent under varying loading and climatic conditions, the difficulty of obtaining comparable measured and theoretical values of deflection is apparent.

An analysis of limited extent was carried out to determine the variation in the modulus of deformation of the subgrade soil that would yield an exact solution. The deflection basin profile obtained for design
section 5-6-4 was used in the analysis. The deflection within the upper 6 feet of subgrade was determined from the LVDT readings. The analysis indicated that in order to obtain an exact solution, the modulus of deformation of the subgrade would have to vary approximately as the one-third power of the vertical stress. This finding is consistent with the experimental evidence that the modulus of deformation of many soil deposits increases with the confining pressure.

To obtain additional information on the load-deformation characteristics of a flexible pavement system, a limited analysis of the Hybla Valley Test Road data (41) was conducted. Of particular interest to this report were the results of the "repetitional" tests conducted on the surface course of pavement sections of varying thicknesses. The test consisted of a single application and release of 16, 32, 48 and 64 lb/in² loads followed by 75 repetitions of a constant unit load of 80 lb/in². The tests were performed using rigid plates and the diameter was varied from 6 to 42 inches. The data is presented in Figure 21 which is a plot of reduced modulus versus the pavement thickness expressed as a multiple of the radius of the plate. The reduced modulus incorporates the measured elastic deflection of the plate, the plate diameter, and the unit load acting on the plate. This form of data representation was used to permit a generalization of the problem and to facilitate the analysis of the data. Each point on the plot represents the results of a single test.

Assuming a two-layer solid, curves were fitted to the points on the basis of the Boussinesq and the Burmister theories of stress distribution. These curves and their solutions are shown on Figure 21. This analysis
Figure 21. Interpretation of Hybla Valley Test Data.
demonstrated that either theory will yield a solution. The acceptability of the solution can only be ascertained by an examination of the values that make up the solution. In this respect the Burmister solution indicated by the broken line, has as its basis a moduli ratio $E_1/E_2$ of 100 and a subgrade modulus of 1200 lb/in$^2$ whereas the Boussinesq solution indicated by the solid line, has as its basis a moduli ratio $E_1/E_2$ of 10, and a subgrade modulus that increases with the thickness of the pavement structure in accordance with the equation

$$E_2 = 2,800 + 1,200 \frac{h_1}{a} \text{ lb/in}^2$$

in which $h_1/a$ is the thickness of the pavement structure expressed as a multiple of the radius of the plate.

The acceptability of one solution relative to the other is open to some question but it would appear that the Boussinesq theory offers a more reasonable solution for the following reasons.

A moduli ratio $E_1/E_2$ of 10 is more reasonable for a flexible pavement system than is a ratio of 100.

The modulus of deformation accorded to the subgrade by the Boussinesq theory is in better agreement with the measured value. A separate analysis of similar tests conducted directly on the subgrade yielded an average subgrade modulus of deformation of 2,800 lb/in$^2$. This is seen to coincide with the Boussinesq solution for the case of $h_1/a = 0$.

In conclusion it may be said that the results of the analysis of the AASHO road test deflection data indicate that the Boussinesq stress distribution theory provides a more adequate picture of the deflection of a flexible pavement than does the layered solid theory. However, the problem of determination of representative elastic constants for deflection analyses remains critical for any further advancements in pavement deflection analysis.
Structural failure studies

As mentioned in Chapter 2, the primary purpose of this phase of the investigation is to examine the performance of the flexible pavement in terms of the permanent changes in the physical features of the roadway. This incorporates such factors as change in transverse and longitudinal profile and extent of cracking of the pavement surface. The major emphasis will be placed on the change in the transverse profile since it is from this data that information regarding plastic shear deformation and consolidation can be derived. The following paragraphs deal with the data as categorized and presented in Chapter 2.

Permanent surface deflection appears in the form of rutting within the wheel paths. Of particular interest are the two basic questions of the mechanics of rutting: that is, firstly, which component or components of the pavement system contribute the most to rutting and secondly, whether rutting is caused primarily by compression or by distortion.

As for the first question, a comparison should be made of the relative amounts contributed by the pavement structure and the subgrade. An analysis of this nature was carried out as part of the AASHO Road Test study. It is reported in reference 27, Report 5, that: "reduction in thickness of the surfacing, base and subbase courses was to a very large degree responsible for the rutting observed in the wheel paths of the pavement surface." This is confirmed by the plot of permanent deflection of surface versus change in structure thickness shown in Figure 22, as based on the data given in Figure 5 and 6. The test data in this figure applies only to those sections that did not fail during the test period and so may be classified as structurally adequate, in spite of the fact that some rut
Figure 22. Permanent Surface Deflection Versus Change in Thickness of the Pavement Structure for Structurally Adequate Pavements.
depths were as large as 1.3 inches. Thus on the basis of AASHO Road Test data it may be surmised that for structurally adequate pavements, the distress occurs almost exclusively within the pavement structure.

If all the pavement sections, including those that failed, are considered, the average contribution of each individual component layer to the permanent surface deflection, as reported in reference 27, Report 5, is as follows:

- Surface course = 32 percent
- Base course = 14 percent
- Subbase course = 45 percent
- Embankment = 9 percent

To make a proper interpretation of the above distribution of layer thickness change, the mechanics of the change in layer thickness should be considered. The following pertinent facts were established regarding that change from AASHO deflection studies:

1. Based on the average of the 1960 spring, summer, and fall trench studies "only 20 percent of the change in thickness of the surfacing and 4 percent of the change in subbase thickness could be accounted for by increases in density of the materials. In the case of the base only 30 percent of the change in thickness determined in the summer of 1960 could be accounted for by increases in density. However, the increase in the density determined in the spring accounted for all of the decrease in thickness of the materials." These statements apply to the thickness changes in the wheelpath.

2. With regard to the "between the wheelpath" study the following was observed:

"Densification of the asphaltic concrete accounted for all of the total thickness change in the surfacing material. The base course in nearly all
the trenches became thicker rather than thinner between the wheelpaths without undergoing much change in density. Between the wheelpaths there was considerable reduction in subbase thickness accompanied by a reduction on the average in subbase density."

From these studies and observations it was concluded that "changes in thickness of the components of the flexible pavements at the AASHO Road Test were due primarily to lateral movement of the materials."

Summarizing the findings outlined in the two preceding paragraphs, the following statements can be made for structurally adequate pavements:

1. Rutting occurred primarily within the pavement structure.
2. The rutting was due primarily to lateral displacements.

If these findings can be accepted as being undisputable then one must examine this pavement response in light of the known stress conditions.

Consider first of all the findings that distress occurred primarily within the pavement structure. The subgrade therefore must have behaved essentially as an elastic solid even under a very limited number of stress repetitions. This was corroborated by the study of the AASHO Road Test subgrade soil by Seed, Chan, and Lee (30). They found the soil to exhibit very high resiliency at relatively low deviator stresses.

This lends support to the concept of using the resilience modulus of the subgrade soil as an index to the extent and character of non-elastic behavior of a flexible pavement. In a very general way it may be said that, in the case of a subgrade of low resilience modulus, a large portion of the non-elastic deformation will occur within the subgrade, while in the case of a subgrade of high resilience modulus, the major portion of distress will occur within the pavement structure.
This statement is made with the assumption that the overall geometrical and load conditions as well as strength characteristics of the pavement structure remain the same. Speaking more strictly it might be said that the phenomenon of structural failure of flexible pavements is governed by the relative resilience of the subgrade with respect to the shear strength of the pavement structure.

For instance, two geometrically identical pavements resting on the same subgrade soil may fail in a different manner, if their shear strength characteristics are significantly different, as may be the case of a poorly compacted gravel base versus a well compacted macadam base. Good evidence supporting this comes from the AASHO Road Test itself, where it was found that much greater shear deformation occurred within the subbase than within the base, under otherwise analogous conditions. This can only partly be attributed to the difference in shearing resistance of the two materials relative to the stress level to which each of them was subjected.

Also, it may be anticipated that, no matter how deformable the subgrade, there always will be theoretically possible to construct a sufficiently thick pavement structure so that all the significant permanent deformation result from the pavement structure shear.

There exists in this respect a certain analogy with the shear failure phenomena of footings resting on soil: they fail in punching shear when their compressibility becomes great enough with respect to their shearing strength; if their strength is reduced in the same proportion as their deformation modulus, the mode of failure of footings
remains the same (14). This discussion points out to the shortcoming of basing the thickness design of a pavement solely on the characteristics of the subgrade soil, and again illustrates the need for an evaluation of the non-elastic behavior of the pavement system.

Finally, it should be remembered that, among the factors affecting the structural behavior and failure of flexible pavements, environmental conditions play an important role. This was dramatically demonstrated in the case of the AASHO Road Test as indicated by the following seasonal distribution of pavement failure:

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<th>Season</th>
<th>% Pavement Failure</th>
<th>Traffic Distribution</th>
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<tr>
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<tr>
<td>Winter</td>
<td>9%</td>
<td>21%</td>
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<tr>
<td>Spring</td>
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<td>25%</td>
</tr>
<tr>
<td>Summer</td>
<td>6%</td>
<td>28%</td>
</tr>
</tbody>
</table>

The fact that 80% of the failures occurred during the spring illustrates the necessity of including environmental effects in flexible pavement design. The detrimental effect of spring break-up lies primarily in the reduction of the shearing resistance of the soils leading to increased shear deformation, and a reduction in the modulus of deformation leading to increased deflections, larger bending strains and consequently greater distress in the form of fatigue failures.

Factors influencing rut depth

To investigate the effect of load repetition on rut depth, Figure
7 should be inspected. This figure shows how the depth of rut increases at a decreasing rate until it reaches a stabilized final value. Such a trend is typical for soils and granular materials in general subjected to stresses which are well below the ultimate strength of the material. These results illustrate that even "structurally adequate" pavements will develop appreciable rutting under a large number of load repetitions. They point out to the need for consideration, in pavement design, of both the elastic and non-elastic phenomena.

A second very important factor influencing rut depth is the vertical stress imposed by wheel loads through the pavement structure to the subgrade. Figure 23 shows a plot of rut depth versus vertical stress on the subgrade, as measured in both the 1959 and 1960 studies. As it can be seen there exists a stress level beyond which rutting rapidly increases and below which it remains essentially constant, indicating that the distress remained exclusively within the pavement structure. For the conditions of this road test this critical vertical stress level appears to lie between 9 and 11 lb/in$^2$, with a slight tendency of increasing with the wheel load.

This is a very significant finding. It may be interpreted to mean that, in AASHO Road Test conditions, rutting is extended into the subgrade soil at vertical subgrade stresses in excess of about 10 lb/in$^2$, while at lower stresses it remains almost exclusively within the pavement structure.

Other such possible findings would be most helpful for establishing rational design criteria along the principle of limiting subgrade stress.
Figure 23. Surface Rut Depth as a Function of Vertical Stress on the Subgrade.
CHAPTER 5

FINAL APPRAISAL AND RECOMMENDATIONS

The main objective of this study was to present a rational, mechanistic interpretation of observations and measurements made on flexible pavements in the AASHO Road Test and other pertinent experimental investigations. The data analysis was made primarily in the light of existing theories of pavement performance, however, some new concepts and approaches were advanced.

The present study was handicapped by the lack of many needed experimental data, particularly of those defining the mechanical behavior of pavement materials and soils in question. Thus, it was not feasible to carry all the analyses far enough to establish with absolute certainty the validity of the known hypotheses, and the soundness of theoretical approaches used. In spite of this, several conclusions that contribute to our understanding of structural behavior of flexible pavements were reached. They are presented below in the hope that they will bring improvements in existing design procedures and that they may serve as a basis for future development of mechanics of flexible pavements. The conclusions will be classified under subheadings corresponding to the main objects of the studies performed.

Stress distribution

The measurements of embankment pressures in the AASHO Road Test, as well as all other recent experimental investigations substantiate earlier findings that the load spreading abilities of flexible pavements
with conventional, untreated bases are very limited. However, they also point out to the importance for stress distribution of environmental factors such as temperature and moisture content of the components of the pavement structure.

At normal temperatures and under slowly moving loads the measured vertical stresses generally follow the pattern predicted by the classical Boussinesq theory for a homogenous solid. Directly under the load they are considerably higher than those predicted by the conventional layered solid (Burmister) theory. They are the highest when the subgrade, as well as the pavement structure are practically saturated with moisture; they are the lowest during the frost periods. The stresses are also affected by the vehicle speed; they are lower under fast moving loads.

It is of interest to note that practically all stress measurements in flexible pavements furnish records of vertical stresses only. There is practically no data on actual horizontal and shearing stresses in pavements.

The explanation of the existing observations is as follows: The pavement acts as a very complex layered solid which, because of lack of tensile strength of some layers exhibits only a very limited slab action. However, this action is considerably increased when all pavement layers become frozen and acquire greater tensile strengths. It also may be more pronounced if the tensile strength of these layers is increased by a treatment with cement or bitumen.
Deflections

Because of lack of reliable data on deformation characteristics of pavement materials and soils no attempt was made to compare absolute magnitudes of deflections with the corresponding theoretical values. However, it was possible to confirm the earlier findings that the deflection basins have a very limited extension and are not at all comparable in size with those predicted by the layered solid theories. It was shown that the basin shapes follow relatively closely the shapes predicted by considering the pavements system to be a layered solid with Boussinesq stress distribution.

These findings are in agreement with those presented in the preceding paragraph and can be explained, generally, by the same arguments.

Structural failure

Structural failure of a flexible pavement may be defined as a state in which repeated application of a specified wheel load results in ever-increasing plastic deformations of the pavement surface. If this definition is accepted, a pavement should be considered structurally adequate for a certain wheel load if the depth of rut caused by repeated application of that wheel load reaches a final value which does not increase with further load applications.

Of course a flexible pavement may be structurally adequate and become inserviceable if the rut depth exceeds certain limits. It is to be expected that the safety factors against structural failure will be of such magnitude that the pavements remain serviceable for the specified number of load repetitions.
Observations indicate that the phenomenon of structural failure of flexible pavements is governed by the relative resilience or compressibility of the subgrade soil with respect to the shear strength of the pavement structure.

In the case of a relatively weak, compressible subgrade and a strong, well compacted, but thin pavement structure, structural failure occurs essentially through punching shear (Figure 2). Rutting is then due primarily to compression and distortion of the subgrade soil.

On the other hand, in the case of a relatively firm, incompressible subgrade and a poorly compacted or generally weak pavement structure, as well as in the case of any subgrade overlain by a very thick pavement structure, failure phenomena may resemble more the phenomenon of general shear of an incompressible soil under a footing (Figure 1). Rutting is then caused primarily by distortion or shear deformation of the pavement structure. Of course, all possible combinations of the two extreme types of phenomena occur in intermediate cases.

It is of interest to note that there apparently exists a critical subgrade stress level beyond which the rutting is extended into the subgrade soil. If the vertical stresses on the subgrade never exceed the critical level, ruts are formed primarily by shear deformations in the pavement structure. This finding justifies the selection of limiting subgrade stress as one of the major design criteria in flexible pavement design.

**Implications concerning existing design procedures**

The findings expressed in the preceding paragraphs have several
implications concerning the soundness of some of the existing design procedures for flexible pavements.

The fact that stress distribution at normal temperatures in conventionally constructed flexible pavements follows relatively closely the pattern predicted by the Boussinesq theory justifies the use of that theory for evaluations of equivalent wheel loads. Such a procedure has been extensively practiced by the Corps of Engineers, (3), (42) and many other organizations (43).

The existence of a critical vertical subgrade stress level, which, quite obviously, must be related to the strength and deformation characteristics of the subgrade, partly supports the basic design philosophy of the CBR-Method at least for conventionally constructed flexible pavements with untreated bases. It also points out to the inadequacy of that method to take into account the better spreading abilities of improved surfacings (plant-mix hot rolled asphaltic concrete) as well as of bases possessing some tensile strength (bituminous macadam and soil cement).

The variation of vertical subgrade stresses with pavement temperature, subgrade moisture conditions and vehicle speed suggests that such factors should find their place in design. They probably could be introduced in a relatively simple manner during evaluation of weighted number of maximum wheel load applications.

The results of this investigation confirm the soundness of a rational, mechanistic approach to design of flexible pavements. It is more obvious than ever that no empirical formula, no matter how elaborate,
can properly embrace the illimited variety of conditions that may be encountered in engineering practice.

**Recommendations for further study**

In the light of findings of this investigation it is evident that there exists a great need for fundamental research in the area of mechanics of flexible pavements. Several new studies could greatly contribute toward understanding of basic phenomena involved and open the road toward the establishment of reliable rational methods of pavement design. The most urgent among these are listed in the following paragraphs.

Concerning the stress distribution, it would be desirable to acquire knowledge on the actual magnitude of horizontal normal and shear stresses in pavement structures, both in the field as well as in the better controlled laboratory conditions. Additional measurements of the effect of tensile strength of pavement layers on vertical stress distribution would also be helpful. Measurements of actual deflections of pavements under carefully controlled conditions should be undertaken, preferably in the laboratory on full-scale models. These measurements should be accompanied by systematic testing of mechanical properties of pavement layers. Special attention should be devoted to fundamental studies of behavior of materials such as coarse-aggregate macadams, bound or unbound, which are probably the least understood. Also, more knowledge will be needed on anisotropy of pavement layers and its effect on stress distribution and deflections.
Qualitative tests on small-scale models of pavements of different geometrical and strength features, resting on a variety of subgrades, should be performed with the primary purpose of obtaining displacement and shear patterns and fully explaining the mechanisms of pavement failure in different conditions. The great potential value of this kind of tests has been proven in several bearing capacity and earth pressure investigations, (14), (44).

Once all this information becomes available, it will not be difficult to develop a rational method of design of flexible pavements, which would be general enough to allow extrapolation of existing experience to any conditions encountered and yet simple enough that it can be used by every highway engineer. The results of the present investigation indicate that this method should include consideration of both the elastic and plastic phenomena in flexible pavements.
REFERENCES


APPENDIX: TABLES
TABLE 1
VERTICAL STRESS ON THE SUBGRADE UNDER DIFFERENT PAVEMENT STRUCTURE THICKNESSES

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<th>Single Axle Load (kips)</th>
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TABLE 2

VERTICAL STRESS ON THE SUBGRADE MEASURED AT DESIGNATED WHEEL LOADS
AND TIRE PRESSURES

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(d) Track Vehicles

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### TABLE 3

**VARIATION IN VERTICAL STRESS ON SUBGRADE ALONG THE LONGITUDINAL REFERENCE AXIS**

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TABLE 4

VARIATION IN VERTICAL STRESSES ON SUBGRADE ALONG TRANSVERSE REFERENCES AXIS

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### TABLE 5
MEASURED PAVEMENT DEFLECTIONS

| Axle Load  | Design Section | Longitudinal Distances (Ft.) | 6 | 5 | 4 | 3 | 2 | 1 | 2/3 | 1/3 | 0 | 1/3 | 2/3 | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|------------|----------------|------------------------------|---|---|---|---|---|---|-----|-----|---|-----|-----|---|---|---|---|---|---|---|---|
| 18 Kips    |                | 5-6-12                        | 0 | 1 | 1 | 4 | 9 | 21 | 26  | 30 | 32 | 28 | 27 | 25 | 13 | 7 | 4 | 2 | 1 | 0 |
|            |                | 4-6-4                         | - | - | 0 | 8 | 34 | 48 | 60  | 63 | 61 | 53 | 42 | 20 | 6  | 1 | 0 | - | - | - |
| Single     |                | 5-6-4                         | - | 0 | 3 | 14| 32 | 40 | 43  | 44 | 42 | 35 | 32 | 18 | 8  | 4 | 2 | 1 | 0 |
|            |                | 4-6-12                        | - | 0 | 2 | 4 | 9 | 25 | 36  | 40 | 43 | 40 | 35 | 29 | 15 | 8 | 4 | 2 | 1 | - |
|            |                | 5-6-4                         | - | 0 | 4 | 14| 29 | 34 | 38  | 40 | 38 | 37 | 34 | 22 | 12 | 6 | 2 | 0 | - | - |
| 32 Kips    |                | 5-6-12                        | 0 | 2 | 5 | 6 | 6  | 7  | 7   | 7  | 6  | 8  | 8  | 9  | 10 | 9  | 8  | 6 | 5 | 4 | 3 |
|            |                | 4-6-4                         | 0 | 4 | 19| 54 | 68 | 76 | 79  | 77 | 71 | 64 | 50 | 65 | 70 | 78 | 85 | 80 | 80 | 75 | 46 | 42 | 23 | 12|
| Tandem     |                | 5-6-4                         | 0 | 2 | 13| 37 | 48 | 54 | 58  | 54 | 50 | 44 | 32 | 49 | 57 | 62 | 64 | 61 | 57 | 49 | 30 | 16 | 6  | 4 |
|            |                | 4-6-12                        | 1 | 4 | 10| 20 | 24 | 27 | 28  | 27 | 25 | 23 | 20 | 26 | 30 | 32 | 33 | 32 | 30 | 26 | 16 | 10 | 6  | 3 |
|            |                | 5-6-4                         | 3 | 7 | 16| 29 | 32 | 34 | 35  | 34 | 32 | 31 | 28 | 36 | 40 | 42 | 44 | 43 | 40 | 36 | 25 | 16 | 10 | 6  |
| Transverse |                | 18 Kips                       | 1.0|0.82|0.57|0.45|0.32|0.25|0.12|0.07|0.025|0.5 |0.82|1.00|1.10|1.83|2.00|2.82|
|            |                | 4-6-4                         | 43 |48 |50  |55  |57  |58  |63  |63  |64  |64  |61  |52  |52  |48  |26  |24  |18  |
| Single     |                | 5-6-4                         | 0.9|0.5 |0.25|0.2 |0.1 |0.025|0.40|0.50|0.65|1.0 |1.8|1.9 |

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**Notes:**
- Measurements are in inches.
- Distances are from the reference point.
### TABLE 6

**Changes in Thickness and Density, Outer Wheelpath, Trench Program, Spring 1960**

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(b) Base

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(b) Base

| 3 4-3-8 | 7.98 7.37 | 131.7 134.0 | -0.61 | -0.14 |
| 4-6-4  | 3.74 3.72 | 133.6 137.4 | -0.02 | +0.11 |
| 4-6-8  | 8.14 7.56 | 134.0 128.3 | -0.58 | +0.35 |
| Mean   | 6.62 6.22 | 133.1 133.2 | -0.40 | -0.01 |

| 4 5-6-12 | 11.38 11.04 | 130.3 143.4 | -0.34 | -1.14 |
| 5-6-8   | 7.98 7.19   | 136.8 135.2 | -0.79 | +0.09 |
| 5-3-12  | 12.32 11.02 | 137.3 135.2 | -1.30 | +0.19 |
| Mean    | 10.56 9.75  | 134.8 137.9 | -0.81 | -0.24 |

| 5 5-9-12 | 12.12 11.54 | 136.7 131.2 | -0.58 | +0.49 |
| 5-6-12  | 11.96 10.84 | 135.9 134.7 | -1.12 | +0.11 |
| 5-9-8   | 7.88 7.46   | 129.3 135.3 | -0.42 | -0.37 |
| Mean    | 10.65 9.95  | 134.0 133.7 | -0.71 | +0.02 |

| 6 6-6-16 | 15.60 14.91 | 139.5 131.3 | -0.69 | +0.92 |
| 6-9-12  | 11.88 11.48 | 136.9 134.8 | -0.40 | +0.18 |
| 6-9-16  | 16.54 16.27 | 136.6 141.3 | -0.27 | -0.57 |
| Mean    | 14.67 14.22 | 137.6 135.8 | -0.45 | +0.19 |
| Over-all Mean | 10.63 10.03 | 134.9 135.2 | -0.59 | -0.02 |

1. Cores taken at 1, 6 and 11 ft from pavement centerline at third points in section; data are interpolations from these measurements.
2. Thickness determined from transverse profile plot at maximum depth of rut; surface profiles prepared from 25 precise level measurements at 1-ft intervals.
3. Average of two tests made at randomly selected locations.
4. Average of two tests in outer wheelpath, one from each side of trench.
## TABLE 7
CHANGES IN THICKNESS AND DENSITY, OUTER WHEELPATH,
TRENCH PROGRAM, SUMMER 1960

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| (c) Subbase |

| 3 4-3-8 | 7.71 | 6.85 | 131.7 | 136.2 | -0.86 | -0.26 |
| 4-6-4  | 3.80 | 3.70 | 133.6 | 137.9 | -0.10 | -0.12 |
| 4-6-8  | 8.14 | 7.24 | 134.0 | 132.7 | -0.90 | +0.08 |
| Mean   | 6.55 | 5.93 | 133.1 | 135.6 | -0.62 | -0.12 |
| 4 5-6-12 | 11.40 | 11.12 | 130.3 | 129.0 | -0.28 | +0.11 |
| 5-6-8  | 7.98 | 7.48 | 136.8 | 130.4 | -0.50 | +0.37 |
| 5-3-12 | 11.76 | 11.30 | 137.3 | 132.9 | -0.46 | +0.38 |
| Mean   | 10.38 | 9.97 | 134.8 | 130.8 | -0.41 | +0.31 |
| 5 5-9-12 | 12.16 | 11.98 | 136.7 | 139.9 | -0.18 | -0.28 |
| 5-6-12 | 11.98 | 10.80 | 135.9 | 134.8 | -1.18 | +0.10 |
| 5-9-8  | 7.78 | 7.38 | 129.3 | 136.9 | -0.40 | -0.46 |
| Mean   | 10.64 | 10.05 | 134.0 | 137.2 | -0.59 | -0.25 |
| 6 6-6-16 | 15.60 | 15.00 | 139.5 | 138.6 | -0.60 | +0.10 |
| 6-9-12 | 12.08 | 11.12 | 136.9 | 130.3 | -0.96 | +0.58 |
| 6-9-16 | 16.54 | 16.20 | 136.6 | 142.5 | -0.34 | -0.71 |
| Mean   | 14.74 | 14.11 | 137.7 | 137.1 | -0.63 | +0.06 |
| Over-all Mean | 10.58 | 10.01 | 134.9 | 135.2 | -0.56 | -0.02 |

1 Cored taken at 1, 6 and 11 feet from pavement centerline at third points in section; data are interpolations from these measurements.

2 Thickness determined from transverse profile plot at maximum depth of rut; surface profiles prepared from 25 precise level measurements at 1-ft intervals.

3 Average of two tests at randomly selected locations.

4 Average of two tests in outer wheelpath, one from each side of trench.
TABLE 8

CHANGES IN THICKNESS AND DENSITY, OUTER WHEEelpATH,
TRENCH PROGRAM, FALL 1960

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(b) Base

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<th>Density (pcf)</th>
<th>Change in Thickness (in.)</th>
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<td>Change in Thickness (in.)</td>
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| 6           | 4-3-16          | 15.47         | -0.81                    |
|             | 4-6-12          | 11.40         | +0.04                    |
|             | 4-6-16          | 11.31         | +0.82                    |
|             | 4-9-12          | 11.31         | -0.69                    |
|             | 4-9-16          | 13.82         | -0.30                    |
|             | 5-6-12          | 10.94         | -0.87                    |
|             | 5-6-16          | 13.59         | 0.03                     |
|             | 5-9-12          | 11.43         | -0.65                    |
|             | 5-9-16          | 12.07         | -0.31                    |
|             | 6-3-12          | 12.07         | -0.55                    |
|             | 6-6-8           | 7.67          | -0.09                    |
|             | 6-6-12          | 12.01         | -0.23                    |
| Mean        | Over-all Mean   | 12.71         | -0.44                    |

| Mean        | 9.95            | 9.50          | 135.7                    |

1. Cores taken at 1, 6 and 11 ft from pavement centerline at third points in section; data are interpolations from these measurements.

2. Thickness determined from transverse profile plot at maximum depth of rut; surface profiles prepared from 25 precise level measurements at 1-ft intervals.

3. Average of two tests made at randomly selected locations; nuclear probe used to determine base density, Rainhart equipment to determine subbase density.

4. Average of two tests in outer wheelpath, one from each side of trench.
TABLE 9

NUMBER OF AXLE REPETITIONS REQUIRED TO PRODUCE SPECIFIED RUT DEPTHS

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<tr>
<th>Design Section</th>
<th>Axle Load</th>
<th>Number of Load Repetitions (Per cent of total 1,114,000)</th>
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<td></td>
<td></td>
<td>.25 in.</td>
<td>.50 in.</td>
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<td>12 Kips single</td>
<td>95</td>
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<td></td>
<td>18 Kips single</td>
<td>11</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>22.4 Kips single</td>
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<td>8</td>
</tr>
<tr>
<td></td>
<td>30 Kips single</td>
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<td>7</td>
</tr>
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<td>5-6-8</td>
<td>18 Kips single</td>
<td>12</td>
<td>80</td>
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<td></td>
<td>22.4 Kips single</td>
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<td>30 Kips single</td>
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TABLE 10
A COMPARISON OF RUT DEPTH AND VERTICAL SUBGRADE STRESS

<table>
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<tr>
<th>Pavement Structure Thickness (in.)</th>
<th>Axle Load Kips, Single</th>
<th>Vertical Stress on the Subgrade (lb/in²)</th>
<th>Rut Depth (In.) Year</th>
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TABLE II

PHYSICAL CHARACTERISTICS OF EMBANKMENT SOIL

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<th>Textural Classification:</th>
<th>Yellow Brown</th>
<th>Silty Clay (A-6)</th>
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<td>Grain Size Distribution:</td>
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<td>Percent of Material Finer Than</td>
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<td></td>
<td>No. 4</td>
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<tr>
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<td>No. 40</td>
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<tr>
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<td>No. 60</td>
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<td>.05 mm</td>
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<tr>
<td></td>
<td>.002 mm</td>
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Atterberg Limits:  
- Liquid Limit = 29.4 percent  
- Plastic Limit = 16.5 percent

Specific Gravity: 2.71

Compacted Density: 115.4 lb. per cu. ft.

Corresponding Moisture Content: 15.5 percent
TABLE 12

PHYSICAL CHARACTERISTICS OF SUBBASE MATERIAL

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<thead>
<tr>
<th>Textural Classification:</th>
<th>Sand-Gravel</th>
<th>Mulch</th>
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<td></td>
<td>1 inch</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>3/4 inch</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>1/2 inch</td>
<td>90</td>
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<tr>
<td></td>
<td>No. 4</td>
<td>71</td>
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<td></td>
<td>No. 10</td>
<td>52</td>
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<td></td>
<td>No. 40</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>No. 200</td>
<td>6.5</td>
</tr>
</tbody>
</table>

Atterberg Limits: Non-plastic
Specific Gravity: 2.70
Compacted Density: 134.5 lbs. per cu. ft.
Corresponding Moisture Content: 3.8 percent
### TABLE 13
PHYSICAL CHARACTERISTICS OF BASE COURSE

<table>
<thead>
<tr>
<th>Textural Classification: Crushed Dolomitic Limestone</th>
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<td>Grain Size: 1-1/2 inch</td>
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<tr>
<td>No. 100</td>
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<tr>
<td>No. 200</td>
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Atterberg Limits: Non-plastic
Specific Gravity: 2.74
Compacted Density: 140 lbs. per cu. ft.
Corresponding Moisture Content: 4.2 percent
TABLE 14
PHYSICAL CHARACTERISTICS OF BINDER AND SURFACE COURSES

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<th>Binder Course</th>
<th>Surface Course</th>
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<td>5.2 percent</td>
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<td>2000 lbs.</td>
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<td>Percent Voids</td>
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<td>Compacted Density</td>
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<td>Percent Voids as Compacted</td>
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TABLE 15
STRENGTH PARAMETERS OF COMPONENTS OF PAVEMENT STRUCTURE

<table>
<thead>
<tr>
<th>Component</th>
<th>Cohesion (lb/in²)</th>
<th>Angle of Shearing Resistance (degrees)</th>
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**MODULUS OF DEFORMATION OF AASHO ROAD TEST PAVEMENT STRUCTURE COMPONENTS**

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\(^a\) Initial tangent modulus

\(^b\) Applies to the entire pavement structure
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# TABLE 20

**INFLUENCE FACTOR FOR SETTLEMENT—UNIFORMLY LOADED CIRCULAR AREA**

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