NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 10

THEORETICAL ANALYSIS OF STRUCTURAL BEHAVIOR OF ROAD TEST FLEXIBLE PAVEMENTS

HIGHWAY RESEARCH BOARD NATIONAL ACADEMY OF SCIENCES - NATIONAL RESEARCH COUNCIL

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THEORETICAL ANALYSIS OF STRUCTURAL BEHAVIOR OF ROAD TEST FLEXIBLE PAVEMENTS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by Highway Planning and Research funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Commerce.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

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FOREWORD

By Staff

Highway Research Board

This report will be of interest to highway engineers engaged in flexible pavement design and research. It analyzes data from the AASHO Road Test and other similar experiments as they apply to the structural behavior of flexible pavements in terms of existing theories. It also touches on the development of new design theories and contains recommendations for further research to consider the mechanical behavior of layered flexible pavement systems in terms of both elastic and plastic phenomena. From this study it is clear that of the existing pavement design theories based on engineering mechanics, rather than on empiricism, none adequately considers all the variables affecting rational pavement design. Furthermore, experimental substantiation of the theoretical hypotheses has not been adequate. However, the knowledge gained from this study will be of considerable value as a foundation for further work on the development of rational pavement design methods.

Highway engineers are presently at a disadvantage in attempting paving structure designs which are both economical and rational because existing theories are, for many reasons, too limited in scope to provide satisfactory solutions to actual problems. This situation is largely due to the manner in which these theories have been developed over the years. Not too many years have passed since designs were primarily by rule-of-thumb, because little information existed in relation to either materials engineering or the mechanical behavior of pavement structures when subjected to various loadings. As both technology and service demands upon highways increased, particularly accentuated by World War II, many new rational design methods came into being which ranged in nature from empirical to theoretical, and materials engineering assumed its rightful place as an engineering science. For varied reasons, these methods largely have not been properly evaluated for their broad applicability to solutions which must place in proper perspective the relationship between the engineering properties of materials and the mechanical behavior of pavement structures.

In order to properly evaluate the existing theories, or possibly to develop new theories, a study of the mechanical behavior of flexible pavements utilizing test data reported in the literature was undertaken. Stresses, deflections, and failure mechanisms received particular attention, and not until the existing hypotheses are verified by such work as this will highway designers be able to proceed with greater confidence in their designs.

Several definite conclusions have been reached which will increase the understanding of the structural behavior of flexible pavements although, on the other hand, the lack of certain experimental data prevented the validity of other hypotheses from being established with any certainty. In general, it is concluded that stress distribution is a function of both the treatment given to bases and environmental factors such as temperature and moisture content of the pavement components; deflection basin shapes are more accurately predicted on the basis of assuming a layered solid and Boussinesq stress distribution; and 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. The implications of these findings in respect to the soundness of some of the existing design procedures are discussed and further emphasize the inadequacy of empirical design methods. Recommendations are made for further work regarding the development of a rational design method for flexible pavements. This would encompass elastic and plastic phenomena so that the method would be broad enough in scope to cover any given conditions and yet be readily adaptable by the engineer to everyday use.

This study, conducted at Georgia Institute of Technology, constitutes an interim report on the first period of work in the general problem area of translating the AASHO Road Test results to local conditions. It has concentrated on the structural behavior of flexible pavements, and a continuation of research in this problem area has been planned to similarly study rigid pavements. The data realized from the current study are viewed as providing the engineer with a good basis for analyzing, developing, or revising flexible pavement design methods and criteria. It is expected that further substantiation of the findings will do much to culminate a universal agreement among engineers as to acceptable design methods for coping with the many varied circumstances associated with the ever-changing demands on today's highways.

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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 Boussingesq 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.

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 the latter 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 is extended into the subgrade. This finding justifies 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.

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.

CHAPTER ONE

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 used. Although 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 analyses 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 empirical relationships between pavement performance on the one hand and vehicle load intensity and soil type on the other (*1, 2*). Intensified airport pavement studies initiated during World War II led ultimately to a semi-empirical extension of the existing relationships into more general criteria, including such variables as tire pressure and number of load applications (*13*). In the same period several theoretical and semi-theoretical methods for pavement design were proposed (*4-11*), which tried to incorporate into design 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, as follows:

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.

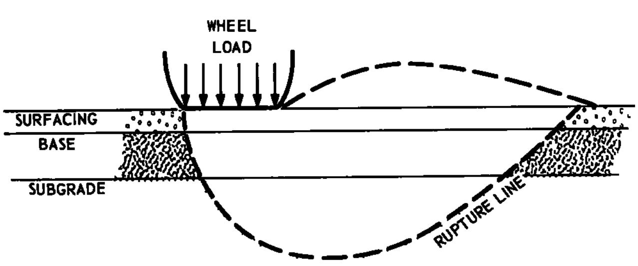

Figure 1. General shear failure of a flexible pavement.

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 of 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 assume that a pavement system fails in general shear, similarly to bearing capacity failure of shallow footings on dense soils (Fig. 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, ϕ. 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.

Among the known methods of this group, the following 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 of 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, μ. 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 limitation of the theoretical deflection of the surface under load to 0.1 in.

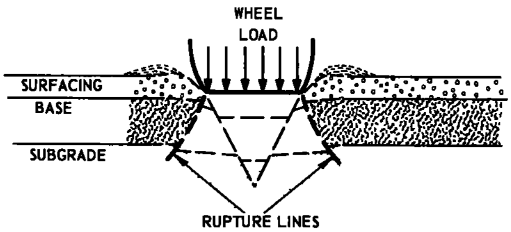

Figure 2. Punching shear failure of a flexible pavement.

The U. S. Navy method uses essentially the same design criterion of limiting deflection, which is set at 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 the 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 viscoelastic 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, as follows:

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

2. It makes it possible to design pavements 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 does not expressly furnish information about pavement displacements.

In spite of the potential merits of this design approach, the two known methods based on it 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 (Fig. 1). Experience shows, however, that pavements more often fail by punching failures, similar to those observed (Fig. 2) under dynamically loaded footings as well as under ordinary footings on soft, loose and layered soils (*14, 15*). It should be added that the amount of investigation done to correlate design findings 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 merit. 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 Ref. *16*), which show the measured vertical

* 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.

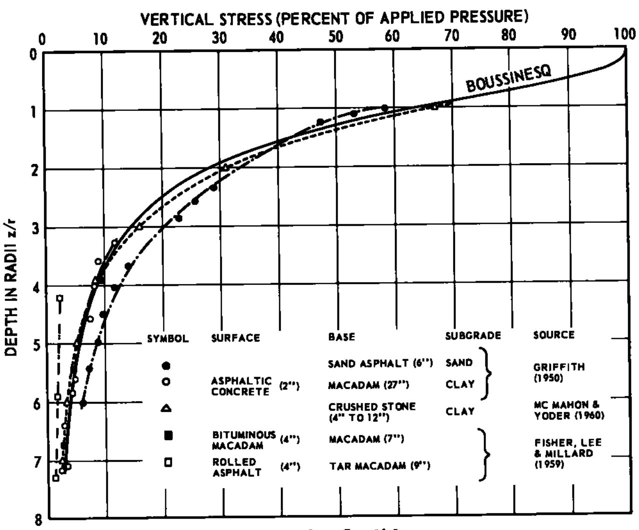

Figure 3. Vertical stresses under flexible pavements.

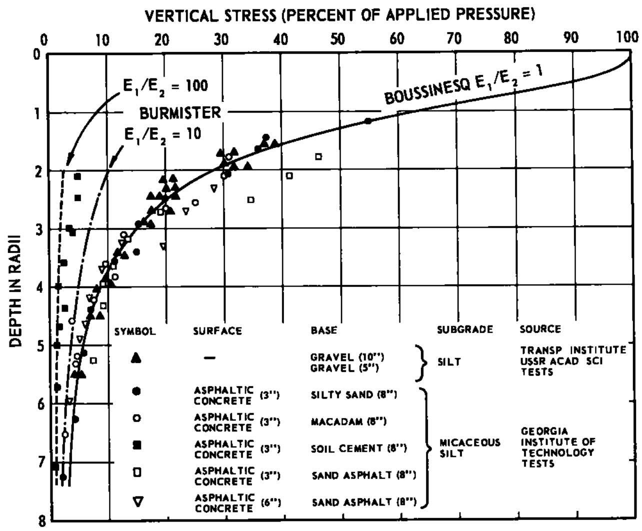

Figure 4. Vertical stresses under flexible pavements.

stresses, σ_z, directly under the loads applied at the surface of flexible pavements, as measured in five full-scale and model investigations, as follows:

1. In-situ tests performed by the Corps of Engineers, U. S. Army, with actual airplane loads on an airfield pavement section at Marietta, Ga. (*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 × 8-ft pavement sections (*24*).
4. Model tests performed at the Georgia Institute of Technology (*25, 26, 18, 19*) with truck tire loads on 12 × 8-ft pavement sections.
5. Model tests performed at the Transportation Institute of the USSR Academy of Sciences with rigid plates on 8 × 5-ft pavement sections (*17*).

In both figures the 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 shown in Figure 4 are the theoretical stress distributions for a two-layer homogeneous isotropic solid having a Poisson's ratio, μ, of 0.50.

Together with observations of size of deflection basins, which all appear to be more confined to the vicinity of the loaded area than is indicated 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 foregoing remarks do not discredit all evaluations of stresses and deflections based on the layered solid theories. They merely point out one of the uncertainties of known methods of the second group in their present form. More basic research is needed to shed light on the actual behavior of flexible pavements under working conditions. Some ideas about necessary theoretical investigations in this direction have been expressed elsewhere (*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 TWO

DATA ON STRUCTURAL BEHAVIOR OF THE AASHO ROAD TEST FLEXIBLE PAVEMENT

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 is being made to examine the structural behavior of AASHO Road Test flexible pavements strictly in a quantitative manner, with attention focused 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 is 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:

1. The nature of the load distribution throughout the depth of the 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 is taken primarily from the results of the AASHO Road Test and is supplemented by results from other road tests and investigations where possible.

In this chapter, the test road data pertinent to the analysis are assembled in accordance with the foregoing categories. The actual analysis of the data is presented in Chapter Four.

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 were 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 have 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 are available. The data given in Table 1 represent the mean of seven weekly observations taken during the summer of 1959, at the designated sections † of Loop 4 (*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 (Table 2) were obtained from the special studies (*27*, Report 6). A pressure cell was installed at the embankment level in 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 psi. 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 are also separated in accordance with wheel and axle configurations. This was done for convenience of analysis.

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 the AASHO Road Test report (*27*, Report 5). For the purpose of comparing the theoretical and the observed pattern of vertical stress

* Hereafter *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

† The sections are designated by three numbers, which refer to the respective thicknesses of surfacing, base course and subbase

TABLE 1

VERTICAL STRESS ON THE SUBGRADE UNDER DIFFERENT PAVEMENT STRUCTURE THICKNESSES

SINGLE-AXLE LOAD (KIPS)	TIRE PRESSURE (PSI)	DESIGN SECTION	VERTICAL STRESS (PSI)
18	67.5	5-0-12	11.0
18	67.5	6-0-12	9.0
18	67.5	3-6-12	9.3
18	67.5	5-6-12	6.3

variation with distance from point of load application, data obtained for the small and the medium scraper were selected. These particular data were selected because they included a wide range of wheel loads and tire pressures. The data corresponding to stresses along longitudinal and transverse reference axes are 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

TABLE 2
VERTICAL STRESS ON SUBGRADE

Vehicle	Axle Load (kips)	Tire Pressure (psi)	Vertical Stress (psi)
(a) Single-Axle, Single-Wheel Vehicles			
Small scraper	15.4	45	5.2
	15.4	30	5.3
	28.0	45	9.4
	28.0	30	8.9
	32.9	45	10.3
	32.9	30	10.0
	39.6	45	11.7
	39.6	30	11.4
	40.9	45	13.8
	47.0	45	13.5
Medium scraper	21.4	45	6.8
	21.4	30	6.1
	35.0	45	7.8
	35.0	30	8.0
	45.6	45	12.1
	45.6	30	11.4
	52.5	45	11.4
	52.5	30	11.5
	55.0	45	11.9
	67.5	45	14.6
GOER	13.4	20	5.9
	23.4	25	9.5
	25.8	30	9.1
	30.2	35	13.0
Cargo trailer K-7	22.1	16	6.1
L-1	31.3	16	7.3
Fluid transporter	10.2	8	2.5
Truck	2.0	24	0.3
	6.0	45	0.9
	12.0	75	1.9
	12.0	80	4.2
	12.0	75	4.9
(b) Single-Axle, Dual-Wheel Vehicles			
Truck	18.0	67.5	6.3
	18.0	80	5.4
	18.0	80	5.3
	18.0	80	5.7
	18.0	80	6.1
	22.4	100	6.0
	22.4	80	6.8
	30.0	80	10.2
	30.0	80	10.2
(b) Single-Axle, Dual-Wheel Vehicles (Cont'd)			
Tank transporter	8.0	90	2.8
	8.5	90	2.9
	8.6	90	3.0
	8.8	90	3.1
	9.5	90	3.2
	14.2	90	4.4
	14.3	90	4.4
	18.4	90	4.8
	18.7	90	5.3
	18.8	90	5.0
	18.9	90	5.5
	19.0	90	4.7
	19.1	90	5.0
	19.3	90	5.2
	19.4	90	5.3
	19.6	90	5.0, 5.2
	20.5	90	5.8
	20.7	90	5.8
	20.8	90	6.0
	21.2	90	6.2
	21.9	90	6.2, 6.4
	27.5	90	8.4
	27.8	90	6.4
	27.9	90	7.7, 8.6
	28.4	90	6.6
	28.6	90	6.2, 7.1
	28.7	90	7.8
	31.6	90	9.8
	31.8	90	9.8
	32.2	90	9.6
	32.3	90	9.4, 9.6
	32.7	90	8.2
	32.9	90	8.9
	33.4	90	8.0
	33.9	90	8.5, 7.8
	42.5	90	12.1
	43.3	90	12.8
(c) Tandem-Axle, Dual-Wheel Vehicle			
Truck	24.0	80	4.4
	32.0	80	4.1
	32.0	80	4.9
	32.0	80	5.6
	32.0	80	5.2
	40.0	80	5.8
	40.0	80	5.1
(d) Track Vehicles			
	19.8	—	2.4
	91.6	—	9.4

road test. From these data, an attempt is made to determine the most significant environmental factors that cause appreciable variations in vertical stresses during the course of a year. The data are presented graphically in Figure 16.

DEFLECTION STUDIES

A considerable portion of the AASHO Road Test 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 development of a relationship between deflection and pavement performance. In the present investigation, a study is made of the applicability of the different methods of deflection analysis to describe the behavior of flexible pavements.

The methods of analysis examined are all based on the theory of elasticity. Thus, insofar as pertinent test road data are concerned, the deflection basin study constitutes 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 Ref. *27*, Report 5.

It should be pointed out that the deflections were measured in one of two ways: first, by means of electronic recording devices (LVDT) utilizing settlement rods, hence the recorded deflections are relative to some finite depth; second, by means of Benkelman beams, 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 ft below the surface of the embankment. The deflections along the longitudinal and the transverse axes were of primary concern to this investigation, hence only these data have been selected (Table 5).

TABLE 3

VARIATION IN VERTICAL STRESS ON SUBGRADE ALONG LONGITUDINAL REFERENCE AXIS

WHEEL LOAD (KIPS)	TIRE PRESSURE (PSI)	VERTICAL STRESS (PSI) AT DESIGNATED DISTANCE						
		18 IN.	12 IN.	6 IN.	0	6 IN.	12 IN.	18 IN.
20.45	45	7.8	10.0	12.0	13.8	12.0	9.6	7.1
		6.6	9.1	10.7	11.4	10.7	8.7	6.6
23.50	45	8.2	10.7	12.8	13.5	12.4	10.3	7.8
		6.9	9.1	10.0	11.0	10.0	8.0	5.9
27.50	45	7.1	9.2	11.0	11.9	11.4	9.6	7.5
		6.2	8.4	10.0	10.8	10.0	8.5	7.0
33.75	45	9.8	12.3	14.1	14.6	13.7	11.6	9.1
		8.4	10.9	12.8	13.4	12.6	10.9	8.4
7.70	30	1.8	3.7	5.0	5.3	5.2	4.1	2.8
21.40	30	2.8	4.1	5.0	5.3	4.8	4.1	3.4
		2.8	4.3	5.3	6.0	5.7	5.2	3.9
14.00	30	4.3	6.4	7.5	8.2	7.5	6.4	4.6
		3.9	6.0	8.0	8.9	8.4	6.9	4.8
16.45	30	5.3	8.5	9.4	10.0	9.3	7.5	5.3
7.70	45	2.1	3.6	4.8	5.2	4.8	3.9	2.5
		2.2	3.1	3.7	3.9	3.7	3.2	2.6
10.70	45	2.3	4.3	6.0	6.8	6.0	4.6	3.2
		2.7	3.9	5.2	5.7	5.0	4.1	3.2
14.00	45	4.3	6.0	7.1	7.3	6.9	5.3	3.9
32.90	45	6.1	8.2	9.6	10.3	9.6	8.4	6.1
		5.2	6.9	8.0	8.5	8.0	6.9	5.2
17.50	30	5.1	6.5	7.6	8.0	7.6	6.2	4.8
		4.5	5.7	6.6	7.0	6.6	5.9	4.8
19.80	30	6.6	9.1	10.7	11.4	10.9	8.7	6.6
		5.9	7.7	9.4	10.0	9.4	7.7	5.9
22.80	30	7.1	9.2	10.7	11.4	10.7	9.2	6.8
		6.2	8.4	9.8	10.3	9.8	8.0	5.9
26.25	30	6.4	9.2	10.7	11.5	11.1	8.9	7.1
		6.6	8.0	9.4	9.6	9.4	8.0	6.6
17.50	45	4.3	6.0	7.1	7.8	7.5	6.8	5.3
		4.6	6.0	7.5	7.8	7.3	6.0	4.8
19.80	45	5.3	7.5	9.0	9.6	9.3	7.5	5.5
22.80	45	5.0	7.8	11.1	12.1	11.2	8.9	6.0
		5.5	8.2	10.7	11.9	11.2	9.4	6.6
26.25	45	6.8	8.9	10.1	11.4	11.0	10.0	8.2
		6.8	8.6	10.0	10.7	10.3	8.9	7.3

TABLE 4

VARIATION IN VERTICAL STRESS ON SUBGRADE ALONG TRANSVERSE REFERENCE AXIS

WHEEL LOAD (KIPS)	TIRE PRESSURE (PSI)	TRANSVERSE OFFSET DISTANCE (IN.) AND CORRESPONDING VERTICAL STRESS (PSI)											
		NORTH						℄					SOUTH
7.70	30	26	22	17	11	5	1		7	13	18	24	
		1.1	1.8	2.5	3.9	5.2	5.3	—	3.9	2.3	1.2	0.5	—
7.70	30	26	24	14	10	5	3		10	12	24	25	
		1.0	1.3	2.5	3.2	4.6	4.6	—	2.5	2.3	0.7	0.4	—
10.70	30	29	25	19	15	6	1		5	7	11	17	21
		1.4	1.8	3.2	4.0	5.7	6.0	—	5.3	5.1	4.2	1.8	1.8
10.70	30	32	22	19	12	9	3		5	7	12	19	24
		1.4	2.5	2.9	3.9	4.3	5.3	—	3.9	3.7	3.4	2.1	1.8
14.00	30	28	25	24	17	14	1	0	2	11	14	19	23
		2.3	2.9	3.2	5.0	5.5	8.2	7.8	7.7	5.5	4.3	2.5	1.8
14.00	30		29	24	20	12	4	0	12	14	23		
		—	2.1	2.8	4.3	6.6	8.9	8.9	5.0	4.6	1.9	—	—
16.45	30	26	22	17	11	5	1		7	13	18	24	
		2.3	3.7	5.3	7.8	10.0	10.0	—	8.0	5.0	2.8	1.6	—
16.45	30	26	24	14	10	5	3		10	12	24		
		2.6	3.2	6.4	7.0	9.3	8.2	—	6.6	5.9	2.0	—	—
7.70	45	32	22	17	13	5	2		1	9	14	23	24
		0.6	1.7	2.8	3.6	5.2	5.2	—	5.2	3.6	2.0	0.7	0.7
7.70	45	26	23	13	11	7	1		13	14	16	21	22
		0.9	1.2	2.8	2.8	4.6	4.6	—	2.0	2.0	1.6	0.9	0.7
22.80	30	29	25	19	15	6	1		5	7	11	17	21
		3.2	4.3	6.0	7.6	11.0	11.4	—	10.3	10.0	8.9	6.4	4.3
26.25	30	33	24	19	16	7	5	0	5	10	12	20	21
		3.0	4.9	6.2	7.3	10.5	11.0	11.5	11.2	10.5	9.4	6.2	5.7
26.25	30	32	29	18	15	6	1		5	8	8	16	23
		2.8	3.6	5.3	6.6	7.8	9.6	—	8.7	8.0	7.8	5.9	3.7
17.50	45	29	21	15	12	5	1		5	9	11	20	24
		2.5	4.1	5.3	6.0	7.5	7.8	—	7.5	6.4	6.2	3.3	2.7
17.50	45	32	20	15	12	6	4		3	7	19	23	
		1.8	3.9	4.1	5.7	6.2	6.8	—	7.8	6.2	3.2	2.5	—
19.80	45	27	25	24	15	12	5		2	9	12	21	24
		3.9	4.5	3.6	8.4	9.8	11.7	—	11.7	8.9	7.7	3.0	2.5
19.80	45		28	22	12	10	3		1	9	11	21	23
		—	2.3	4.1	6.4	7.9	9.8	—	9.6	7.5	6.7	2.9	2.5
22.80	45	29	22	17	12	6	1		3	10	14	19	20
		2.5	4.3	6.0	8.6	10.7	11.4	—	11.9	8.7	7.6	5.2	5.0
22.80	45	32	24	17	13	4	1		7	10	17	22	
		2.3	3.9	7.0	8.6	12.1	12.1	—	11.0	10.0	6.8	4.4	—
26.25	45	29	21	15	12	5	1		5	9	20	22	24
		3.7	5.7	7.3	8.4	10.7	11.4	—	11.0	10.3	5.7	4.8	3.9
26.25	45	32	27	15	14	12	4		3	7	19	23	24
		2.7	4.1	6.8	6.4	8.4	8.6	—	10.7	10.5	6.9	4.8	4.6
10.70	45	36	24	17	13	4	1		7	10	17	22	
		1.4	1.8	3.6	4.8	6.0	6.8	—	5.7	4.8	2.7	1.6	—
10.70	45	29	22	17	12	6	1		3	10	14	19	20
		1.1	2.0	3.6	3.6	5.5	5.7	—	5.9	4.8	3.2	1.8	1.8
14.00	45		28	22	12	10	3		1	9	11	21	23
		—	2.0	3.1	5.4	6.4	8.0	—	7.3	5.7	5.0	1.8	1.6
14.00	45		27	24	15	12	5		2	9	12	21	24
		—	2.5	2.9	6.2	7.1	9.4	—	9.1	6.8	5.7	2.1	1.6
16.45	45	32	22	17	13	5	2		9	14	23		
		1.7	3.8	5.8	6.2	9.6	10.3	—	7.1	4.5	1.8	—	—
16.45	45		26	23	11	7	1		13	14	16	21	22
		—	2.3	3.2	6.6	7.7	8.7	—	5.0	4.3	3.9	2.3	2.3
17.50	30	33	24	19	16	7	5	0	5	10	12	20	21
		2.1	3.2	4.6	5.2	7.1	7.8	8.0	7.5	6.6	5.7	3.6	3.4
17.50	30	32	29	18	15	6	1		5	8	16	20	23
		2.1	2.5	4.3	4.8	6.6	7.0	—	6.4	5.7	3.7	2.9	2.5
19.80	30		29	24	20	12	4	0	12	14	23		
		—	3.2	3.6	5.7	8.5	11.4	11.4	6.2	5.9	2.7	—	—
19.80	30		28	24	17	14	1	0	2	11	14	19	23
		—	2.8	3.6	5.5	6.6	10.0	10.0	9.6	7.1	5.7	3.7	2.5
22.80	30	32	22	19	12	9	3		5	12	14	19	24
		2.9	4.6	6.6	6.4	8.9	10.3	—	9.2	8.6	6.2	5.7	4.3
20.45	45	28	23	17	11	4	2	0	6	11	12	19	24
		4.3	5.5	8.4	10.0	12.8	13.0	13.8	11.0	8.2	7.8	3.9	2.3

TABLE 4—*Continued*

WHEEL LOAD (KIPS)	TIRE PRESSURE (PSI)	TRANSVERSE OFFSET DISTANCE (IN.) AND CORRESPONDING VERTICAL STRESS (PSI) NORTH						℄					SOUTH
20.45	45	31	27	22	16	11	3		1	7	13	18	24
		3.4	4.3	5.9	7.8	9.6	11.0	—	10.7	8.7	6.1	4.1	2.9
23.50	45	31	27	22	16	11	3		1	7	13	18	24
		3.2	4.3	5.5	7.5	8.9	11.0	—	10.1	8.9	6.2	3.9	3.2
23.50	45	28	26	22	16	11	2	0	6	11	19	21	24
		5.2	4.8	6.9	8.6	11.4	13.5	13.5	11.0	8.9	4.3	2.6	2.7
27.50	45	33	28	24	16	12	5	0	6	11	17	19	24
		2.7	4.3	4.5	6.6	8.0	10.0	10.9	10.0	8.4	5.2	5.2	3.1
27.50	45	32	25	21	13	8	2		5	8	12	17	23
		3.6	5.0	6.2	9.1	11.0	11.9	—	10.7	10.0	8.6	6.0	3.7
33.75	45	30	25	21	13	8	2		5	8	12	17	21
		4.6	7.8	9.1	11.0	13.4	14.6	—	12.8	11.9	11.0	8.2	5.0
33.75	45	34	33	24	16	12	5	0	3	6	16	19	24
		2.9	3.9	4.3	8.9	10.0	13.0	13.4	12.4	14.6	6.9	5.9	3.6

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. In the present study an attempt has been made 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,

TABLE 5

MEASURED PAVEMENT DEFLECTIONS

AXLE LOAD | DESIGN SECTION | LVDT READING AT DESIGNATED DISTANCE FROM REFERENCE POINT

AXLE LOAD	DESIGN SECTION	LONGITUDINAL DISTANCE (FT) 6	5	4	3	2	1	2/3	1/3	0	1/3	2/3	1	2	3	4	5	6	7
18-Kip single	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	—	—
	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	—

AXLE LOAD	DESIGN SECTION	LONGITUDINAL DISTANCE (FT) 4	3	2	1	2/3	1/3	0	1/3	2/3	1	2	1	2/3	1/3	0	1/3	2/3	1	2	3	4	5
32-Kip tandem	5-6-12	0	2	2	5	6	6	7	7	7	6	6	8	8	9	10	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
	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

AXLE LOAD	DESIGN SECTION	TRANSVERSE DISTANCE (FT) 1.0	0.82	0.57	0.45	0.32	0.25	0.12	0.07	0	0.25	0.5	0.82	1.00	1.10	1.83	2.00	2.82
18-Kip single	4-6-4	43	48	50	55	57	58	63	63	64	64	61	52	52	48	26	24	18

AXLE LOAD	DESIGN SECTION	TRANSVERSE DISTANCE (FT) 0.9	0.5	0.25	0.2	0.1	0	0.25	0.40	0.50	0.65	1.0	1.8	1.9
	5-6-4	34	36	36	36	34	36	36	28	32	30	25	10	10

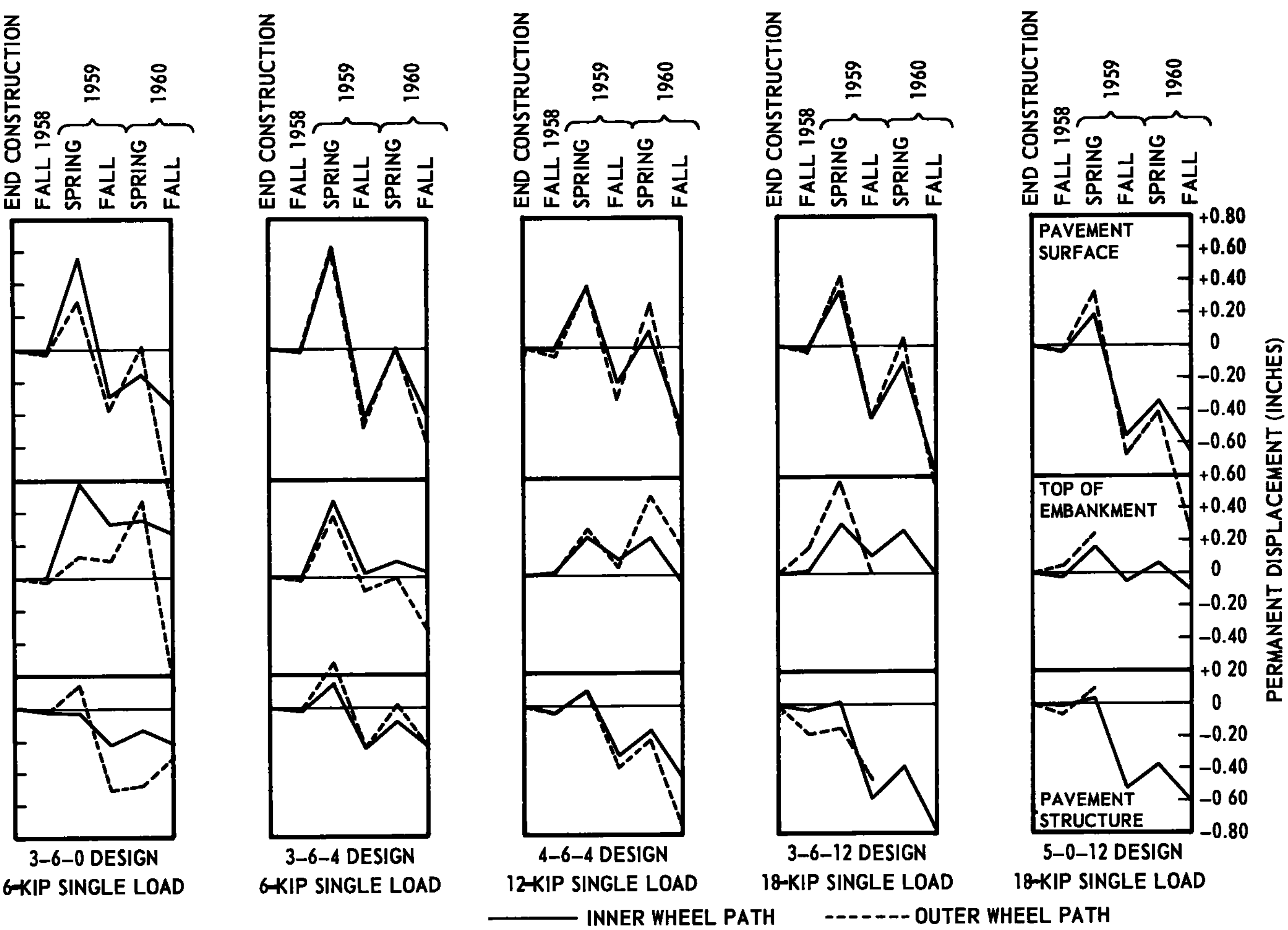

Figure 5. Permanent deflections of AASHO Road Test flexible pavements.

the change in the transverse profile of the pavement surface was determined by periodic precise level and profilometer measurements. In addition, 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 are shown 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 nonelastic 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 which the changes in density and layer thickness of each pavement component were determined. The data dealing with the changes in thickness are given in Tables 6, 7, and 8, which were taken from Ref. *27*, Report 5.

3. *Factors influencing rut depth.*—A convenient measure of nonelastic deformation is the depth of the ruts that develop in the wheelpaths. Many factors contribute to the formation of these signs of pavement distress. An attempt is made in the present study to correlate these factors with the depth of ruts developed in the AASHO Road Test. Considerable work has been done in this area and the results are presented in Ref. *27*, Report 5. Some of the findings are repeated here in order to present as complete a picture as possible. The influence of the following factors on rut depth is examined:

(a) Load repetition.—Typical plots of rut depth vs axle-load repetitions are given in Figure 7 (*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 in. (Table 9) was obtained from AASHO Road Test Data System 4199.

(c) Vertical stress on subgrade.—Results of a study of the effect of base thickness on the rut depth are available (*27*, Report 5). These data were re-analyzed so as to obtain a relationship between the vertical stress on the subgrade and rut depth (Table 10).

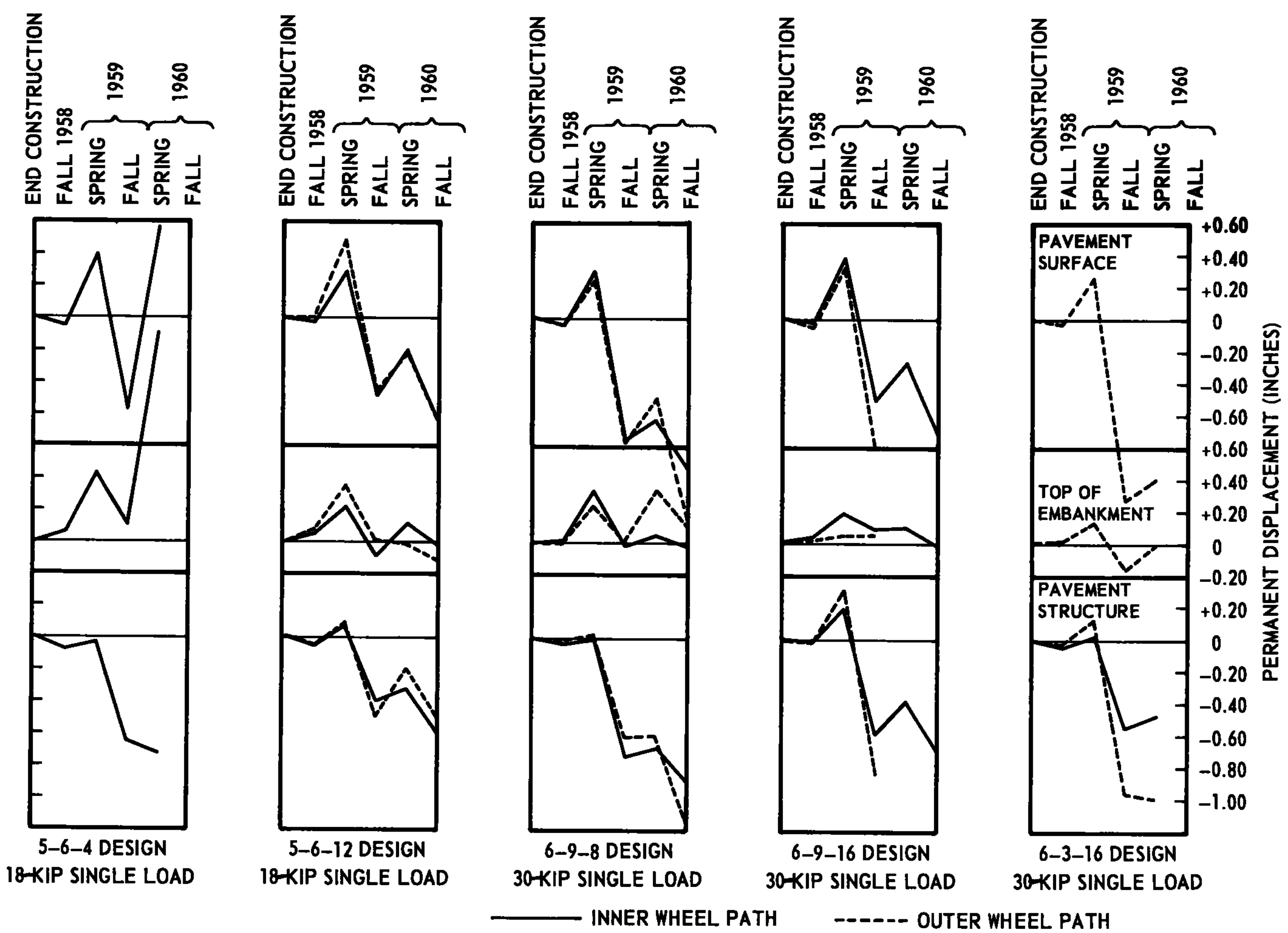

Figure 6. Permanent deflections of AASHO Road Test flexible pavements.

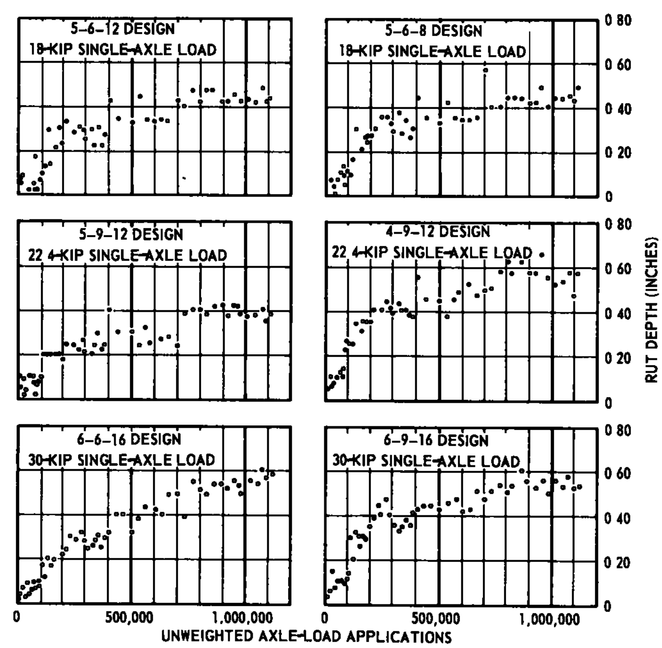

Figure 7. Effect of repetitive loading on depth of rut.

TABLE 6

CHANGES IN THICKNESS AND DENSITY, OUTER WHEELPATH, TRENCH PROGRAM, SPRING 1960

		THICKNESS (IN.)		DENSITY (PCF)		CHANGE IN THICKNESS (IN.)	
LOOP	DESIGN	INITIAL [1]	TRENCH [2]	INITIAL [3]	TRENCH [4]	TOTAL OBSERVED	DUE TO DENSIFICATION
(*a*) SURFACING							
3	4-3-8	4.04	3.81	149.3	150.8	—0.23	—0.04
	4-6-4	4.17	3.65	148.9	148.3	—0.52	+0.02
	4-6-8	3.93	4.07	149.2	149.9	+0.14	—0.02
	Mean	4.04	3.84	149.1	149.7	—0.20	—0.02
4	5-6-12	5.31	4.73	149.2	151.6	—0.58	—0.09
	5-6-8	4.87	4.94	149.1	150.9	+0.07	—0.06
	5-3-12	4.90	4.40	148.6	152.5	—0.50	—0.13
	Mean	5.03	4.69	149.0	151.7	—0.33	—0.09
5	5-9-12	5.03	4.29	150.5	150.8	—0.74	—0.01
	5-6-12	5.03	4.83	149.0	150.8	—0.20	—0.06
	5-9-8	5.06	4.53	149.3	149.6	—0.53	—0.01
	Mean	5.04	4.55	149.6	150.4	—0.49	—0.03
6	6-6-16	5.66	5.90	149.5	153.0	+0.34	—0.13
	6-9-12	5.84	5.48	149.6	150.3	—0.36	—0.03
	6-9-16	5.94	5.80	148.1	151.2	—0.14	—0.12
	Mean	5.78	5.73	149.1	151.5	—0.05	—0.09
Mean		4.97	4.70	149.2	150.8	—0.27	—0.05
(*b*) BASE							
3	4-3-8	3.32	3.13	145.0	143.4	—0.19	+0.04
	4-6-4	5.60	5.66	142.6	145.8	+0.06	—0.13
	4-6-8	5.96	5.96	143.5	149.1	0.00	—0.23
	Mean	4.96	4.92	143.7	146.1	—0.04	—0.08
4	5-6-12	6.50	6.45	140.6	146.3	—0.05	—0.26
	5-6-8	6.00	6.09	140.7	149.0	+0.09	—0.35
	5-3-12	3.10	2.87	139.9	137.2	—0.23	+0.06
	Mean	5.20	5.14	140.4	144.2	—0.06	—0.14
5	5-9-12	9.24	9.53	137.1	147.8	+0.29	—0.72
	5-6-12	6.14	6.09	143.1	140.5	—0.05	+0.11
	5-9-8	9.02	8.74	138.6	146.9	—0.28	—0.54
	Mean	8.13	8.12	139.6	145.1	—0.01	—0.32
6	6-6-16	6.16	5.77	138.1	141.5	—0.39	—0.15
	6-9-12	9.22	8.66	141.9	136.2	—0.56	+0.37
	6-9-16	8.60	8.57	140.2	141.4	—0.03	—0.07
	Mean	7.99	7.67	140.1	139.7	—0.33	+0.02
Mean		6.57	6.46	140.9	143.8	—0.11	—0.14
(*c*) SUBBASE							
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
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

LOOP	DESIGN	THICKNESS (IN.) INITIAL [1]	THICKNESS (IN.) TRENCH [2]	DENSITY (PCF) INITIAL [3]	DENSITY (PCF) TRENCH [4]	CHANGE IN THICKNESS (IN.) TOTAL OBSERVED	CHANGE IN THICKNESS (IN.) DUE TO DENSIFICATION
(*a*) SURFACING							
3	4-3-8	3.90	3.45	149.3	150.2	—0.45	—0.02
	4-6-4	3.79	3.29	148.9	151.9	—0.50	—0.08
	4-6-8	3.93	3.66	149.2	151.2	—0.27	—0.05
	Mean	3.88	3.47	149.1	151.1	—0.41	—0.05
4	5-6-12	5.31	4.94	149.2	152.5	—0.37	—0.12
	5-6-8	4.87	4.36	149.1	152.8	—0.51	—0.12
	5-3-12	4.91	4.28	148.6	150.2	—0.63	—0.05
	Mean	5.03	4.53	149.0	151.8	—0.50	—0.09
5	5-9-12	5.04	4.57	150.5	152.4	—0.47	—0.06
	5-6-12	5.03	4.54	149.0	152.1	—0.49	—0.10
	5-9-8	5.06	4.63	149.3	151.3	—0.43	—0.07
	Mean	5.04	4.58	149.6	151.9	—0.46	—0.08
6	6-6-16	5.56	5.18	149.5	152.9	—0.38	—0.13
	6-9-12	5.84	5.37	149.6	151.6	—0.47	—0.08
	6-9-16	5.94	5.57	148.1	151.7	—0.37	—0.14
	Mean	5.78	5.37	149.1	152.3	—0.41	—0.12
Mean		4.93	4.49	149.2	151.7	—0.45	—0.08
(*b*) BASE							
3	4-3-8	3.31	2.98	145.0	148.2	—0.33	—0.07
	4-6-4	5.78	5.44	142.6	146.4	—0.34	—0.15
	4-6-8	5.96	6.10	143.5	141.0	+0.14	+0.10
	Mean	5.02	4.84	143.7	145.2	—0.18	—0.05
4	5-6-12	6.44	6.12	140.6	145.0	—0.32	—0.20
	5-6-8	6.00	5.86	140.7	133.6	—0.14	+0.30
	5-3-12	3.06	3.18	139.9	142.0	+0.12	—0.05
	Mean	5.17	5.05	140.4	140.2	—0.12	+0.01
5	5-9-12	9.20	8.80	137.1	141.9	—0.40	—0.32
	5-6-12	5.86	5.64	143.1	144.9	—0.22	—0.07
	5-9-8	8.89	8.73	138.6	140.2	—0.16	—0.10
	Mean	7.98	7.72	139.6	142.3	—0.26	—0.15
6	6-6-16	6.16	6.02	138.1	142.2	—0.14	—0.18
	6-9-12	9.16	8.44	141.9	142.0	—0.72	—0.01
	6-9-16	8.60	8.42	140.2	141.9	—0.18	—0.10
	Mean	7.97	7.63	140.1	142.1	—0.34	—0.11
Mean		6.54	6.31	140.9	142.4	—0.23	—0.07
(*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
Mean		10.58	10.01	134.9	135.2	—0.56	—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 8

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

LOOP	DESIGN	THICKNESS (IN.)		DENSITY (PCF)		CHANGE IN THICKNESS (IN.)	
		INITIAL [1]	TRENCH [2]	INITIAL [3]	TRENCH [4]	TOTAL OBSERVED	DUE TO DENSIFICATION
(a) SURFACING							
3	3-3-8	2.99	2.64	149.0	152.7	—0.35	—0.07
4	4-3-12	3.90	3.48	148.5	151.9	—0.42	—0.09
	4-6-8	4.03	3.54	148.8	152.3	—0.49	—0.09
	3-6-12	2.72	2.49	149.9	152.9	—0.23	—0.05
	5-3-8	4.81	4.35	149.5	151.3	—0.46	—0.06
	Mean	3.86	3.46	149.2	152.1	—0.40	—0.07
5	3-6-8	2.93	2.61	148.4	151.0	—0.32	—0.05
	3-6-12	2.68	2.42	148.1	152.3	—0.26	—0.08
	3-9-8	3.03	2.76	148.0	151.1	—0.27	—0.06
	3-9-12	3.10	2.70	149.3	153.4	—0.40	—0.09
	4-6-8	3.97	3.51	146.4	150.1	—0.46	—0.10
	4-6-12	3.97	3.30	147.6	152.4	—0.67	—0.13
	4-9-8	4.03	3.64	148.5	152.1	—0.39	—0.10
	4-9-12	4.03	3.45	148.6	153.6	—0.58	—0.14
	5-6-4	4.87	4.47	148.0	150.4	—0.40	—0.08
	5-6-8	5.09	4.91	150.1	151.6	—0.19	—0.05
	Mean	3.77	3.38	148.3	151.8	—0.39	—0.09
6	4-3-16	3.70	3.12	144.9	151.2	—0.58	—0.16
	4-6-12	3.61	3.12	148.5	150.6	—0.49	—0.05
	4-6-16	3.62	3.42	149.2	152.5	—0.20	—0.08
	4-9-12	3.85	3.33	149.9	153.4	—0.52	—0.09
	4-9-16	3.75	3.49	148.9	153.2	—0.26	—0.11
	5-6-12	4.90	4.11	150.3	152.7	—0.79	—0.08
	5-6-16	4.87	4.60	149.1	152.7	—0.27	—0.12
	5-9-12	4.78	4.16	150.2	153.1	—0.62	—0.09
	5-9-16	4.65	4.36	151.4	153.9	—0.29	—0.08
	6-3-12	5.81	5.16	148.0	154.0	—0.65	—0.24
	6-6-8	5.78	5.36	149.7	152.5	—0.42	—0.11
	6-6-12	5.72	5.52	149.4	154.4	—0.20	—0.19
	Mean	4.59	4.15	149.1	152.9	—0.44	—0.12
Mean		4.12	3.70	148.8	152.3	—0.41	—0.10
(b) BASE							
3	3-3-8	3.56	3.54	142.0	No trench densities taken	—0.02	
4	4-3-12	3.24	3.42	143.9		+0.18	
	4-6-8	6.44	6.18	144.1		—0.26	
	3-6-12	5.72	5.82	143.9		+0.10	
	5-3-8	3.22	2.82	143.7		—0.40	
	Mean	4.66	4.56	143.9		—0.10	
5	3-6-8	6.40	6.36	144.6		—0.04	
	3-6-12	6.14	5.63	141.9		—0.51	
	3-9-8	9.14	9.20	143.2		+0.06	
	3-9-12	9.06	8.80	141.7		—0.26	
	4-6-8	6.74	6.31	138.1		—0.43	
	4-6-12	6.16	6.26	139.6		+0.10	
	4-9-8	9.11	9.16	145.0		+0.05	
	4-9-12	8.66	8.26	142.5		—0.40	
	5-6-4	6.07	6.25	140.6		+0.18	
	5-6-8	5.91	5.92	141.9		+0.01	
	Mean	7.34	7.22	141.9		—0.12	
6	4-3-16	2.52	2.48	146.9		—0.04	
	4-6-12	6.18	5.85	141.7		—0.33	
	4-6-16	6.12	5.88	141.0		—0.24	
	4-9-12	9.04	8.37	141.0		—0.67	
	4-9-16	9.24	8.70	141.6		—0.54	
	5-6-12	6.01	5.34	138.1		—0.67	
	5-6-16	6.04	5.46	138.8		—0.58	
	5-9-12	8.82	8.83	139.0		+0.01	
	5-9-16	8.98	8.42	138.8		—0.56	
	6-3-12	2.77	2.67	141.3		—0.10	
	6-6-8	5.94	5.55	142.5		—0.39	
	6-6-12	5.70	5.28	141.0		—0.42	
	Mean	6.45	6.07	141.0		—0.38	
Mean		5.50	5.35	141.8		—0.16	

TABLE 8—*Continued*

LOOP	DESIGN	THICKNESS (IN.) INITIAL [1]	THICKNESS (IN.) TRENCH [2]	DENSITY (PCF) INITIAL [3]	DENSITY (PCF) TRENCH [4]	CHANGE IN THICKNESS (IN.) TOTAL OBSERVED	CHANGE IN THICKNESS (IN.) DUE TO DENSIFICATION
(*c*) SUBBASE							
3	3-3-8	7.52	6.99	131.0	No trench densities taken	—0.53	
4	4-3-12	11.73	11.37	135.7		—0.36	
	4-6-8	7.94	7.80	136.7		—0.14	
	3-6-12	12.20	11.70	137.4		—0.50	
	5-3-8	7.97	8.37	138.2		+0.40	
	Mean	9.96	9.81	137.0		—0.15	
5	3-6-8	7.88	7.20	135.8		—0.68	
	3-6-12	12.54	11.72	138.7		—0.82	
	3-9-8	7.71	7.48	131.3		—0.23	
	3-9-12	12.03	10.88	133.7		—1.15	
	4-6-8	7.90	7.30	137.6		—0.60	
	4-6-12	11.86	11.55	133.7		—0.31	
	4-9-8	7.86	6.57	136.3		—1.29	
	4-9-12	12.24	11.87	133.9		—0.37	
	5-6-4	3.40	3.21	133.5		—0.19	
	5-6-8	8.08	7.59	135.1		—0.49	
	Mean	9.15	8.54	135.0		—0.61	
6	4-3-16	16.28	15.47	—		—0.81	
	4-6-12	11.36	11.40	139.4		+0.04	
	4-6-16	15.92	15.10	135.9		—0.82	
	4-9-12	12.00	11.31	132.2		—0.69	
	4-9-16	15.12	14.82	137.6		—0.30	
	5-6-12	11.81	10.94	134.6		—0.87	
	5-6-16	15.56	15.59	136.7		+0.03	
	5-9-12	12.08	11.43	138.6		—0.65	
	5-9-16	15.38	15.07	129.3		—0.31	
	6-3-12	12.29	11.74	135.2		—0.55	
	6-6-8	7.76	7.67	139.5		—0.09	
	6-6-12	12.24	12.01	139.1		—0.23	
	Mean	13.15	12.71	136.2		—0.44	
Mean		9.95	9.50	135.7		—0.43	

[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

DESIGN SECTION	AXLE LOAD	NUMBER OF LOAD REPETITIONS (PER CENT OF TOTAL 1,114,000) 0.25 IN.	0.50 IN.	0.75 IN.
4-6-8	12-Kip single	95	—	—
	18-Kip single	11	19	—
	22.4-Kip single	3	8	9
	30-Kip single	6	7	8
5-6-8	18-Kip single	12	80	—
	22.4-Kip single	10	12	60
	30-Kip single	7	9	10

TABLE 10

COMPARISON OF RUT DEPTH AND VERTICAL SUBGRADE STRESS

PAVEMENT STRUCTURE THICKNESS (IN.)	AXLE LOAD (KIPS) SINGLE	VERTICAL STRESS ON SUBGRADE (PSI)	RUT DEPTH (IN.) 1959	RUT DEPTH (IN.) 1960
23	30	12.0	0.52	0.73
24	30	11.2	0.44	0.58
25	30	10.5	0.34	0.45
26	30	9.8	0.32	0.44
27	30	9.0	0.32	0.44
28	30	8.2	0.32	0.44
16	18	14.2	0.51	—
17	18	13.1	0.45	—
18	18	12.0	0.35	—
19	18	10.9	0.30	0.60
20	18	9.8	0.25	0.50
21	18	9.0	0.25	0.42
22	18	8.2	0.25	0.42
11	12	18.7	0.47	—
12	12	16.5	0.42	—
13	12	14.3	0.37	—
14	12	12.8	0.32	0.65
15	12	11.6	0.27	0.49
16	12	10.1	0.25	0.39
17	12	9.2	0.23	0.29

CHAPTER THREE

DATA ON PHYSICAL PROPERTIES OF AASHO ROAD TEST MATERIALS

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

Inasmuch as 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 given in Tables 11, 12, 13, and 14 represent the averages of numerous tests conducted by the AASHO Road Test staff prior to and during construction of the test road (*27*, Report 5).

STRENGTH CHARACTERISTICS

The strength characteristics of significance for this investigation are the cohesion, c, and angle of shearing resistance, ϕ. 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 representing average values.

DEFORMATION CHARACTERISTICS

The deformation characteristics pertinent to this investigation are the modulus of deformation, E, and the Poisson's ratio, μ, of the soil. A review of published data on the modulus of deformation of the AASHO Road Test materials reveals that (a) very few such results are available and (b) there is a wide range of values within any one investigation as well as between investigations. The latter is to be expected, because 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

TABLE 11

PHYSICAL CHARACTERISTICS OF EMBANKMENT SOIL

Textural classification:	Yellow-brown silty clay (A-6)	
Grain size distribution:	*Sieve No. or Grain Size*	*Percent Finer*
	No. 4	99.0
	No. 10	96.8
	No. 40	91.0
	No. 60	87.7
	No. 200	80 6
	0.02 mm	62.8
	0.05 mm	42.3
	0.002 mm	15.3
Atterberg limits:	Liquid limit = 29.4 percent	
	Plastic limit = 16.4 percent	
Specific gravity:	2.71	
Compacted density:	115 4 pcf	
Corresponding moisture content.	15.5 percent	

TABLE 12

PHYSICAL CHARACTERISTICS OF SUBBASE MATERIAL

Textural classification:	Sand-gravel mulch	
Grain size distribution:	*Sieve No. or Grain Size*	*Percent Finer*
	1 in.	100
	¾ in.	96
	½ in.	90
	No. 4	71
	No. 10	52
	No. 40	25
	No. 200	6.5
Atterberg limits:	Non-plastic	
Specific gravity:	2.70	
Compacted density:	134.5 pcf	
Corresponding moisture content:	3.8 percent	

TABLE 13

PHYSICAL CHARACTERISTICS OF BASE COURSE

Textural classification:	Crushed dolomitic limestone	
Grain size distribution:	*Sieve No. or Grain Size*	*Percent Finer*
	1½ in.	100
	1 in.	90
	¾ in.	80
	½ in.	68
	No. 4	50
	No. 10	36
	No. 40	21
	No. 100	14.5
	No. 200	11.5
Atterberg limits:	Non-plastic	
Specific gravity:	2.74	
Compacted density:	140 pcf	
Corresponding moisture content:	4.2 percent	

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 completely 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 a modulus. Another modulus 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 briefly reviews the studies of deformation characteristics performed for the AASHO Road Test; all the significant results are summarized in Table 16:

1. Cooperative materials testing program.—As outlined in HRB Special Report 66 (*29*), several 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 materials 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 these 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 also 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 psi.

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 psi was obtained.

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

4. Ohio State University studies.—In a study of AASHO Road Test deflections at 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.

TABLE 14

PHYSICAL CHARACTERISTICS OF BINDER AND SURFACE COURSES

PROPERTY	BINDER COURSE	SURFACE COURSE
Asphalt content (%)	4.2	5.2
Stability (lb)	1,770	2,000
Flow	11.2	11.1
Voids (%)	4.8	3.6
Compacted density (pcf)	149.0	146.8
Voids as compacted (%)	7.7	6.5

TABLE 15

STRENGTH PARAMETERS OF COMPONENTS OF PAVEMENT STRUCTURE

COMPONENT	COHESION (PSI)	ANGLE OF SHEARING RESISTANCE (DEG)
Embankment soil	14	23
Subbase	5	40
Base	9	55

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 psi 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 because 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(kg/sq cm)$=$100 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 deflections 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 can be seen from Table 16, there are significant

TABLE 16

MODULUS OF DEFORMATION OF AASHO ROAD TEST PAVEMENT STRUCTURE COMPONENTS

INVESTIGATING AGENCY	BASIS FOR DETERMINATION	MODULUS OF DEFORMATION (PSI) SUBGRADE	SUBBASE	BASE	SURFACING
Kansas Highway Dept.	Triaxial compression test	1,300	8,000 [a]	10,000 [a]	—
Univ. of California	Triaxial compression test	5,500	—	—	—
Asphalt Institute	Van der Poels stiffness factor	3,000	—	15,000	150,000
Ohio State University	Triaxial compression tests	4,000	15,000		160,000
Georgia Inst. Tech.	Triaxial compression tests	1,040 [a]	—	—	—
Indirect determination	From plate load tests	2,040	—	—	—
	From CBR tests	2,800 to 5,600	51,000	>80,000	—
	From measured pavement deflections	7,000	63,000 [b]		

[a] Initial tangent modulus
[b] Applies to the entire pavement structure.

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 pavement 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 has been made in this report to make evaluations of results with the absolute values of deformation moduli as given in Table 16. Limited use has been made, however, 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 the values in Table 16, a ratio of E_1/E_2 of 10 was assumed.

CHAPTER FOUR

ANALYSIS OF TEST RESULTS

This chapter, devoted to the analysis of the results referred to in Chapter Two, is subdivided in a similar manner so as to make 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 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 analyses and investigations have been directed toward the establishment of the pattern of stress distribution in a soil due to a load at the surface. From these investigations, two basic methods of stress analysis for flexible pavements have been adopted. The first method is based on the well-known Boussinesq solution for stress distribution in a homogeneous isotropic solid; the second, on the equally well-known Burmister solution for layered solids. Because these methods in many instances yield vastly different results for identical conditions, there exists a real need for assessing the 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 the well-known basic assumptions on which the classical elasticity theory is based, the following simplifying assumptions are made in all of the stress analyses:

1. The tire-pavement surface contact pressure is assumed to be uniform and equal to the tire pressure whereever 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 the 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 the Boussinesq and Burmister theories are also shown. Here, again, there is good agreement with the Boussinesq theory.

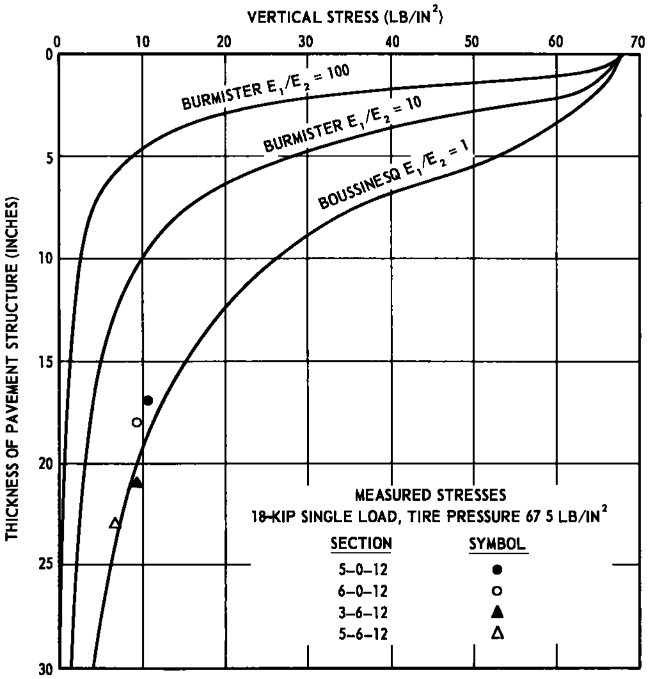

Figure 8. Effect of thickness of pavement structure on vertical subgrade stresses.

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 Figure 9 that the observed stresses are in close agreement with the theoretical stresses after Boussinesq, although the latter are generally somewhat higher. Plotted in the same figure are theoretical stresses for a two-layer solid after Burmister, for moduli ratios of 10 and 100.

Although not indicated in Figure 9, it was found that the Burmister solution corresponding to a modular ratio of 5 formed the outer limit for the observed values in question

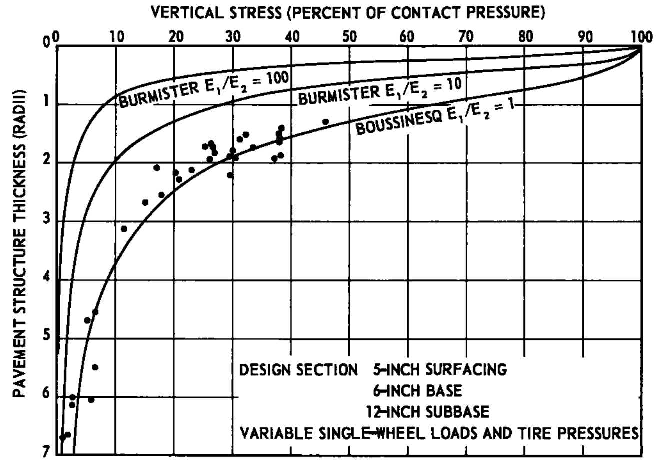

Figure 9. Vertical stresses under varying conditions of wheel load and tire pressure (single wheel and axle).

and that the layered solid solutions based on moduli ratios greater than 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. Figure 10, however, 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,

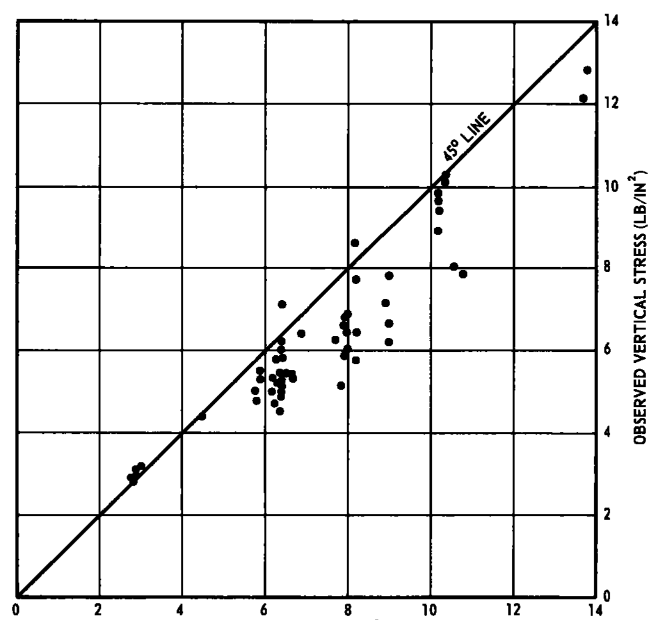

Figure 10. Vertical stresses under varying conditions of wheel load and tire pressure (multiple wheel and axle).

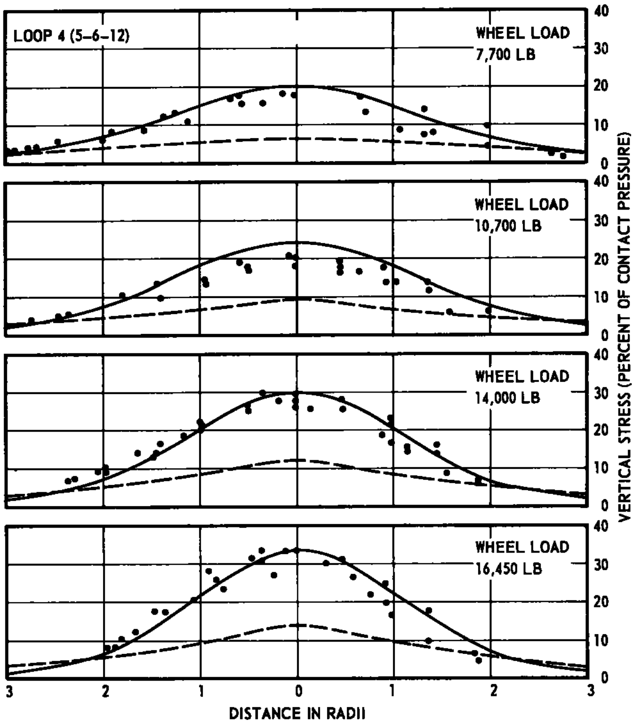

Figure 11. Observed distribution of vertical stresses; wheel loads 7,700 to 16,450 lb; tire pressure 30 psi.

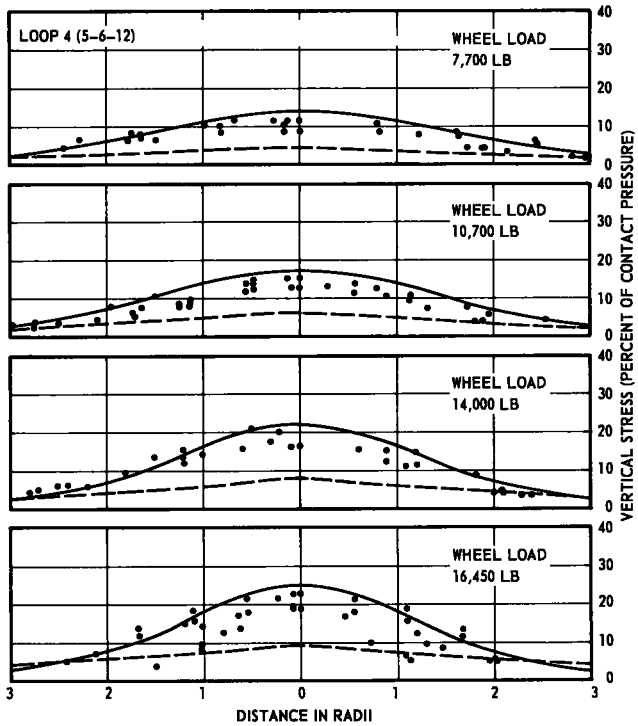

Figure 12. Observed distribution of vertical stresses; wheel loads 7,700 to 16,450 lb; tire pressure 45 psi.

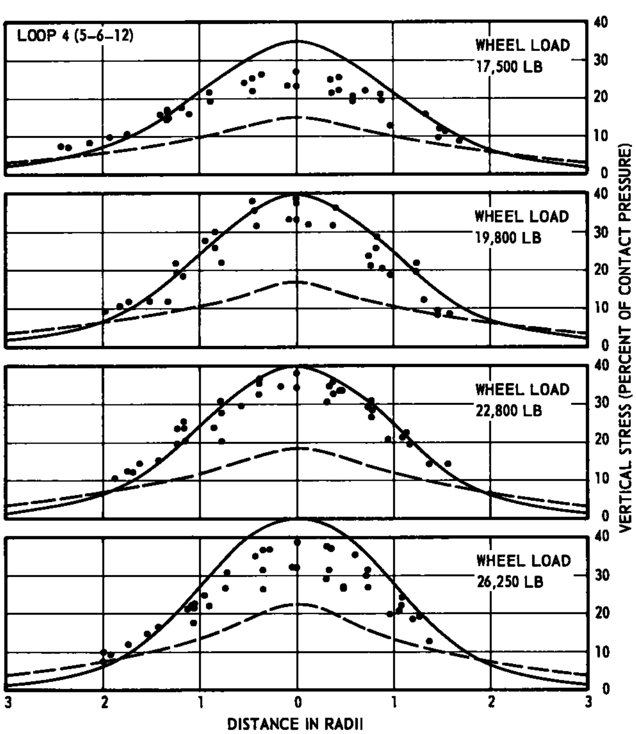

Figure 13. Observed distribution of vertical stresses; wheel loads 17,500 to 26,250 lb; tire pressure 30 psi.

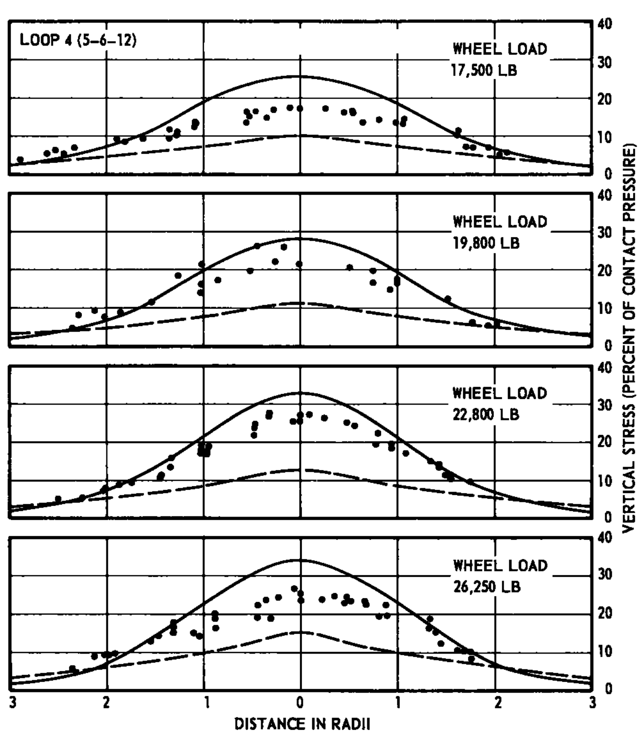

Figure 14. Observed distribution of vertical stresses; wheel loads 17,500 to 26,250 lb; tire pressure 45 psi.

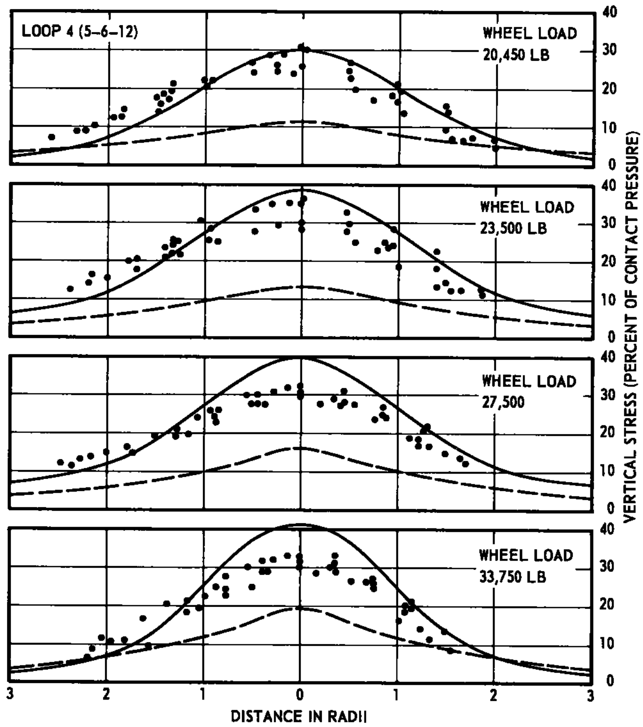

Figure 15. Observed distribution of vertical stresses; wheel loads 20,450 to 33,750 lb; tire pressure 45 psi.

but not to the extent that the use of the Boussinesq theory in these cases would be precluded. A good agreement with the layered solid theory can be obtained only if a moduli ratio of $E_1/E_2 = 2$ is selected.

To show the distribution of vertical stresses at points other than directly beneath the load, the stress contour data given in Tables 3 and 4 are 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 10 are included on each plot. 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 E_1/E_2 value.

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

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

A comparison of the temperature plot and the vertical stress plot indicates that, in general, decreasing temperatures are associated with reduced vertical stresses. In particular, the extremely low vertical stresses correspond, without 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° 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 subfreezing temperatures (*37, 38*). It may also be expected that even granular waterbound 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; hence it may possess sufficient tensile strength at intermediate temperatures to behave essentially as a slab, thus reducing the vertical stresses. The AASHO Road Test studies showed that at temperatures above 80° 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 also appears to be a slight effect from 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-time curve, to a certain degree, shows the same trend as the moisture content-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 indicate 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

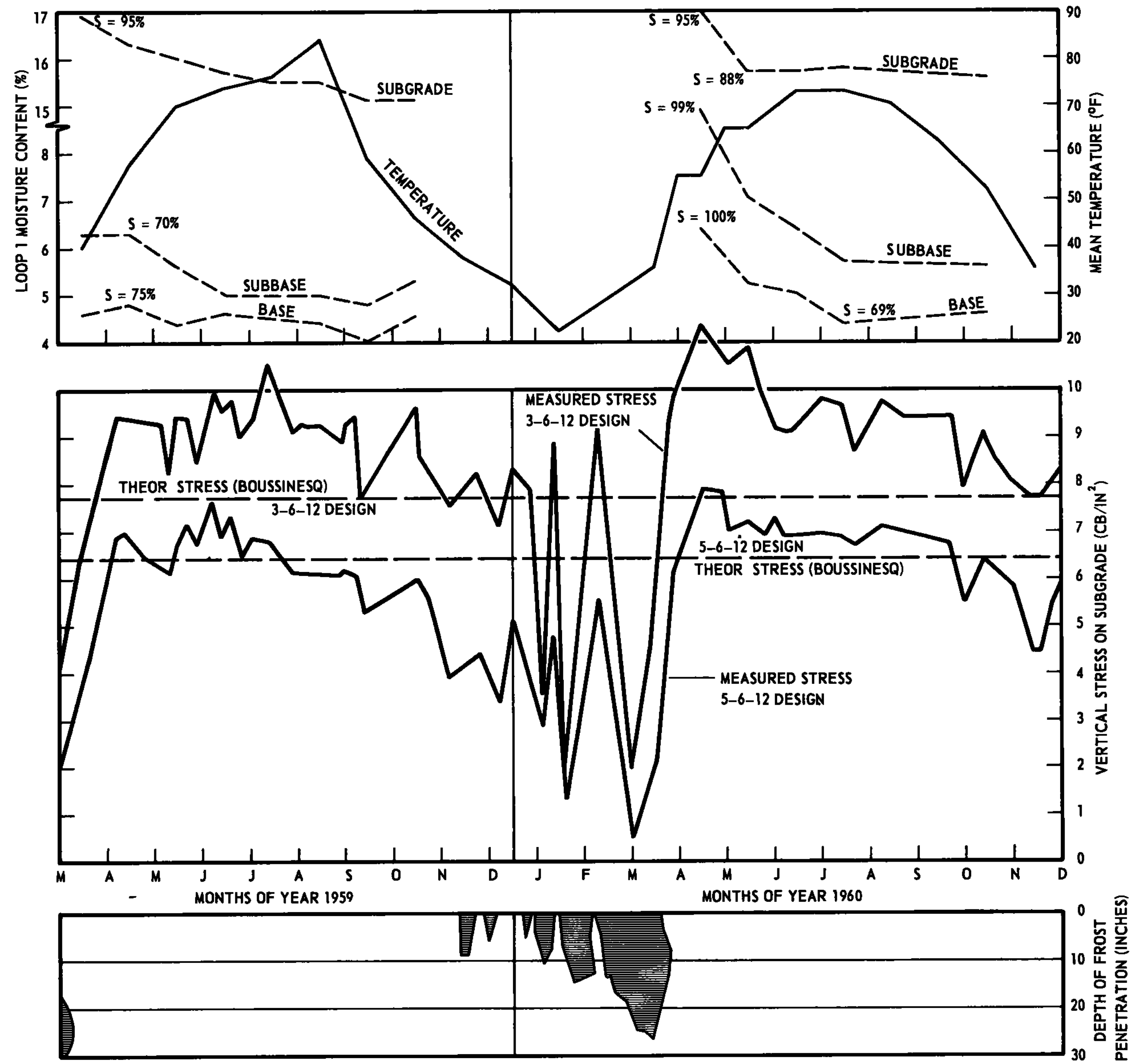

Figure 16. Variation of vertical stresses with environmental changes, AASHO Road Test (Loop 4, 18-kip single axle load).

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, varying slightly 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 recent years, deflections of flexible pavements have been the object of increased interest to highway engineers. In fact, 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 in Chapter One, they are used by some agencies in their pavement thickness design criteria.

Numerous investigations have been directed toward establishing a rigorous solution to the problem of predicting the deflection of a flexible pavement system. As explained in Chapter One, most of the solutions have the elastic theory as their basis, although several of the more recent investigations treat the flexible pavement materials as visco-elastic. To date no reliable general method of analysis has evolved that would permit a sufficiently accurate prediction of deflections, although isolated instances of ob-

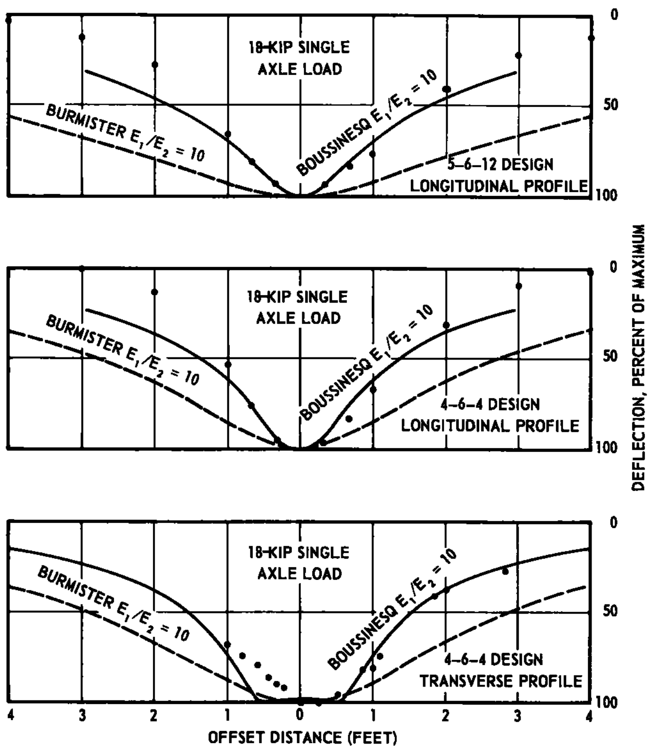

Figure 17. Measured deflection basins, sections 5-6-12 and 4-6-4; 18-kip, single axle, dual wheel load; tire pressure 67.5 psi.

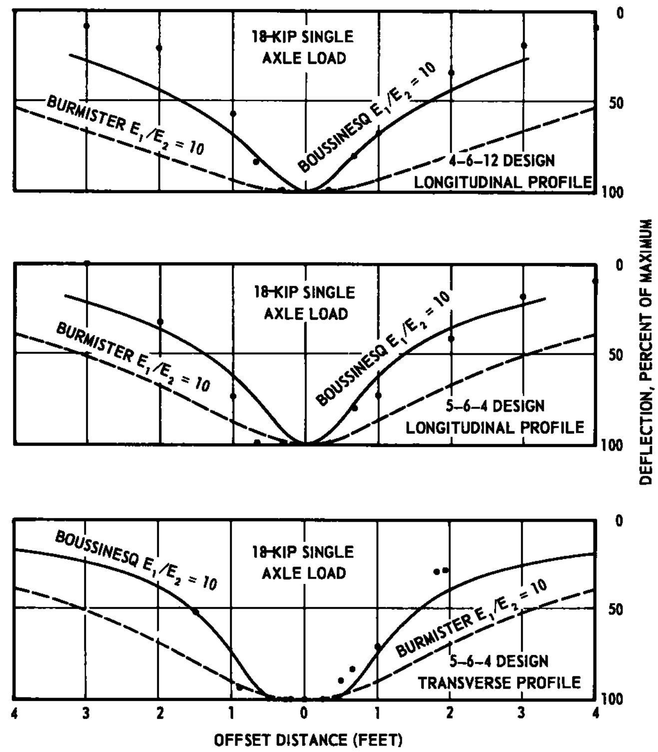

Figure 18. Measured deflection basins, sections 4-6-12 and 5-6-4; 18-kip, single axle, dual wheel load; tire pressure 67.5 psi.

served deflections have been successfully analyzed on a theoretical basis.

This investigation has been 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 was 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

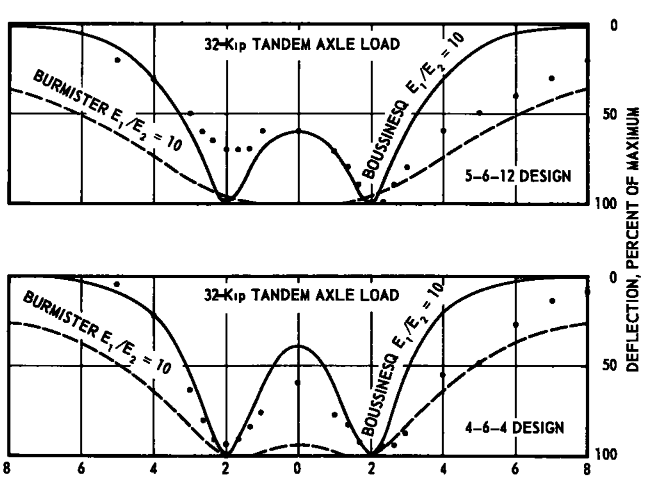

Figure 19. Measured deflection basins, sections 5-6-12 and 4-6-4; 32-kip, tandem axle, dual wheel load; tire pressure 69.5 psi.

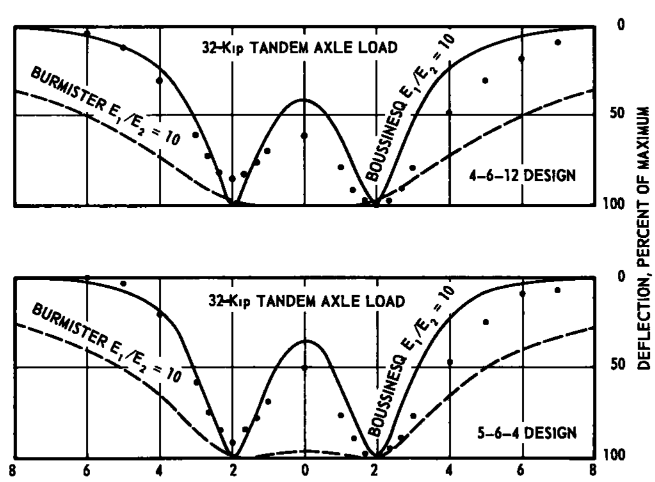

Figure 20. Measured deflection basins, sections 4-6-12 and 5-6-4; 32-kip, tandem axle, dual wheel load; tire pressure 69.5 psi.

TABLE 17
INFLUENCE FACTOR FOR SETTLEMENT, UNIFORMLY LOADED CIRCULAR AREA

	x/a												
z/a	0	0.2	0.4	0.6	0.8	1.0	1.2	1.5	2.0	3.0	4.0	5.0	6.0
(*a*) $\mu=0.5$													
0	1.000	0.990	0.958	0.903	0.813	0.637	0.468	0.356	0.258	0.169	0.126	0.100	0.083
0.1	0.995	0.985	0.953	0.896	0.803	0.634	0.472	0.357	0.259	0.169	0.126	0.100	—
0.2	0.981	0.970	0.937	0.877	0.780	0.627	0.481	0.361	0.260	0.170	0.126	0.100	0.083
0.3	0.958	0.947	0.910	0.850	0.752	0.617	0.488	0.366	0.262	0.170	0.127	—	—
0.4	0.923	0.917	0.882	0.819	0.725	0.605	0.490	0.370	0.264	—	—	—	—
0.5	0.894	0.883	0.847	0.786	0.699	0.592	0.490	0.375	0.267	0.171	0.127	0.101	0.084
0.6	0.858	0.846	0.813	0.758	0.673	0.579	0.486	0.377	—	—	—	—	—
0.7	0.820	0.808	0.776	0.703	0.648	0.542	0.481	0.378	0.273	—	—	—	—
0.8	0.781	0.770	0.740	0.690	0.624	0.548	0.474	0.374	—	—	—	—	—
0.9	0.793	0.733	0.705	0.662	0.604	0.532	0.466	0.378	—	—	—	—	—
1.0	0.707	0.713	0.672	0.631	0.578	0.518	0.457	0.375	0.278	0.176	0.130	0.097	0.085
1.2	0.640	0.633	0.612	0.569	0.535	0.487	0.438	0.368	0.279	0.179	0.130	0.102	0.086
1.5	0.555	0.550	0.525	0.510	0.479	0.445	0.407	0.352	0.276	0.182	0.132	0.103	0.086
2.0	0.448	0.445	0.425	0.420	0.403	0.382	0.359	0.322	0.265	0.183	0.136	0.106	0.087
2.5	0.373	0.370	0.363	0.356	0.345	0.331	0.315	0.291	0.253	0.180	0.136	0.106	0.089
3.0	0.315	0.315	0.311	0.306	0.298	0.290	0.278	0.261	0.233	0.177	0.136	0.110	0.089
4.0	0.243	0.243	0.240	0.238	0.233	0.228	0.222	0.216	0.199	0.164	0.132	0.110	0.089
5.0	0.194	0.194	—	—	—	0.187	—	—	0.172	0.145	0.123	0.106	0.089
6.0	0.167	—	—	—	—	0.160	—	—	0.151	0.134	0.117	0.100	0.089
(*b*) $\mu=0.33$													
0	1.000	0.990	0.958	0.903	0.813	0.637	0.468	0.356	0.258	0.169	0.126	0.100	0.083
0.1	0.972	0.962	0.930	0.874	0.783	0.623	0.470	0.356	0.259	0.169	0.126	0.100	—
0.2	0.940	0.929	0.897	0.840	0.747	0.607	0.473	0.358	0.259	0.170	0.126	0.100	0.083
0.3	0.904	0.893	0.858	0.802	0.712	0.591	0.474	0.360	0.260	0.170	0.127	—	—
0.4	0.859	0.854	0.822	0.764	0.680	0.573	0.471	0.361	0.261	—	—	—	—
0.5	0.824	0.814	0.781	0.727	0.650	0.556	0.466	0.363	0.262	0.170	0.126	0.101	0.084
0.6	0.784	0.773	0.744	0.696	0.622	0.540	0.459	0.362	—	—	—	—	—
0.7	0.744	0.734	0.706	0.640	0.595	0.506	0.451	0.360	0.265	—	—	—	—
0.8	0.705	0.695	0.669	0.627	0.570	0.505	0.441	0.353	—	—	—	—	—
0.9	0.717	0.659	0.635	0.599	0.550	0.488	0.431	0.355	—	—	—	—	—
1.0	0.633	0.640	0.603	0.568	0.523	0.473	0.421	0.350	0.265	0.172	0.128	0.109	0.084
1.2	0.569	0.563	0.546	0.508	0.481	0.441	0.400	0.340	0.263	0.173	0.127	0.100	0.085
1.5	0.491	0.487	0.464	0.454	0.428	0.400	0.368	0.322	0.256	0.174	0.128	0.101	0.085
2.0	0.394	0.392	0.374	0.371	0.357	0.340	0.321	0.290	0.242	0.172	0.130	0.102	0.085
2.5	0.327	0.325	0.319	0.313	0.304	0.293	0.280	0.260	0.228	0.167	0.128	0.102	0.086
3.0	0.276	0.276	0.273	0.269	0.262	0.256	0.246	0.232	0.209	0.162	0.127	0.104	0.086
4.0	0.212	0.212	0.210	0.210	0.205	0.201	0.196	0.191	0.177	0.148	0.121	0.102	0.084
5.0	0.170	0.170	—	—	—	0.164	—	—	0.152	0.130	0.112	0.097	0.083
6.0	0.146	—	—	—	—	0.140	—	—	0.133	0.119	0.105	0.091	0.081
(*c*) $\mu=0.25$													
0	1.000	0.990	0.958	0.903	0.813	0.637	0.468	0.356	0.258	0.169	0.126	0.100	0.083
0.1	0.965	0.955	0.923	0.867	0.777	0.620	0.469	0.356	0.259	0.169	0.126	0.100	—
0.2	0.927	0.916	0.885	0.828	0.737	0.601	0.470	0.357	0.259	0.170	0.126	0.100	0.083
0.3	0.887	0.876	0.842	0.787	0.700	0.583	0.470	0.358	0.260	0.170	0.126	—	—
0.4	0.839	0.834	0.803	0.747	0.666	0.563	0.465	0.358	0.260	—	—	—	—
0.5	0.805	0.792	0.767	0.708	0.635	0.545	0.459	0.359	0.261	0.170	0.126	0.101	0.084
0.6	0.761	0.750	0.722	0.677	0.606	0.528	0.450	0.357	—	—	—	—	—
0.7	0.720	0.711	0.684	0.620	0.578	0.495	0.441	0.354	0.262	—	—	—	—
0.8	0.681	0.672	0.647	0.607	0.553	0.491	0.431	0.346	—	—	—	—	—
0.9	0.693	0.636	0.613	0.579	0.533	0.474	0.415	0.348	—	—	—	—	—
1.0	0.610	0.617	0.581	0.548	0.506	0.459	0.410	0.342	0.261	0.171	0.128	0.101	0.084
1.2	0.547	0.541	0.525	0.489	0.464	0.427	0.388	0.331	0.258	0.181	0.126	0.100	0.085
1.5	0.471	0.467	0.445	0.436	0.412	0.386	0.356	0.312	0.250	0.171	0.127	0.101	0.085
2.0	0.377	0.375	0.358	0.356	0.343	0.327	0.309	0.280	0.235	0.168	0.128	0.101	0.084
2.5	0.313	0.311	0.305	0.300	0.291	0.281	0.269	0.250	0.220	0.163	0.126	0.101	0.085
3.0	0.264	0.264	0.261	0.257	0.251	0.245	0.236	0.223	0.201	0.157	0.124	0.102	0.085
4.0	0.202	0.202	0.201	0.201	0.196	0.192	0.188	0.183	0.170	0.143	0.117	0.099	0.082
5.0	0.162	0.162	—	—	—	0.157	—	—	0.146	0.135	0.108	0.094	0.081
6.0	0.139	—	—	—	—	0.134	—	—	0.127	0.114	0.101	0.088	0.079

TABLE 17—*Continued*

z/a	x/a 0	0.2	0.4	0.6	0.8	1.0	1.2	1.5	2.0	3.0	4.0	5.0	6.0
(*d*) $\mu = 0.0$													
0	1.000	0.990	0.958	0.903	0.813	0.637	0.468	0.356	0.258	0.169	0.126	0.100	0.083
0.1	0.950	0.940	0.908	0.853	0.764	0.613	0.467	0.356	0.259	0.169	0.126	0.100	0.083
0.2	0.900	0.890	0.859	0.804	0.716	0.588	0.486	0.356	0.259	0.170	0.126	0.100	0.083
0.3	0.851	0.841	0.808	0.756	0.674	0.571	0.461	0.355	0.259	0.169	0.126	—	—
0.4	0.798	0.793	0.764	0.712	0.637	0.543	0.453	0.353	0.258	—	—	—	—
0.5	0.756	0.747	0.718	0.670	0.603	0.526	0.444	0.351	0.258	0.169	0.126	0.101	0.083
0.6	0.712	0.703	0.678	0.637	0.572	0.502	0.432	0.347	—	—	—	—	—
0.7	0.677	0.662	0.638	0.579	0.544	0.471	0.421	0.343	0.257	—	—	—	—
0.8	0.631	0.623	0.601	0.565	0.517	0.463	0.409	0.333	—	—	—	—	—
0.9	0.644	0.587	0.567	0.538	0.497	0.444	0.397	0.333	—	—	—	—	—
1.0	0.561	0.569	0.537	0.508	0.471	0.428	0.386	0.326	0.252	0.168	0.126	0.100	0.084
1.2	0.501	0.496	0.482	0.449	0.430	0.398	0.363	0.313	0.247	0.168	0.125	0.099	0.084
1.5	0.429	0.426	0.406	0.399	0.379	0.355	0.330	0.292	0.238	0.166	0.125	0.099	0.083
2.0	0.342	0.340	0.324	0.324	0.313	0.299	0.284	0.259	0.220	0.161	0.124	0.100	0.083
2.5	0.283	0.281	0.277	0.272	0.265	0.256	0.246	0.230	0.204	0.154	0.121	0.097	0.083
3.0	0.239	0.238	0.236	0.233	0.229	0.223	0.215	0.204	0.185	0.147	0.118	0.098	0.083
4.0	0.183	0.183	0.182	0.182	0.177	0.174	0.170	0.166	0.155	0.132	0.110	0.094	0.079
5.0	0.147	0.147	—	—	—	0.142	—	—	0.132	0.115	0.101	0.088	0.077
6.0	0.125	—	—	—	—	0.121	—	—	0.115	0.104	0.093	0.082	0.074

percentage of the maximum) are used rather than absolute values. In this way, discrepancies directly attributable to the use of inaccurate or nonrepresentative 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 μ) of the component layers. They were computed as outlined in Fig. 7 of Ref. *16*. 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. These factors, which may be very useful for similar computations, are given in Table 17.

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.

As can be seen from Figures 17-20, 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 by the layered solid theory.

This finding is 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 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. Because, 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 ft 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-psi loads, followed by 75 repetitions of a constant unit load of 80 psi. The tests were performed using rigid plates with diameters of 6 to 42 in.

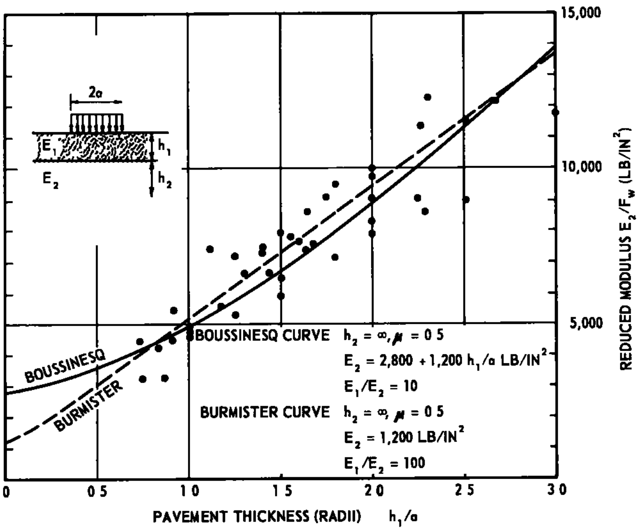

Figure 21. Interpretation of Hybla Valley plate load test data.

In Figure 21 the reduced modulus, which incorporates the measured elastic deflection of the plate, the plate diameter, and the unit load acting on the plate, is plotted against the pavement thickness expressed as a multiple of the radius of the plate. Each point represents the results of a single test. This form of data representation was used to permit a generalization of the problem and to facilitate the analysis of the data.

Assuming a two-layer solid, curves were fitted to the points on the basis of the Boussinesq and the Burmister theories of stress distribution. This analysis demonstrated that either theory can yield an interpretation. The acceptability of the interpretation can be judged, however, only by examination of the values of the moduli used in the analysis (in this case as shown in Fig. 21).

The acceptability of one interpretation relative to the other is open to question, but it appears that the Boussinesq theory offers a more reasonable interpretation for the following reasons:

1. A moduli ratio of $E_1/E_2 = 10$ is more reasonable for a flexible pavement system than is a ratio of 100.
2. 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 psi. 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 Two, the primary purpose of this phase of the investigation was 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 is placed on the change in the transverse profile, because it is from these data that information regarding plastic shear deformation and consolidation can be derived. The following deals with the data as categorized and presented in Chapter Two.

Permanent surface deflection appears in the form of rutting within the wheelpaths. Of particular interest are the two basic questions of the mechanics of rutting: that is, first, which component or components of the pavement system contribute the most to rutting and, second, 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, which reported (*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 wheelpaths of the pavement surface." This is confirmed by Figure 22, which is based on Figures 5 and 6. The test data in this figure apply 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 depths were as large as 1.3 in. Thus, on the basis of AASHO Road Test data, it may be surmised that for

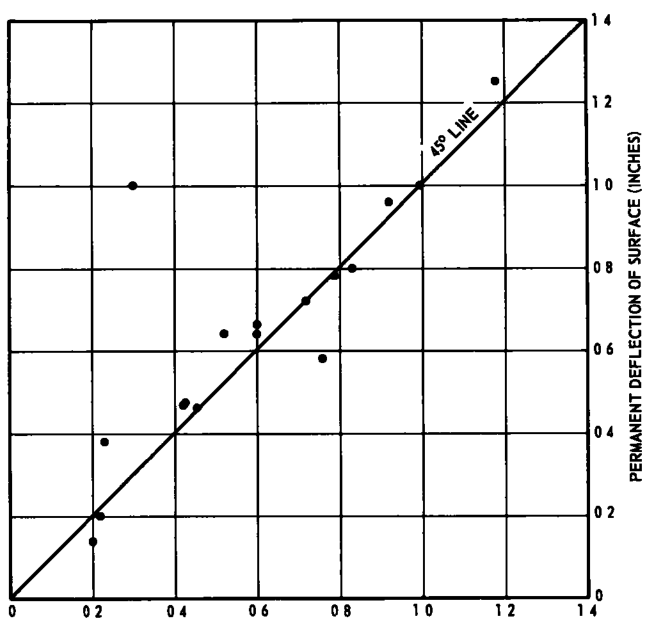

Figure 22. Permanent surface deflection vs change in thickness of permanent structure for structurally adequate pavements.

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 (*27*, Report 5) is: surface course, 32 percent; base course, 14 percent; subbase course, 45 percent; embankment, 9 percent. To make a proper interpretation of this distribution of layer thickness change, the mechanics of the change in layer thickness should be considered. The following pertinent facts regarding that change were established 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 increase 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-wheelpaths 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 these findings, 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, one must examine this pavement response in light of the known stress conditions.

Consider first the finding 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*), who 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 nonelastic 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 nonelastic deformation will occur within the subgrade, whereas 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 the 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 it always will be theoretically possible to construct a sufficiently thick pavement structure so that all the significant permanent deformation results 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 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 nonelastic 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:

Season	*Pavement Failure* (%)	*Traffic Distribution* (%)
Fall	5	26
Winter	9	21
Spring	80	25
Summer	6	28

The fact that 80 percent 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 and point out the need for consideration, in pavement design, of both the elastic and nonelastic phenomena.

A second important factor influencing rut depth is the vertical stress imposed by wheel loads through the pavement structure to the subgrade. Figure 23 shows the relationship of rut depth to vertical stress on the subgrade, as measured in both the 1959 and 1960 studies. As 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 psi, with a slight tendency of increasing with the wheel load.

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

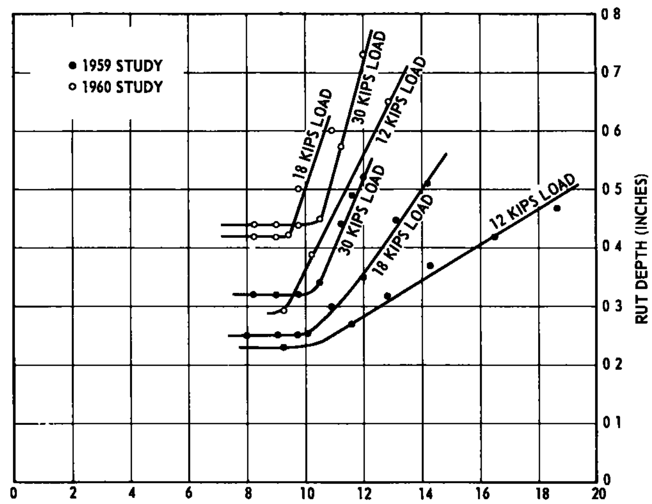

Figure 23. Surface rut depth as a function of vertical stress on the subgrade.

Other such possible findings would be most helpful for establishing rational design criteria along the principle of limiting subgrade stress.

CHAPTER FIVE

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, although 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. Therefore, 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 understanding of structural behavior of flexible pavements were reached. They are presented in the following 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 are classified under headings 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 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 homogeneous solid. Directly under the load they are considerably higher than those predicted by the conventional layered solid (Burmister) theory. They are highest when the subgrade, as well as the pavement structure, is practically saturated with moisture; they are lowest during frost periods. The stresses are also affected by the vehicle speed, being 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 are 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 pavement 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 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 unserviceable 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 (Fig. 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 supporting a very thick pavement structure, failure phenomena may resemble more the phenomenon of general shear of an incompressible soil under a footing (Fig. 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 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 unlimited 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, under both field and 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 is 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 under different conditions. The great potential value of this type of test has been proved 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.

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