Report No. FHWA/RD-81/032

FLEXIBLE PAVEMENT OVERLAY DESIGN PROCEDURES

Vol. 1. Evaluation and Modification of the Design Methods

August 1981

Final Report

Prepared for

U.S. Department of Transportation
Federal Highway Administration
Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

FOREWORD

This two-volume report, FHWA/RD-81/032 and 81/033, presents the results of research conducted by Resource International, Inc. for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9315. This work was a part of FCP Project 6D, "Structural Rehabilitation of Pavement Systems." The study was initiated to evaluate the overlay procedure developed under contract DOT-FH-11-8544 by Austin Research Engineers, Inc., published in reports FHWA-RD-75-75 and FHWA-RD-75-76. This procedure, along with several others, was compared and a slightly revised version has been recommended for implementation. Volume 1 discusses the evaluation and modification of the flexible overlay design procedure, and Volume 2 is a user manual for the revised procedure.

The overlay method presented is a combination and modification of several existing methods and incorporates the latest state-of-the-art concepts in pavement evaluation and overlay determination. The overlay thickness is determined based on a fatigue distress function developed from the AASHO Road Test data. The existing pavement is evaluated using nondestructive dynamic deflection measurements and a visual survey which includes general observations regarding drainage, the existence of rutting and the presence and type of cracking.

Copies of this two-volume report are being given widespread distribution by FHWA Bulletin. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Richard E. Hay for

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No. FHWA/RD-81/032	2. Government Accession No.	3. Recipient's Catalog No. PB82-136672
4. Title and Subtitle FLEXIBLE PAVEMENT OVERLAY DESIGN PROCEDURES VOLUME 1. EVALUATION AND MODIFICATION OF THE DESIGN METHODS		5. Report Date August 1981 6. Performing Organization Code 343039
7. Author(s) Kamran Majidzadeh and George J. Ilves		8. Performing Organization Report No.
9. Performing Organization Name and Address Resource International Inc. 130 E. Wilson Bridge Road Worthington, Ohio 43085		10. Work Unit No. (TRAIS) FCP 35D2-192 11. Contract or Grant No. DOT-FH-11-9315
12. Sponsoring Agency Name and Address Offices of Research & Development Federal Highway Administration U.S. Department of Transportation Washington, D.C. 20590		13. Type of Report and Period Covered Final Report Sept. 1977-Nov. 1980 14. Sponsoring Agency Code S01162

15. Supplementary Notes

FHWA Contract Manager: R.W. May (HRS-14)

16. Abstract

A review and evaluation of existing flexible pavement overlay design procedures, including the ARE procedure (FHWA Contract DOT-FH-11-8544) resulted in a fully computerized rational method for the design of flexible overlay of flexible pavements. The overlay method presented is a combination and modification of several existing methods and incorporates the latest concepts in pavement evaluation and overlay determination. The overlay thickness is determined based on a fatigue distress function developed from the AASHO Road Test data and relates horizontal tensile strain in the asphalt layer to the number of equivalent 18 Kip (80kN) axle loads to failure. The existing pavement is evaluated using NDT deflection measurements and a visual survey which includes general observations regarding drainage, the existence of rutting, and the presence and type of cracking. The deflection data is analyzed using 3-layer linear elastic theory; the in-situ layer stiffnesses are determined by matching measured deflections with those computed from layer theory. The in-situ asphalt modulus is compensated for temperature and this adjusted modulus is used in design computation. Base and subgrade moduli are corrected for stress effects when test loads differ from design loads, or when the state of stress is changed as a result of adding an overlay. Also incorporated is an environmental factor which enables NDT to be conducted during most of the year. This design method has been compared against overlay design methods used by 3 different states in the U.S. Limited comparisons show that the proposed procedure has universal applicability and results in overlay thicknesses consistent with local methods.
This volume is the first in a series. The other is: Flexible Pavement Overlay Design Procedures. User Manual, Volume 2 (FHWA-RD-81/033).

17. Key Words Flexible pavement, overlay design, pavement evaluation, fatigue, cracking, deflection basin analysis, layers stress correction	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	181	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

PREFACE

This research report outlines design procedures for asphaltic concrete overlay on flexible pavements. The analytical techniques and design procedures recommended in this document are based on sound, fundamental principles of the mechanics of paving materials and elastic layered systems.

The framework for these proposed overlay design procedures was initially developed by Austin Research Engineers, under contract with the Federal Highway Administration, Office of Research, Contract No. DOT-FH-11-8544. The FHWA-ARE design procedures were presented in two separate volumes: Report No. FHWA-RD-75-75 and FHWA-RD-75-76.

The evaluation of this new FHWA-ARE overlay design was contracted to Resource International Inc., under the financial support of the Office of Research, Federal Highway Administration, Contract No. DOT-FH-11-9315.

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CHAPTER 1
INTRODUCTION

1.1 OBJECTIVES

The objectives of this research report is to provide the pavement engineer with a rational and ready reference to design procedures for asphaltic concrete overlay of flexible pavements based on elastic layer theory. The design procedures and the analytical techniques presented have been formulated to predict the structural fatigue response of asphaltic concrete overlays for various design conditions, including geometrical and material properties, loading conditions and environmental variables.

1.2 SCOPE

This research project has been concerned with the evaluation and validation of the FHWA-ARE (1) overlay design procedures for flexible pavements. The overlay design procedures have been evolved from recent developments in the field of material characterization, from fundamental principles of pavement multi-layered elastic systems and pavement evaluation methodologies.

The FHWA-RII flexible pavement overlay design procedures presented in this report have been formulated after a detailed review of all currently available pavement overlay design methodologies and selection of the more promising procedures for evaluation by a panel of consultants, which was selected from among many highly recognized and promiment highway engineers. The review board, in accordance with the project requirements, requested that field data be obtained from in-service data, which the panel then screened for purposes of analysis, verification, and examination of the rationality, sensitivity, and degree of reasonableness of the calculated overlay thickness.

The review board, upon examination of the design examples provided for data from in-service pavement conditions, recommended the procedure that is documented in this report to be adopted as the FHWA flexible pavement overlay design procedure.

In this report, the results of the research investigation are presented in six chapters, enumerated as follows:

1. INTRODUCTION
2. EVALUATION OF DESIGN PROCEDURES
3. FORMULATION OF A STANDARD DESIGN PROCEDURE

4. DESIGN CRITERIA AND PROCEDURES DESCRIPTION
5. MATERIAL CHARACTERIZATION
6. DESIGN EXAMPLE - VERIFICATION

In Chapter 1, which is an introductory chapter, the scope of study, the research tasks and the functions and responsibilities of the panel are discussed. Chapter 2 contains a detailed discussion of all currently available design methods, their evaluations and panel discussions and considerations for selecting the most promising design methods. The rationale for the selection of the FHWA-RII design procedure and design criteria recommended by the panel are outlined in Chapters 3 and 4.

The procedures for material characterization, field evaluation and operating instructions are provided in Chapter 5. Typical design examples on various pavement sites are provided in Chapter 6.

The User and Implementation Manual for the FHWA-RII (2) overlay design procedures is provided as a separate publication. However, the Appendices include an annotated bibliography and a data file and guidelines for selection of typical moduli of pavement components.

1.3 REVIEW BOARD FUNCTIONS

To achieve the objectives of this study, it was required that an in-depth and unbiased evaluation of currently available design procedures be carried out, and that the more promising and attractive design procedures be selected for detailed evaluation. A panel of consultants of prominent highway engineers was selected to act as a review board and render judgment on the evaluation, validation, sensitivity analysis and the degree of reasonableness of the various currently available design methods.

The panel of consultants, which was selected to represent various state and user agencies, included the following notable highway engineers:

Jatinder Sharma, P.E., Engineer of Pavement Design
Florida Department of Transportation

Gene R. Morris, P.E., Engineer of Research
Arizona Department of Transportation

Fred Finn, P.E., Senior Project Engineer
Woodward-Clyde Consultants

George Sherman, P.E., Engineer of Materials
California Department of Transportation (retired)

Dale Peterson, P.E., Research and Development Engineer
Utah Department of Transportation

James Shook, P.E.
The Asphalt Institute

Leon O. Talbert, P.E., Engineer of Research
ODOT, Host panel member representing host state.

The panel functions were:

(1) to conduct a review and preliminary evaluation of the documentation pertaining to all currently available overlay design methods;

(2) to select the more promising candidate methods for further detailed evaluation and examine for rationality, mathematical limitations and sensitivity;

(3) to design a system analysis process for final screening and correlation under various field service conditions;

(4) to review the results of analysis of selected overlay design procedures using actual input data, and recommend and document a "standard" method of pavement overlay design.

CHAPTER 2
EVALUATION OF OVERLAY DESIGN PROCEDURES

The formulation of a "standard" flexible pavement overlay design procedure was preceded by an in-depth and critical review of the current state of the art. This review included assembly of all pertinent information concerning design procedures currently in use by various agencies, as well as the review of procedures developed by Austin Research Engineers under FHWA contract DOT-FH-8544. The state of the art review included the design methodologies recommended by The Asphalt Institute (3), California (4), Oklahoma (5), Utah (6), Louisiana (7), Virginia (8), Ohio (9), and procedures adopted by Shell (10), South Africa (11), and England (12). In this chapter the results of preliminary evaluation and the selection of most promising methods for detailed evaluation are presented.

2.1 EVALUATION PROCEDURES

To conduct a preliminary evaluation of the assembled literature pertaining to overlay design procedures, a set of evaluation criteria was selected to examine the uniqueness of various methodologies. This evaluation included the following considerations:

(1) rationality of approach;

(2) analysis techniques; does it include detailed structural analysis of multi-layer pavements;

(3) the basis for the condition evaluation of existing pavements; does it consider a rational method of pavement condition evaluation or use a subjective method;

(4) can it be used to estimate the remaining life of pavements;

(5) does it consider various design criteria such as rutting and fatigue or is it based on empirical methods;

(6) can it be used to optimize various overlay design strategies and material properties;

(7) the nature of input variables; does it include the stress, temperature and environmental dependency of material variables;

(8) does it include the nature of pavement distress and identify the mechanisms involved;

(9) the simplicity and the reliability of analysis.

In this review process, it was ascertained that each method was evaluated without bias for its rationality and acceptability as a design procedure and examined as to whether its unique functions, if any, could be adopted for the formulation of the standard method.

The evaluation process using the stated criteria was expected to distinguish those design procedures which do not include rigorous analysis of the structural conditions, or methodologies which are merely extensions of empirical design methods modified for overlay design procedures.

This preliminary evaluation process was also aimed at identifying those procedures which are limited due to built-in factors pertaining to experience and judgment, or which lack rational consideration to the determination of remaining life, distress functions, etc.

2.2 PRELIMINARY EVALUATION

The review of currently available overlay design procedures indicated that the majority of these methods are based on empirical concepts developed on the basis of built-in experience factors of local significance, and that such methods lack consideration to mechanistic theories. As a result of this evaluation process and with consideration to the evaluation criteria presented, the overlay design procedures were placed into three distinct categories:

(1) Entirely empirical - Maximum deflection-based methods;

(2) Simplified graphical procedures generated from rational and mechanistic models, using in-situ moduli and deflection basin parameters;

(3) Rational mechanistic, based on multi-layer elastic analysis and computerized procedures, FHWA-ARE.

2.2.1 Maximum Deflection-Based Methods

The majority of empirical overlay design procedures are based on the utilization of maximum deflections of existing pavements and on the concept of limiting deflection criteria to arrive at the overlay thickness. According to these entirely empirical procedures, the overlay thicknesses are calculated to reduce the deflections to a tolerable or allowable level, which is determined from the empirical interrelation between deflection and axle load applications.

The maximum deflection overlay design procedures are numerous and include methods from The Asphalt Institute (3), California (4), Oklahoma (5), Louisiana (7), British Road Research Laboratory (12), Pennsylvania (13), and Utah (6).

In these procedures, the overlay design calculations are carried out as follows:

(i) Pavement condition evaluation is conducted using a load-inducing device such as the Benkelman Beam, Lacroix Deflectometer, Road Rater, Dynaflect, or a similar instrument, and the maximum deflection is measured at predetermined intervals.

(ii) Depending upon the specific design procedures adopted by the user agencies, a "representative" or a "design" maximum deflection is calculated for each test section. A design maximum deflection could be calculated using either an 80 or 95 percentile, an average, or a value based on mean standard deviation of measurements.

(iii) The selection of the test frequency - i.e., the number of tests per mile - differs among the various design agencies and in general, the frequency of test measurements are insufficient to fully characterize the pavement structure.

(iv) A standard or reference testing period is selected which depends on the user agency's climatic conditions and prior experience. In certain states, such as California, no specific testing period is recommended, whereas other states, such as Utah, Louisiana and Pennsylvania, recommend specific testing periods.

(v) The design maximum deflection is adjusted to a reference temperature (such as 60, 68 or 75°F) (15.5, 20 or 24°C) using a Temperature Adjustment Factor. The derivation of the TAF in most design procedures is modeled after the Kentucky (14) procedure. The measured pavement deflection that occurs in different thermal regions is highly dependent upon the pavement geometry, modulus-temperature relationships, and temperature distribution within the pavement layered system. The TAF, however, does not consider the pavement geometry and layered structure.

(vi) The allowable pavement deflection for an estimated traffic is determined from an empirical relation developed between traffic (number of 18 kips (80 kN) axle loading) and the deflections. Such nomographs are developed by various user agencies from field data on in-service pavements and from deflection measurements on large numbers of projects and correlation with field performance and overlay thickness requirements.

(vii) In many deflection-based methods, the required overlay thicknesses are also presented in terms of structural numbers and reduction in pavement deflection, and graphical solutions or nomographs are provided for engineering applications.

A major criticism of the use of deflection-based methods is that deflection-load relationships used in these procedures have been derived with consideration to local conditions and have built-in experience and judgment factors that cannot be extrapolated to other regions. That is, the extension of such procedures would undoubtedly constitute an unrealistic extrapolation of field performance of one region to another.

These empirical procedures, however, serve a good purpose in providing easy-to-use and field-proven design practices for the region in which the design scheme has been formulated, based on many years of trial and error, and experience.

2.2.2 Simplified Graphic Procedures Using Deflection Basin Parameters

The overlay design procedures in this category are based on semi-rigorous and rational analyses of structural conditions of in-situ pavements prior to the overlay.

These design procedures employ the deflection basin to obtain an estimate of pavement structural conditions and in-situ stiffness, such as moduli of pavement layers and their effective thicknesses. In these design schemes, such as the procedures used in Virginia (8), Ohio (9), Texas (15), and by Shell (10), the deflection basins are measured by a load-inducing device such as Dynaflect, Road Rater, Falling Weight Deflectometer (FWD) or other similar instruments. In these procedures, except for the Shell design, the pavement structure is approximated by a two-layer elastic half-space. The pavement structure is formally characterized by either its in-situ modulus or in-situ thickness.

The in-situ modulus, also defined as the "effective modulus", refers to the modulus of an existing pavement having a thickness equal to its original design value. The majority of overlay design procedures discussed in this category are based on the concept of effective in-situ modulus. The in-situ modulus of the pavement layer or the subgrade is estimated using nomographs, design charts, and graphical solutions and computer programs.

It should be noted, however, that actual in-situ pavement thickness will quite often differ from the design thickness. Therefore, the calculation of an in-situ modulus which is based on the assumption of a constant thickness might also reflect the statistical and inherent variations of the thickness itself. As a result, it is more appropriate to use the term "in-situ stiffness" in lieu of in-situ modulus in order to reflect the combined variations of both thickness and modulus.

The in-situ thickness concept, also known as "effective thickness", has been presented by the Shell (10) design method, which reflects the relative value of the in-situ thickness in reference to the original design.

According to the Shell (10) procedures, it is assumed that the pavement layer retains its original modulus or stiffness, but its deterioration in service is reflected by a hypothetical change in the pavement thickness. For example, an eight-inch thick pavement might have an equivalent thickness of five inches after a few years in service. This means in essence that the pavement works effectively as a five-inch pavement which has just been constructed.

In these procedures, the overlay design calculations are carried out as follows:

(i) In contrast to maximum deflection procedures, the deflection profile is determined with more than one sensor and at more than one position. The Shell (10) design, using a Falling Weight Deflectometer, measures the deflection basin at two distinct points. Other procedures using Dynaflect or Road Rater, however, measure the deflection profile at five or four positions, respectively.

(ii) The measured deflection profile provides an estimation of load spreadability, pavement surface and support conditions, such as surface curvature index, base curvature index and spreadability.

(iii) The pavement in-situ modulus, subgrade modulus and effective thickness are estimated using nomographs, design charts or computer programs. The Shell (10) design also assumes a relation between the base mdoulus, the thickness of the base and the subgrade moduli. To estimate the base modulus, the base thickness is assumed as known.

(iv) The use of Temperature Adjustment depends on the design procedure. The Shell (10) design does not use a TAF for correction of deflection; instead, sections within the same average testing temperature are considered together. The mean pavement temperature is obtained and the modulus of asphaltic concrete is estimated from the moduli-temperature curve for the test temperature conditions. The Shell (10) design used mean annual air temperature to estimate asphalt modulus. The subgrade modulus is assumed to remain constant throughout the life of the pavement. There is no specific guideline for the recommended testing period or adjusting the deflection subgrade modulus to a critical time.

(v) The Ohio (9) procedure recommends a temperature adjustment factor to adjust the asphalt modulus to a reference temperature; however, the correction for subgrade modulus is not clearly identified.

(vi) The procedures do not consider the stress dependency of the subgrade and granular base course layer. In the Shell (10) design procedure, the stress dependency of subgrade soils is not considered, therefore, the procedures for the estimation of base modulus might require verification.

(vii) The procedure for the determination of remaining life and the estimation of overlay thickness vary according to user agencies. The Shell (10) design calculates the overlay thickness using the effective or equivalent thickness. The required overlay thickness is the difference between the required thickness for the design life and the in-situ effective thickness. The remaining life is the difference between the design life and the actual loading.

(viii) The use of rational distress functions differs among the user agencies. The overlay design schemes are generally developed for fatigue distress criteria only and do not consider the rutting phenomena as a critical design procedure. These criteria are developed empirically from observed performance of in-service pavements. The Shell (10) design alone uses a fatigue criteria developed for lab tests and adjusted for field conditions to account for variation in the loading.

In summary, the results of this preliminary evaluation indicate that overlay design procedures in this category are simplified procedures which have been developed on the basis of semi-rigorous and rational analysis of multi-layer elastic pavement systems. The Shell (10) design uses a three-layer system whereas the Ohio (9), Texas (15) and Virginia (8) designs use a two-layer approximation of the pavement structure. The distress function used is a rational fatigue distress criteria developed for laboratory specimens. The concept of effective thickness used by the Shell (10) design simplifies the design life and overlay calculations as compared to the concept of effective modulus. These procedures, in general, have remained relatively unknown and have not been implemented by highway departments.

2.2.3 Mechanistic Overlay Design Procedures Using Multi-Layer Elastic Analysis

The overlay design procedures in this category are based on the sound theoretical formulation of pavement response and mechanistic analysis of a multi-layer elastic system. These mechanistic schemes involve rational distress functions, material characterization procedures, and rigorous analysis of pavement structural conditions. Although the basic approaches to the formulation of these mechanistic overlay design schemes are the same, various research institutes, universities and user agencies have developed their own computer softwares, material characterization techniques and assumptions pertaining to the use of a distress function.

The South African (11) pavement design procedure is a typical example of a very rigorous, rational procedure which takes into consideration most factors influencing pavement performance and which could also be extended to include the design of pavement overlays.

The FHWA overlay design procedure developed by the Austin Research Engineers, under FHWA Contract DOT-FH-11-8544, and presented in Report Nos. FHWA-RD-75-75 (1) and FHWA-RD-75-76

(16) constitute such a mechanistic and rational overlay design which has attempted to translate and unify very highly sophisticated theories into simplified and implementable design procedures.

The overlay design schemes are based on the following:

(i) The existing pavements are evaluated using Dynaflect, Benkelman Beam, or Road Rater, and visual observations to include an estimate of the extent of Class 2 and Class 3 cracking.

(ii) The deflection measurements are carried out on the pavement structure. Only the maximum deflection (W_1) or the first sensor reading of the Dynaflect is used. The road is divided into sections having similar properties, such as maximum deflection, W_1, and Class 2 and Class 3 cracking.

(iii) Statistical analysis of data or deflections is carried out to determine whether various sections have statistically significant differences in their deflections (W_1). The overlay design is based on a design deflection, W_a, determined from mean $\bar{W}$ and standard deviation, SD:

$$W_a = \bar{W} + Z\ SD$$

where Z is the confidence level required.

(iv) The subgrade support modulus is estimated from the design deflection, W_a.

(v) The FHWA-ARE procedure considers the stress dependency of the subgrade modulus. The subgrade modulus, estimated from the deflection for a Dynaflect loading, is adjusted to a modulus corresponding to an 18 kip (80 kN) loading condition using graphical procedures.

(vi) The overlay design procedures require that the moduli of the paving material - asphaltic concrete, base course and subbase - be determined by laboratory testing. The asphaltic concrete samples are tested at 70°F (21°C), corresponding to a mean annual temperature used in the design. The base course and subbase are tested at confining pressures ranging from 10 to 20 psi (69 to 138 kPa), and the subgrade is tested at confining pressures ranging from 2 to 12 psi (14 to 83 kPa).

(vii) The distress functions used in the analysis have been developed for the fatigue and rutting criteria, and are based upon the results of the AASHO Road Test.

(viii) The structural analysis and the stress calculations are based on ELSYM5 computer program, which uses a five-layer elastic half-space developed by University of California researchers. It is capable of handling up to ten circular loads, but cannot take into consideration the shear stress.

(ix) The remaining life analysis, similar to other rational procedures, is calculated as a difference between the design life and total traffic experienced. The design life calculation is based upon currently-measured pavement values, which probably gives a more conservative design.

(x) In this overlay design scheme, it is recommended that the pavement evaluation be carried out at the worst season, especially in areas where moisture is a problem. The design also relies on the designer's judgment to adjust the layer moduli to the worst conditions expected in the field.

The overlay design scheme presented above is based upon rational principles of pavement performance. Nevertheless, there are several areas of potential improvement of the proposed procedures.

2.3 SELECTION OF MORE PROMISING DESIGN PROCEDURES

A preliminary evaluation of currently available overlay design procedures indicated that there are many alternative solutions and design methodologies applicable to overlay of flexible pavements. Those existing design procedures were, as discussed previously, categorized into the following three design methodologies:

(1) Entirely empirical - Maximum deflection-based methods;

(2) Simplified graphical procedures - Deflection basin method;

(3) Rational Mechanistic Multilayer Analysis - FHWA-ARE.

To select the more promising design procedures for further in-depth evaluation, the review board examined each method without bias for its rationality and acceptability as a design procedure. It was recognized* that often, engineers are looking for simple, straight-forward procedures which provide, with one or two inexpensive and brief observations, answers to all the design requirements.

The Utah (6) design procedure, as discussed previously, is typical of the "entirely empirical - maximum deflection-based methods" design category and is a procedure which could satisfy the engineer's need for a simple short-cut approach to design. The reasoning for the selection of the Utah (6) method as a "More Promising Candidate" is as follows:

1. It is a simple and straight-forward procedure.

2. It is a working method, already in existence in three to four states in various formats, and has been accepted by practicing engineers.

3. It has built-in experience; it is empirical; and it has been formulated using field data.

4. It utilizes field data from pavement condition evaluations of in-situ pavements using maximum deflection, and makes adjustments for environmental conditions.

It should be pointed out, however, that such simple or short-cut procedures are often inadequate and cannot explain the causes of pavement failure; nor can such procedures make diagnoses of pavement behavior**.

Within the second category of design schemes, a simplified graphical procedure using deflection basin parameters was selected as one of the promising candidate methods. The review board's justifications for this selection were primarily based on its simplicity and the unique attractiveness of the equivalent thickness concept.

It was indicated that the concept of equivalent or reduced thickness deserves serious consideration. It provides a semi-rational, mechanistic and, at the same time, an extremely easy-to-follow procedure for the determination of required overlay thickness. The other significant advantages of this procedure were noted as follows:

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*Commentary by Review Board
**Panel Commentary

1. Use of deflection basin parameters to measure in-situ pavement moduli and effective thicknesses;

2. Simplified two-layer approximation of pavement structure;

3. Use of elastic layer theory to arrive at stresses and strains in the pavement structure;

4. Use of a distress function for fatigue and rutting failure. The distress function could be derived, based either on laboratory testing or using field performance data.

5. The results have the potential for representation in the form of non-dimensionalized or normalized graphical solutions.

The review panel also selected the semi-computerized, rational, mechanistic overlay design procedure, developed under an FHWA Contract (FHWA-ARE) (1) as one of the more promising techniques.

As a justification for the selection, it was stated that although practicing engineers are most often interested in short-cut methods and easy-to-follow schemes, among various design alternatives available, simple procedures are unfortunately inadequate to provide explanation for pavement problems.

The use of the FHWA-ARE (1) overlay design procedure is a step forward toward a rational pavement analysis and diagnostic pavement investigation.

The justifications for the selection of this method as a "More Promising Candidate" are as follows:

1. It is a rational procedure, recognizing the state of stresses, strains and material properties in a pavement layered structure.

2. A multilayer elastic program, ELSYM5 (17), is used for the calculation of stresses and strains in the pavement structure.

3. It takes into consideration the in-situ pavement conditions and the subgrade support, and deflection variability within a given pavement section is incorporated into the analysis.

4. The design and analysis are based on detailed laboratory evaluations of the moduli of the pavement component layers.

5. In this design scheme, the in-situ moduli of the existing pavement structure are selected with consideration to the severity of pavement distress. Lower moduli values are measured for pavements with a greater degree of distress manifestation.

6. The distress function used in this design procedure has been developed on the basis of AASHO Road Test performance data.

2.4 AN EVALUATION OF THE METHODS SELECTED AS MORE PROMISING CANDIDATES

In Section 2.3, the justification for the selection of the three more promising candidate methods was discussed. These three methods - Utah (6) empirical procedures; effective thickness; and rational FHWA-ARE (1) semi-computerized method - were subjected to detailed evaluation and a sensitivity analysis of pertinent variables.

The results of the sensitivity analyses and the criticisms of the three more promising candidate methods are as follows:

2.4.1 Utah Maximum Deflection

The Utah (6) method is an empirical design procedure which utilizes the maximum pavement deflection to arrive at the required pavement overlay. This method, which is more widely recognized and used in practice than the other two procedures, was originally developed on the basis of the maximum Benkelman Beam deflection and its correlation with Dynaflect maximum deflection.

The results of sensitivity analysis and theoretical modeling, as shown in Figure 1, indicates that the interrelation between pavement deflection using Benkelman Beam and Dynaflect is highly dependent on the pavement modulus and pavement thickness, especially for thinner pavements.

The second theoretical weakness in the Utah (6) method, as well as in all maximum deflection-based procedures, is the determination of the Temperature Adjustment Factor for maximum deflection.

The results of the theoretical investigation, as shown in Figure 2, based on the modulus-temperature relationship

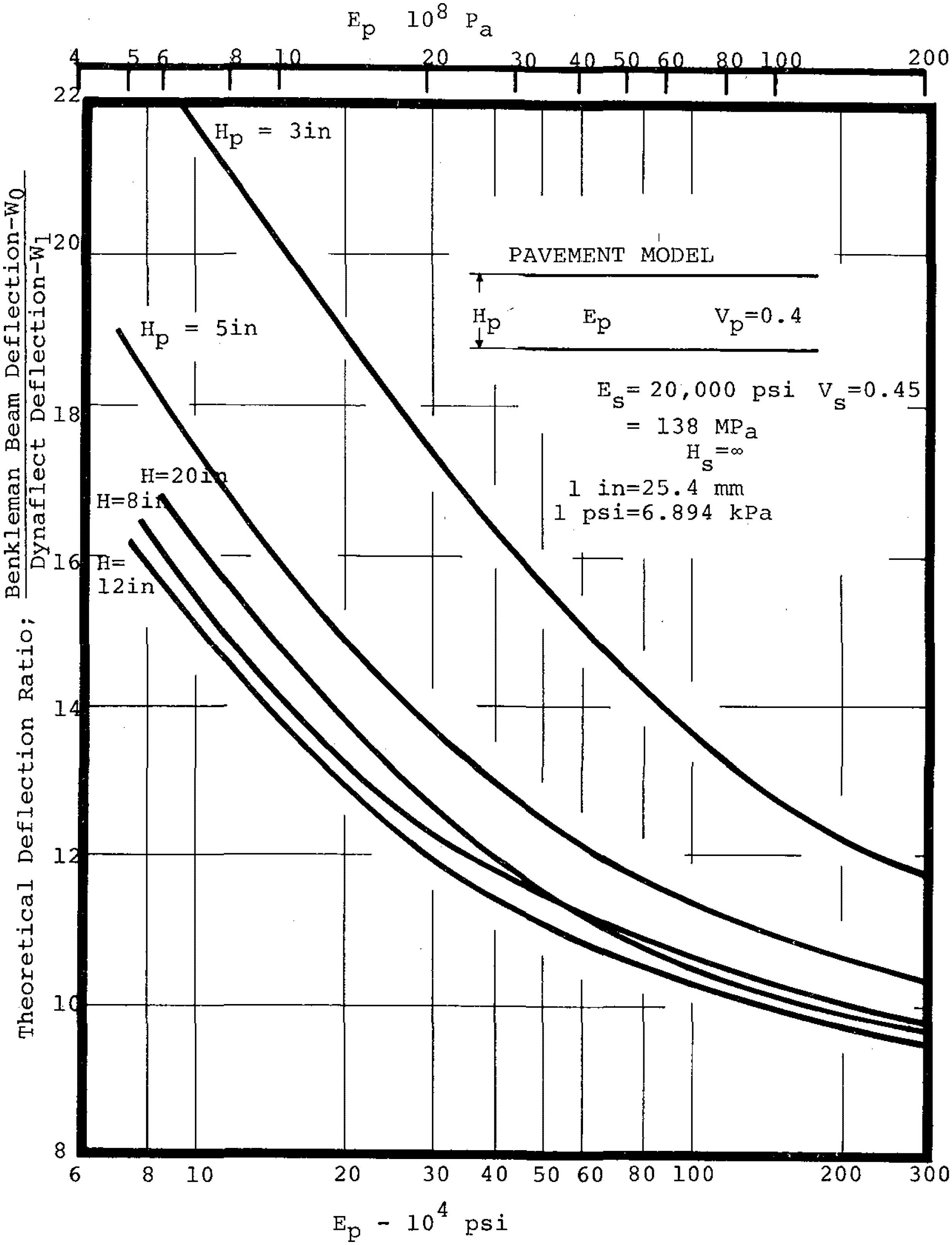


Figure 1. Variation of Theoretical Deflection Ratio (W_0/W_1) with Pavement Modulus for Various Pavement Thicknesses

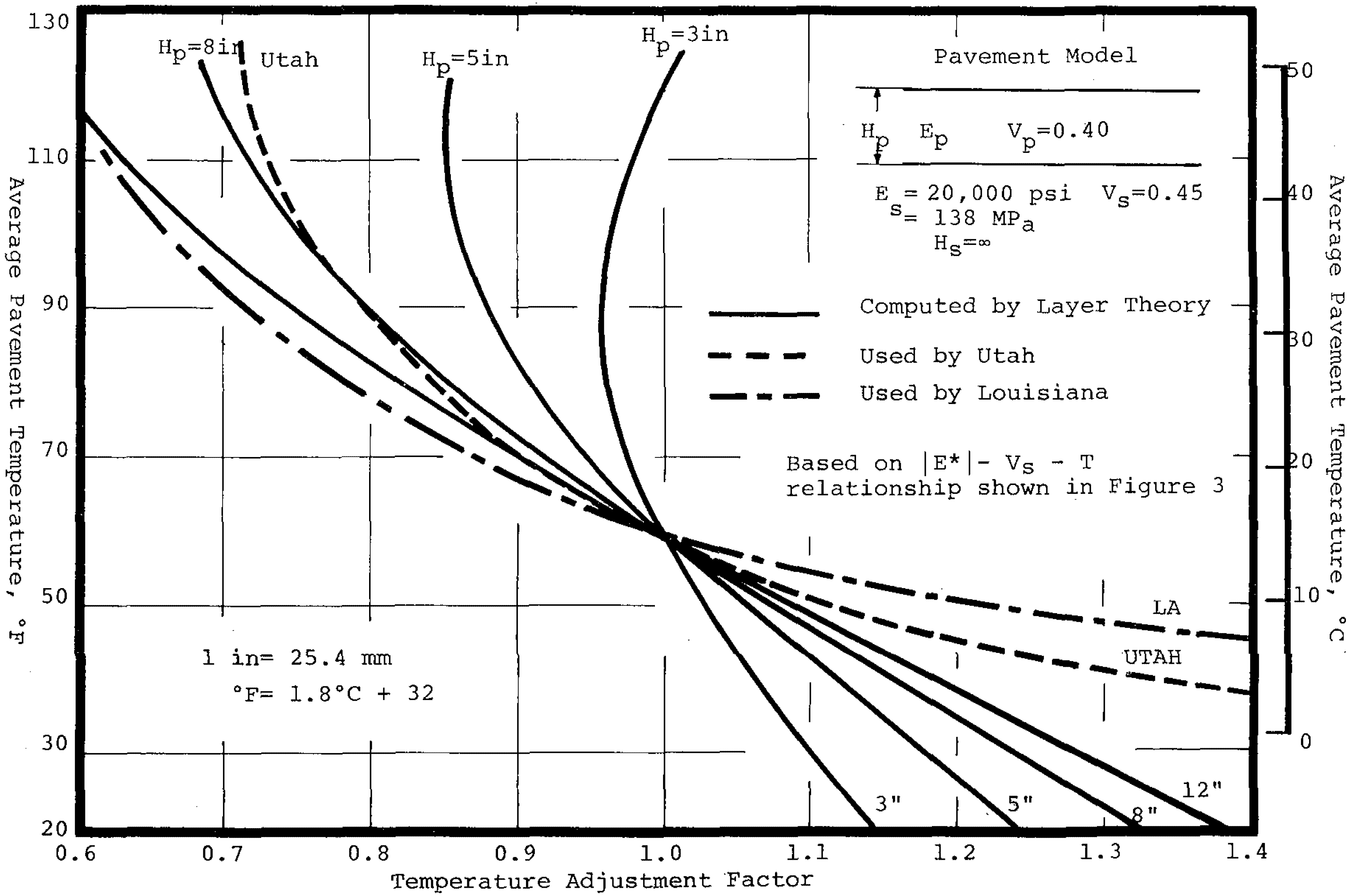


Figure 2. Temperature Adjustment Factor for Dynaflect Maximum Deflection W_1

shown in Figure 3 (18), indicate that the Temperature Adjustment Factor - pavement temperature relationship is not a unique function. Rather, the TAF-pavement temperature relationship is highly dependent upon the pavement thickness, especially for thinner pavements. Furthermore, the adjustment of maximum deflection for a temperature change does not reflect the adjustment needed in the deflection basin nor the adjustment required in the pavement moduli.

Thirdly, the design procedures that are based on maximum deflection alone are extremely sensitive to the subgrade support conditions, and often result in over-conservative designs. The results of sensitivity analysis and theoretical evaluation of the maximum deflection method, as compared to the Equivalent or Effective Thickness procedure (Table 1), indicates that the required overlay thickness could, in many cases, lead to overdesign. It should also be noted that the maximum deflection procedure, lacking consideration to the deflection basin, cannot evaluate the in-situ characteristics of pavement components, such as base, subgrade and pavement surface layers.

2.4.2 Two-Layer Graphical Procedures

In this design system, the shape of the deflection basin and its variation due to material properties as well as to environmental conditions are a major design consideration. In this overlay design procedure, as presented in Appendix A, the engineer might choose between two design alternatives:

(i) In-situ stiffness; or

(ii) Equivalent thickness of existing pavement.

Both alternatives are based on a two-layer approximation of the pavement system. The graphical solutions using three-layer pavement structures would require very large numbers of graphs and time-consuming procedures for overlay calculations.

According to the in-situ stiffness procedure, the pavement in-situ moduli are calculated using maximum deflection and spreadability. These in-situ moduli or stiffnesses are then used in the two-layer model to compute the resultant stresses and strains, and the overlay requirement is subsequently calculated using an approximate distress function.

This procedure, although more simple and rational than the Utah (6) method, involves approximation of the pavement by a two-layer system and involves a number of steps which are not quite easy to follow. The results of the sensitivity analysis and data evaluation from in-service pavements indicate that distressed pavements requiring overlay are rarely

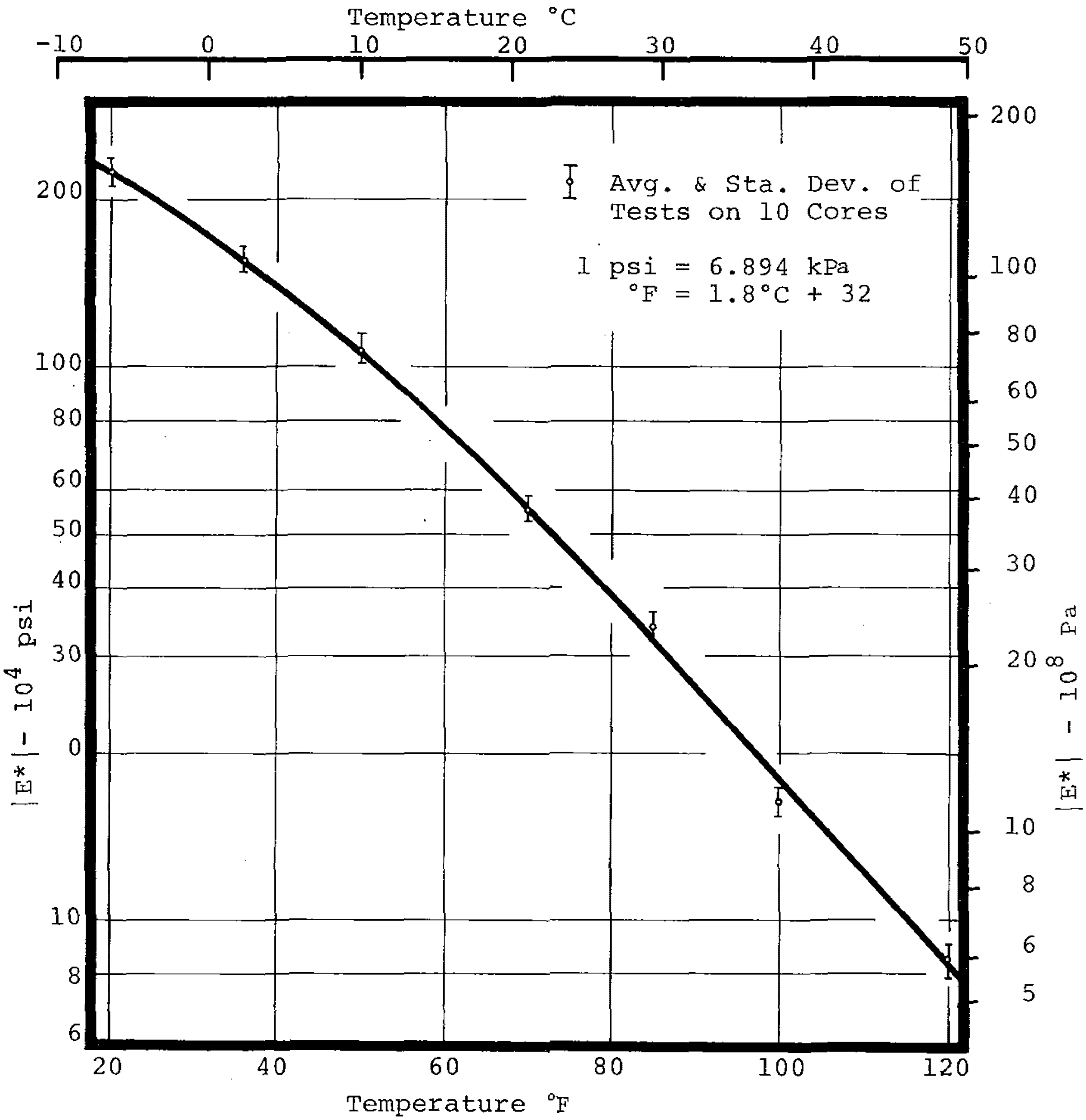


Figure 3. Dynamic Modulus of a Dense Graded Asphaltic Concrete at 8 Hz. (18).

TABLE 1. OVERLAY REQUIREMENTS OF PAVEMENTS WITH VARIOUS EFFECTIVE THICKNESSES

Dynaflect W_1 mils	Meas. SP %	Effective Thickness H_{eff}(1) in	E_s 10^3 psi	ε_r(2) μ in/in	Overlay Required H(3) in	H(4) in
1.37	50.0	4.0	20.0	310	6.0	6.0
1.26	53.0	5.0	20.0	253	5.0	5.0
1.16	56.2	6.0	20.0	209	4.0	4.0
0.97	61.8	8.0	20.0	146	2.0	2.0
0.82	66.8	10.0	20.0	106	0	0
2.53	53.2	4.0	10.0	405	7.1	14
2.27	57.0	5.0	10.0	320	6.1	13
2.02	60.8	6.0	10.0	256	5.1	12
1.65	67.2	8.0	10.0	175	3.1	10
1.39	72.3	10.0	10.0	125	1.1	8
1.18	76.3	12.0	10.0	94	0	6
6.80	60.8	4.0	3.0	585	8.6	≈31
5.80	65.9	5.0	3.0	435	7.6	≈30
5.02	70.2	6.0	3.0	340	6.6	≈29
4.00	76.7	8.0	3.0	225	4.6	≈27
3.30	81.6	10.0	3.0	157	2.6	≈25
2.75	85.5	12.0	3.0	122	0.6	≈23

(1) Pavement represented by two-layer model with $H_p = H_{eff}$ and E_p = 500,000 psi (H_{eff} & E_s determined from 2-layer graphical procedure)

(2) Strain under pavement layer with $H_p = H_{eff}$ (from layer theory)

(3) Overlay required to reduce horizontal strain at bottom of pavement to 106 μ in/in (determined from layer theory)

(4) Overlay required to reduce maximum Dynaflect deflection to 0.82 mils (determined from layer theory)

1 mil = 0.025 mm
1 in = 25.4 mm
1 psi = 6.895 kPa

1 μ in/in = 10^{-6} mm/mm

represented by a two-layer system.

The second design alternative, "equivalent thickness", is more attractive and straight-forward than the in-situ modulus concept. This design procedure, however, is more adaptable to the two-layer pavement structure. The formulation of graphical solutions for three-layer pavement systems is quite involved and results in very large numbers of graphs. It should be noted, however, that the overlay thickness calculated using either method meets the test for degree of reasonableness and is in accordance with accepted engineering practices.

In Table 2, the calculated overlay thickness for the two-layer graphical solution is compared with the overlay requirements using Utah (6) maximum deflection procedures.

It should be noted that the effective thicknesses and subgrade moduli listed in Tables 1 and 2 have been determined from the graphical procedure using the tabulated deflection and spreadability values, and these layer properties have been used in the multilayer analysis. The required overlay in Table 1 has been determined from layer theory using two criteria: H(3) is the overlay thickness required to reduce the critical strain under the asphalt layer to 106 microstrain (the allowable strain for 3.1×10^6 EAL from the ARE (1) distress function), and H(4) is the overlay thickness required to reduce the maximum Dynaflect deflection to 0.82 mils (0.021 mm), which is the allowable deflection for 3.1×10^6 EAL from Utah (6) procedure. The overlay thicknesses in Table 2 have been derived as follows: H(2) represents the thickness required by the Utah (6) method, based on the tabulated deflection, and H(3) has been determined by the effective thickness procedure described in Appendix A to limit tensile strain to 106 microstrain. The difference between H(3) in Tables 1 and 2 is that the subgrade has been assumed to be stress-dependent in Table 2.

It is interesting to note that although the Utah (6) design is based on limiting the deflection to a certain value, it does not achieve its desired objective according to layer theory, i.e., H(4) in Table 1 (the thickness of overlay predicted by layer theory) is considerably greater than H(2) in Table 2 (predicted by the Utah (6) method). Also, as may be seen from the comparison in Table 2, the Utah (6) method requires considerably lower overlay thicknesses than does the 2-layer effective thickness method for relatively good subgrades, but requires substantially thicker overlays for weak subgrades, pointing out the sensitivity of deflection-based methods to subgrade modulus.

TABLE 2. COMPARISON OF OVERLAY THICKNESSES DETERMINED BY UTAH METHOD AND TWO-LAYER GRAPHICAL PROCEDURE

Dynaflect Meas.		Effective Thickness (1)		Overlay Required	
W_1 mils	SP %	H_{eff} in	E_s 10^3 psi	H (2) in	H (3) in
1.37	50.0	4.0	20.0	2.8	8.8
1.26	53.0	5.0	20.0	2.3	7.8
1.16	56.2	6.0	20.0	1.8	6.8
0.97	61.8	8.0	20.0	0.6	4.8
0.82	66.8	10.0	20.0	0	2.8
2.53	53.2	4.0	10.0	7.9	9.5
2.27	57.0	5.0	10.0	7.0	8.5
2.02	60.8	6.0	10.0	6.0	7.5
1.65	67.2	8.0	10.0	4.4	5.5
1.39	72.3	10.0	10.0	3.0	3.5
1.18	76.3	12.0	10.0	1.9	1.5
6.80	60.8	4.0	3.0	≈16	10.5
5.80	65.9	5.0	3.0	≈14	9.5
5.02	70.2	6.0	3.0	≈13	8.5
4.00	76.7	8.0	3.0	11.5	6.5
3.30	81.6	10.0	3.0	10.1	4.5
2.75	85.5	12.0	3.0	8.6	2.5

(1) Pavement represented by two-layer model with $H_p = H_{eff}$ and E_p = 500,000 psi

(2) Overlay determined by Utah method with allowable deflection of 0.82 mils

(3) Overlay required using the two-layer graphical procedure of Appendix A to reduce the tensile strain to 106 microstrain.

1 mil = 0.025 mm
1 in = 25.4 mm
1 psi = 6.895 kPa

2.4.3 Rational Overlay Design - FHWA-ARE

This design procedure which is the most rational of three procedures is a mechanistic based method using semi-computerized analysis procedures.

The evaluation of the FHWA-ARE (1) method, using in-service performance data was not possible. This design method is based on the moduli of pavement components, obtained from laboratory testing, which are not available except for very limited experimental pavements. Although the verification of the model using input in-service performance data was not possible, the evaluation procedures included the examination of the rationality of the procedures and test for the reasonableness of the methodology.

The major drawback of this overlay design procedure was cited as its dependence on the laboratory input data. At the present, the laboratory data, as a standard practice are not available and any design procedures dependent on such data are probably not quite implementable.

CHAPTER 3
FORMULATION OF A "STANDARD" DESIGN PROCEDURE

A critical review of the state of the art and the evaluation of three more promising candidate methods, as was discussed in Section 2.3, resulted in the development of specific guidelines to be incorporated into the formulation of a "standard" design procedure. These recommended guidelines were concerned with the functions and limitations of the "standard" methods as a universal design procedure for overlay design of flexible pavements.

3.1 GUIDELINES FOR THE FORMULATION OF A "STANDARD" METHOD

The FHWA-RII overlay design procedure, as was conceived by the Review Board, is an overlay design methodology which is primarily concerned with the calculations of the overlay thickness requirements for an existing flexible pavement. This design procedure, contained in a computer program called OAF, is not intended to establish the justification, priority level, or the need for an overlay; rather, it is merely a design procedure which follows after the decision to overlay and the maintenance priorities have been established.

Secondly, the FHWA-RII Flexible Pavement Overlay Design Procedure is limited to the calculations of the overlay structure by brute strength methods, neglecting the analyses for reflection cracking, crack propagation, and the influence of cracking in the underlying layers. Since the design requirements that are concerned with reflection cracking have not yet been fully developed, the engineer should consult research documentation on the subject of fracture mechanics as conducted at Ohio State University, California, and elsewhere.

Thirdly, the FHWA-RII Flexible Pavement Overlay Design Procedure does not address itself directly to field conditions requiring special treatment, such as swelling clays, drainage improvement, and the use of unconventional paving mixtures such as sulfur-asphalt, asphaltic mixtures with waste by-products, etc. In such cases, the design engineer is expected to consult other sources for obtaining required data on distress functions, moduli data, and other related information.

Fourthly, it should be emphasized that in this design procedure, as in any other methodology regardless of its level of sophistication, there shall be no valid substitution for the engineering judgment and use of good, sound engineering principles.

The other recommended guidelines for the formulation of a "standard" design method are as follows:

3.1.1 Distress Function

The most essential element of a rational and standard procedure is the distress function relating the pavement performance to the states of stresses and strains within the pavement structure.

The fatigue distress function, as used in The FHWA-ARE (1) design procedure, was recommended by the Review Board to be used also as the distress function in the formulation of the FHWA-RII design method. This fatigue distress function is a field performance-oriented function and reflects the temperature cycling and the environmental conditions which the AASHO Road Test pavements experienced; therefore, this distress function is temperature-independent and incorporates the AASHO Road Test temperature cycling and environmental conditions. It should be noted, however, that the FHWA-ARE (1) distress functions which have been developed from the results of field pavement performance are quite conservative. The pavement life is calculated based on the occurrence of the first sign of Class 2 cracking. Most pavements, however, remain serviceable even after experiencing 10 to 20% cracking and therefore, the assumption of the occurrence of the first sign of cracking as a terminal state is quite conservative. It has been suggested that the FHWA-ARE (1) distress function could be shifted to correspond to higher levels of pavement cracking. Laboratory and field data have indicated that fatigue life requirements for a 100% areal cracking is several times longer than the life required for the first sign of Class 2 cracking. It should further be noted that the FHWA-ARE (1) distress functions have been developed assuming that the granular layers and subgrade have stress-independent moduli. The FHWA-ARE (1) design methodology incorporates stress correction of subgrade modulus only, and only when the test load is different from design load, but does not recognize that stresses (and consequently, layer moduli) also change as a result of changing layer thicknesses, even if gravity forces (self-weight effects) are neglected.

Since the FHWA-RII overlay design procedure is based on stress-dependent base, subbase and subgrade layers and also includes gravity forces, and since the total thickness above the subgrade varied from 5 inches (127 mm) to 31 inches (787 mm) for the pavement sections of the AASHO Road Test included in the development of the fatigue distress functions, the authors felt that consistency with the overlay design methodology required the redevelopment of the fatigue distress function incorporating the same concepts that are used in the overlay design analysis.

In summary, the FHWA-RII Flexible Pavement Overlay Design procedure utilizes a fatigue distress function developed from the AASHO Road Test data, and even though it incorporates all the stress-dependency concepts used in overlay analysis, it still incorporates the AASHO Road Test temperature regime, materials and environmental conditions. However, it should be stated that the proposed distress function could be replaced by any similar functions representing other paving materials and environmental conditions not represented by the AASHO Road Test data.

The FHWA-ARE (1) rut depth distress function was considered by the Review board as a rather weak criterion of little significance for overlay design purposes. The FHWA-ARE (1) rut depth model is based on a regression model reflecting the occurrence of rutting in the 99 overlay sections of the AASHO Road Test. The model, however, includes regression terms related to the state of stresses and strains in the subgrade. It is therefore implied that subgrade rutting contributes to the rutting of the overlaid pavement structure. However, it is considered that the rutting of a subgrade after the pavement has been overlaid is a rare phenomenon and the rutting occurs mostly in the asphalt layer*. The addition of asphalt overlays reduces the stresses and strains in the subgrade and the lower pavement layers, reducing the potential for rutting. Therefore, the Review Board considered rutting of an overlaid pavement as primarily a mix design problem, and it should be treated as such. The rutting of the asphaltic mixtures is also best represented by the VESYS G Flexible Pavement Design System (19).

*NOTE: If, the cause of surface rutting is observed to be the subgrade at any location, engineering judgment should be used to determine if reconstruction is warranted.

3.1.2 Development of the FHWA-RII Fatigue Distress Function

As stated in the previous section, the FHWA-ARE (1) fatigue distress function has been developed assuming that each of the 27 sections of the AASHO Road Test used in this development had the same base, subbase and subgrade modulus. However, the lab test data presented in Appendix B, Volume 1 of the ARE report (1), as well as reported by other authors, is clearly stress-dependent. Since the FHWA-ARE (1) design method does not consider stress-dependency of base and subbase layers, and since subgrade modulus does not have as great an effect on asphalt layer strains as the base modulus, the assumption of constant moduli from section to section is reasonably consistent with usage by ARE (1), especially when the total thickness of material above the subgrade does not vary significantly.

The FHWA-RII overlay design methodology, however, assumes that base, subbase, and subgrade layers may have stress-dependent moduli. The design method also assumes that gravity forces resulting from the self-weight of materials also contribute to stresses (see Section 3.2.2); therefore, as the layer thicknesses change, the moduli also change, and this change is not always insignificant. Table 3 is a comparison of layer moduli and critical asphalt layer strains used by ARE (1) and RII in developing the fatigue distress functions.

The lab test data presented in Appendix B of the ARE report (1) were used to develop the stress-dependent moduli equations for the base, subbase and subgrade layers. A regression analysis resulted in the following equations:

(a) AASHO Base

$$M_r = 18,300\ \theta^{.446} \tag{3-1}$$

With $R^2 = 0.996$

(b) AASHO Subbase

$$M_r = 7,820\ \theta^{.441} \tag{3-2}$$

with $R^2 = 0.81$

(c) AASHO Subgrade (with confining pressure 2.5 psi (17.2 kPa))

$$M_r = 19,800\ \sigma_d^{-1.124} \tag{3-3}$$

with $R^2 = 0.97$

where θ is the bulk stress and σ_d the deviatoric stress.

These equations, together with an asphalt modulus of 460,000 psi (3.17×10^9 kPa) and layer densities of 145 and 138 (specific gravity, 2.325 and 2.213) for the asphalt and lower layers, respectively, were used with the OAF program to compute the critical strains in the asphalt layer, ε_{xx} and ε_{yy} (Table 3). The critical strains, ε_{xx} and ε_{yy} have been computed assuming a 9 kip (40 kN) load applied to a dual wheel inflated to 75 psi (517 kPa), with tire separation of 13.1 in. (333 mm) and represent the maximum of strains at three locations: directly under the center of one tire, at the edge of one tire, and midway between the tires. ε_{xx} represents the strain in the direction perpendicular to traffic and ε_{yy}, in the direction parallel to traffic. As may be seen from Table 3, ε_{yy} is significantly greater than ε_{xx} (up to 50% for some pavement sections) and results from using dual wheel representation for the load. As was stated above, the load is represented, both in this and in ARE's analysis, by a dual wheel, i.e., two circularly loaded areas separted by 13.1 inches (333 mm) along the x-axis (perpendicular to traffic). This

TABLE 3. AASHO SECTIONS USED IN FATIGUE EQUATION DEVELOPMENT

AASHO SECT. NO.	LAYER THICKNESSES INCHES (mm)			LAYER MODULI# PSI (MPa)			MICRO STRAIN			EQUIVALENT AXLE LOADS TO FAILURE
							ARE+	RII		
	H1	H2	H3	BASE	SUBBASE	SUBGRADE	Exx	Exx	Eyy	
727*	1(25.4)	0(0.0)	4(101.6)	-	7.5(52)	1.2(8)	542	1047	1394	3
755*	1(25.4)	6(152.4)	0(0.0)	17.4(120)	-	1.8(13)	366	485	711	26
717*	1(25.4)	3(75.2)	4(101.6)	21.4(148)	7.0(48)	1.7(11)	367	514	756	45
719*	1(25.4)	6(152.4)	4(101.6)	39.9(275)	7.7(53)	2.7(19)	371	299	424	122
710	2(50.8)	3(75.2)	4(101.6)	18.0(124)	7.3(50)	2.1(14)	273	486	693	4,770
758	2(50.8)	6(152.4)	0(0.0)	16.9(117)	-	2.2(15)	249	498	627	7,250
145	4(101.6)	3(75.2)	0(0.0)	17.0(117)	-	3.2(22)	241	311	462	13,500
161	4(101.6)	6(152.4)	0(0.0)	17.6(121)	-	3.4(24)	193	266	391	24,600
583	4(101.6)	0(0.0)	4(101.6)	-	10.8(74)	3.2(22)	268	319	475	75,100
156	3(75.2)	6(152.4)	8(203.2)	35.4(244)	11.7(81)	4.2(29)	193	213	301	108,500
619	4(101.6)	0(0.0)	8(203.2)	13.8(95)	-	3.6(25)	233	264	392	129,700
111	2(50.8)	6(152.4)	8(203.2)	43.6(300)	11.4(78)	3.9(27)	235	218	310	137,700
439	5(127.0)	3(75.2)	4(101.6)	21.2(146)	11.0(76)	4.1(28)	174	209	306	149,600
473	4(101.6)	6(152.4)	4(101.6)	24.1(166)	10.1(70)	3.9(27)	174	220	322	149,600
140	4(101.6)	6(152.4)	8(203.2)	31.3(216)	12.0(82)	4.7(33)	163	186	272	200,200
260	5(127.0)	6(152.4)	8(203.2)	29.8(205)	12.6(87)	5.4(37)	139	156	231	672,000
319	5(127.0)	3(75.2)	8(203.2)	26.0(179)	12.4(85)	4.7(33)	158	182	270	672,100
575	4(101.6)	3(75.2)	12(304.8)	35.1(242)	14.8(102)	4.9(34)	175	188	279	724,700
625	4(101.6)	6(152.4)	13(330.2)	41.0(283)	15.1(104)	5.9(41)	157	156	229	724,700
261	5(127.0)	3(75.2)	12(304.8)	32.5(224)	14.9(102)	5.5(38)	148	159	237	1,122,000
297	6(152.4)	3(75.2)	8(203.2)	26.2(181)	13.1(90)	5.4(37)	135	148	220	1,418,000
325	6(152.4)	6(152.4)	8(203.2)	29.4(203)	13.3(92)	6.0(42)	120	131	194	1,419,000
336	6(152.4)	3(75.2)	12(304.8)	31.0(214)	15.1(104)	6.2(43)	127	133	200	1,589,000
445	5(127.0)	6(152.4)	12(304.8)	36.1(249)	14.7(102)	6.2(43)	134	139	206	1,833,000
477	4(101.6)	9(228.6)	12(304.8)	44.8(309)	14.9(103)	7.0(48)	148	139	203	1,833,000
427	5(127.0)	9(228.6)	12(304.8)	41.2(284)	15.2(104)	7.1(49)	125	122	182	1,963,000
333	6(152.4)	9(228.6)	16(406.4)	41.2(284)	16.6(115)	8.5(58)	104	101	153	3,862,000

#ASPHALT LAYER MODULUS 460,000 psi (3172 MPa) for all sections
+LAYER Moduli used by ARE for all sections: AC 460,000 psi (3172 MPa), Base 40,000 psi (276 MPa), Subbase 20,000 psi (138 MPa), Subgrade 5,000 psi (34 MPa)
*Omitted from fatigue equations - inconsistent with other data

representation causes greater bending about the x-axis (ε_{yy} strain) than about the y-axis (ε_{xx} strain) because the load is more concentrated along the y-axis; the load is distributed over a width 2a along the y-axis and over a width 2a + TS along the x-axis (where a is the radius of the loaded area and TS the tire separation). The weighted 18 kip (80 kN) axle load application prior to Class 2 cracking (22) in Table 3 (from Table 1 of the ARE report (1))and ε_{yy} were used to develop the following fatigue distress function:

$$N_f = 7.56 \times 10^{-12} \ (1/\varepsilon)^{4.68} \qquad (3\text{-}4)$$

with an $R^2 = 0.93$

The distress function published in the ARE report (1) is

$$N_f = 9.73 \times 10^{-15} \ (1/\varepsilon)^{5.16} \qquad (3\text{-}5)$$

however, a regression equation using the data from Table 1 of the ARE report (1) results in

$$N_f = 1.28 \times 10^{-18} \ (1/\varepsilon)^{6.19} \qquad (3\text{-}6)$$

with an R^2 of 0.80. Although this equation looks different in form from equation 3-5, the N_f's computed from the two equations are very close to each other.

A comparison of the correlation coefficients shows that the RII distress function (equation 3-4) developed using the procedures detailed in this report fits the AASHO data significantly better then does the ARE procedure (1). There are two primary reasons for this:

1). The formation of cracks depends on the maximum strain ε_{yy}; since ε_{xx} and ε_{yy} differ by a factor that is dependent on pavement layer thickness and stiffnesses, a poorer correlation would be expected when ε_{xx} is used instead,

2). The total thickness of material above the subgrade varied from 7 to 31 inches (128 to 787 mm), and the base and subbase thicknesses varied from 0 to 9 inches (0-229 mm) and 0 - 16 inches (0 - 406 mm), respectively, for the AASHO sections used in developing the distress functions. These variations in layer thicknesses result in significant changes in the state of stress existing within these layers. Therefore the assumption (used by ARE) that all pavement sections had the same base, subbase and subgrade moduli independent of pavement geometry is inconsistent with lab data. This assumption has not been made in developing the RII distress function.

It should be emphasized that the use of the RII distress function with theories deriving tensile strains from principles different from those stated in this manual is invalid.

As indicated in Table 3, the four pavement sections having a 1 inch (25.4 mm) asphalt layer have been omitted from the above regression equation (the same sections were deleted from the ARE equation) because the N_f for these sections is entirely inconsistent with the rest of the data. It is dubious whether failure resulting in three, or even 122 load applications can be considered to be fatigue failure; furthermore, the vertical strains in the base and subgrade layers are significantly higher for these sections than for the remaining sections. It is therefore possible that base/subgrade failure and/or excessive rutting contributed to the early failure observed in these pavements.

3.1.3 Pavement Testing Period

Although it is desirable that pavement evaluations be carried out at a most critical period of a year, it is believed, however, that such a field evaluation might lead to an overly conservative design. Secondly, the execution of field evaluation programs at most critical periods are quite impractical. Since these periods are often so short in duration, any statewide field investigation becomes difficult and almost impossible. Therefore, it is recommended that field measurements be conducted in a time period where statewide field evaluation is not only convenient but also when base and subgrade support values have stabilized and represent, as an approximation, average annual conditions.

The design procedure is based on measurements made during the time of the year when base and subgrade support values represent, as an approximation, average annual condition. Measurements made at other times (for example, worst condition encountered after spring thaw or best condition in late summer/early fall) may be accomodated through the use of a "seasonal factor" applied to the allowable axle loadings. This manual offers a guide (see Table 3 and Section 2.4.3, Volume 2 (2)) to the seasonal factor, but the user is encouraged to develop his own factor using procedures outlined in Appendix A, Volume 2 (2) of this report. Measurements should not be made during the part of the year when any section of the pavement structure is in a frozen condition. It is also recommended that measurements not be taken during summer afternoons when average asphalt temperature exceeds about 100°F (38°C), especially for pavements with granular bases. Layer theory assumes that the materials are isotropic, i.e., that the modulus in bending is the same as the compressive modulus. This, however, is not the case for most asphalt mixes at higher temperature - as the temperature increases, the bending modulus decreases faster than the compressive modulus. The deflection directly under the load is a function of compressive modulus, but deflections away from the load depend on the load transfer efficiency of the pavement layer, or the bending modulus. This difference in moduli (bending and compressive) leads to nonlinear behavior, with measured deflections being higher under the load and lower away from the load than would be expected from isotropic materials.

3.1.4 Environment

The influence of environment is reflected in the pavement response at various environmental conditions. To adjust the pavement measurements to those of the most critical period, regional factors have been developed for various climatic conditions. The regional factor is a multiplier to the traffic intensity which replaces the damage at a weakened environmental condition by an equivalent increase in the traffic-induced damage at a reference environmental condition.

The damage due to environmental conditions is induced by the temperature, moisture, water table, drainage, and especially, freeze and thaw conditions in the spring.

The adjustment for the pavement temperature is achieved by means of a Temperature Adjustment Factor, and the measurements of mean air temperature. This adjustment is applied to the asphalt layer modulus rather than to measured deflections, as is done by most other design schemes.

The accumulation of moisture in the base and subgrade similarly affects the pavement performance. It is recognized that moisture in the subgrade generally changes toward equilibrium conditions with time. Therefore, most pavements requiring overlay have often reached this equilibrium condition in the subgrade and base course.

Drainage problems are also a consideration for environmental effects. The remedial solution for drainage problems, however, often includes significant reconstruction which could only become a cost-effective alternative if the required overlay is relatively thick.

3.1.5 Remaining Life Analysis

The pavement overlays, except for those to improve the riding quality, are constructed on mildly-cracked pavements in most instances. The present design procedures, such as FHWA-ARE (1) or any other known procedure, do not recognize the effect of cracking in the underlying layers on the life expectancy of the overlay. In a brute strength method, such as in the FHWA-ARE (1) model, the effect of existing cracked pavement is incorporated by assigning a lower modulus to the underlying pavement layers. That is, an existing cracked pavement is replaced by an equivalent layer of lower modulus, and therefore affects the state of stresses and strains in the pavement structure.

For the remaining life analysis of existing uncracked pavements, it could be shown that the addition of an overlay increases the remaining life by reducing the induced strains at the bottom of the existing pavement. However, as the life of the existing pavement becomes used up, and the remaining life is reduced to about 10 percent or less, it is logical to assume that the existing pavement structure will become cracked, and the strains at the underside of the overlay need to be considered as a criterion for fatigue life calculations.

It was recommended that the FHWA-ARE (1) remaining life analysis be incorporated into the "standard" design. Although this procedure is a conservative approach for uncracked existing pavements, no other rational alternatives appear to be simple enough for consideration in this "standard" design.

3.2 FRAMEWORK OF PROCEDURES AND PANEL RECOMMENDATIONS

The guidelines for the formulation of a "standard" method, as was discussed in the previous sections, resulted in the proposed standard overlay design procedure with the following major features:

(a) elastic layer theory is used in the analysis model;

(b) existing pavement is characterized by three layers;

(c) effective in-situ modulus concept is used;

(d) in-situ pavement conditions are evaluated from nondestructive dynamic deflection measurements;

(e) asphalt layer stiffness is corrected from measurement temperature to design temperature;

(f) base, subbase and subgrade stiffnesses are adjusted for stress dependency from testing stresses to design stresses and include gravity effects as well as changes in stresses due to changes in pavement geometry;

(g) the remaining life is determined from critical strains using the fatigue relationship developed by RII from AASHO Road Test data (equations 3-4);

(h) environmental effects are considered by modifying design traffic through a *regional factor;*

(i) a guide for the development of a seasonal factor is presented, allowing field testing to be conducted during most of the year;

(j) the design procedure is completely computerized in a single program.

This procedure computes the required overlay thickness based on fatigue criteria alone. It does not consider rutting for reasons discussed previously, nor does it address the question of *reflection cracking.*

The analysis procedure and the program, OAF, are described in detail in subsequent chapters. The required input information and the generation of this data are described fully in Volume 2 (2) of this report, along with output format and a guide to output interpretation.

3.2.1 Existing Pavement Evaluation

As was indicated in Section 2.4, the FHWA-ARE (1) procedure utilizing laboratory testing to characterize existing pavement layers was considered unimplementable because most user agencies are not staffed nor have the sophisticated facilities needed to conduct the extensive laboratory testing necessary for this method of analysis. It should also be noted that the simplification of the FHWA-ARE (1) procedure, in which default values are used for the pavement layers and subgrade moduli are computed from field deflection measurements, is somehow subjective and is dependent on the visual observations of in-situ conditions. For such reasons, it was recommended that estimation of in-situ conditions be based on more rational procedures, such as the procedures developed utilizing measured pavement surface deflections and the shape of the deflection basin to evaluate the in-situ layer moduli. Such a method adopted as a "standard" procedure has the advantage that field testing is fast, simple and non-destructive, and that laboratory testing is no longer required. It is also based on sound engineering principles rather than on subjective methods.

In accordance with the panel's recommendation, the evaluation procedure for the "standard" method is based on measured pavement deflections of a three-layer or a four-layer representation of the pavement structure. The three-layer analysis is tried first, and if no three-layer solution is found, the four-layer analysis is used. Although the FHWA-ARE (1) procedure uses a four-layer model to characterize an existing pavement, it was felt that assuming a pavement system to be represented by three layers reduces the difficulty of determining layer moduli from measured deflections. In principle, up to five layer properties could be determined from five measured deflections. The calculation of layer moduli from surface deflections, however, requires the simultaneous solution of sets of non-linear equations, which becomes much more complex and time-consuming as the number of variables increases.

The three-layer analysis model, which will be described more fully in Chapter 4, is based on elastic layer theory and assumes that the pavement is made up of an asphalt layer, a base layer (base and subbase are combined into one layer), and a semi-infinite subgrade layer. The layer stiffnesses are determined from measured deflections by varying layer moduli, computing the resulting surface deflections from these moduli, and repeating the entire procedure until the computed deflections match the measured deflections.

In this "standard" design procedure, the entire overlay design methodology is computerized, permitting all the necessary computations, such as determination of the layer moduli from measured deflections, the temperature and stress corrections, determination of the remaining life, and the amount of overlay required, to be accomplished in one program.

The design procedure is based on the effective modulus concept rather than the effective thickness approach because the effective thickness concept requires a priori knowledge of the asphalt and base layer moduli, and the computation methods required to solve for effective thicknesses are much more complex. Of course, the effective modulus approach requires that layer thicknesses be known; however, these are usually available from construction records.

3.2.2 Stress Correction

Commercially available pavement deflection testers generally apply loads that are smaller than the load resulting from an 18 kip (80 kN) axle. Because most subgrade materials and granular bases have moduli that depend on applied stresses, it is necessary to adjust the moduli determined from deflection measurements to design conditions, i.e., apply a stress correction. The FHWA-ARE (1) design model includes a stress correction for subgrade but not for the base or subbase layers. The FHWA-ARE (1) method uses nomographs and an iterative procedure to adjust the subgrade modulus to design load; trying to use similar nomographs to correct both base and subgrade moduli for stress effects becomes quite tedious, since the stresses in the base are affected by the subgrade modulus and vice versa. This consideration further influenced the decision to computerize the design procedure.

3.2.3 Testing Equipment

The design procedure requires the evaluation of layer stiffness for three or four layers, requiring testing equipment capable of measuring pavement surface deflections at a minimum of four locations from the applied load. While the theory used is perfectly general and applicable to any loading or deflection measuring geometry, various subroutines have to be tailored to specific testing devices in order to reduce the required computer time. The panel recommended that the program be written for the Dynaflect geometry because a recent survey indicates that many more State Departments of Transportation have this type of equipment than any other. To avoid showing any preferance the program has been written for the Dynaflect, as well as the Road Rater, and both the trailer-mounted and van-mounted Falling Weight Deflectometers, and it is possible to modify the program for any other loading or deflection measuring configuration, provided that the above requirements are met.

3.2.4 Deflection Data Analysis

Most deflection-based overlay design methods use statistical techniques to combine deflections measured at different locations within a section to arrive at a design deflection that represents the overlay project. These methods assume that a linear relationship exists between maximum deflection and overlay thickness, i.e., that deflection alone governs the amount of overlay needed. The proposed "standard" design procedure, however, evaluates the conditions of each layer in the pavement and determines the overlay requirement based on these values.

Other than for large increases in user traffic, a pavement may require an overlay because the asphalt, base or subgrade layer is weak, or possibly because all three are weak in any one location. The amount of overlay required to correct each of these deficiencies is different and each condition has a different influence on deflection magnitude and the shape of the deflection basin. The subgrade support value has a great effect on the magnitude of measured deflections but a much smaller effect on the shape of the deflection basin. On the other hand, the stiffness of the asphalt layer has a greater effect on the shape of the deflection basin than on deflection magnitude. Furthermore, these relationships are very non-linear.

It is therefore apparent that combining deflection measurements from different locations within the overlay project to form representative deflections and using these values in the analysis model may result in an evaluation that is not representative of any part of the project. The use of an arithmetic mean of the deflection (average W1 through W5 for the section) for analysis will probably result in an overlay thickness that approximates the average design thickness required for that section, but this will, in general, be an underdesign in something like 50% of the cases. A scheme that computes a representative deflection profile using

$$W_{id} = \bar{W}_i + BSD_i \tag{3-7}$$

where *i* refers to the i^{th} sensor, $\bar{W}_i$ is the average value of the i^{th} sensor, SD_i is the standard deviation of the i^{th} sensor, and B is a constant depending on the desired confidence level is a step in the right direction, since it will result in a thicker overlay. A B of 2 is generally associated with a 97% confidence level; however, the use of 2 in the above equation does not necessarily result in an overlay thickness that is adequate in 97% of the cases, since a very complex and non-linear relationship relates deflections to overlay requirements.

If deflection data at each test location are analyzed separately, the in-situ stiffnesses at these locations are determined along with the required overlay thickness. This allows the user to divide the overlay project into sections having similar overlay needs and vary the overlay thickness accordingly. Since the amount of overlay required depends not only on the maximum deflection but also on the shape of the deflection basin, such delineation is almost impossible from deflection data alone.

Areas requiring thick overlays also readily stand out; alternative rehabilitation strategies such as reconstruction or recycling may become economically desirable. Furthermore, since in-situ layer stiffnesses are available at each test location, areas with reduced base or subgrade stiffnesses may benefit from improved drainage, thus reducing the amount of overlay needed.

Based on the above consideration, it was recommended that deflection data at each test location be analyzed separately. The panel also recommended that the percentile concept rather than standard deviation be used in determining project overlay needs.

CHAPTER 4
DESIGN CRITERIA AND PROCEDURES DESCRIPTION

4.1 SCOPE OF PROCEDURES

An existing pavement may require overlay for various reasons, such as fatigue cracking, rutting, low skid resistance, surface deterioration in the form of spalling and ravelling, etc. The design procedure described herein is not intended to establish maintenance and repair needs and priorities, or to predict the future need of an overlay; rather, it attempts to determine the required overlay thickness after the decision to overlay is made. The selection of an overlay thickness is made based on a fatigue cracking model developed from the AASHO Road Test data tempered by experience and other studies. The procedure does not consider rutting as a distress mechanism, primarily because rutting of overlaid pavements in the United States is most often associated with unstable paving mixes; nor is it intended to consider localized distress modes due to expansive soils or severe environmental stresses. The question of reflection cracking is addressed only indirectly in that badly cracked existing pavements which are most likely to develop reflection cracking generally require very thick overlays or, as an alternative, a total or partial reconstruction or recycling may be more economical, thus minimizing the potential for reflection cracking.

Since the fatigue model is based primarily on AASHO Road Test data, this procedure infers that the overlay materials and construction specifications will not significantly differ from those now in use. However, overlays using reinforcing fabric and improved paving mixtures such as sulfur and latex-modified asphaltic materials can be accomodated by appropriate modification of the distress function.

Although several design options are presented, the primary design procedure is based on determination of in-situ pavement layer properties from non-destructive deflection measurements on existing pavements. The procedure determines the overlay thickness requirement for each test location; consequently, the areas requiring significantly thicker overlays readily stand out, enabling the designer to consider partial or total reconstruction, recycling or other remedial measures such as drainage improvement prior to overlay. The designer can vary the overlay thickness along the highway as required by field conditions and also select an appropriate reliability level for design.

Figure 4 is a simplif..ed flow chart showing the steps and decisions used in the program. Along with the basic procedure, four other options are offered:

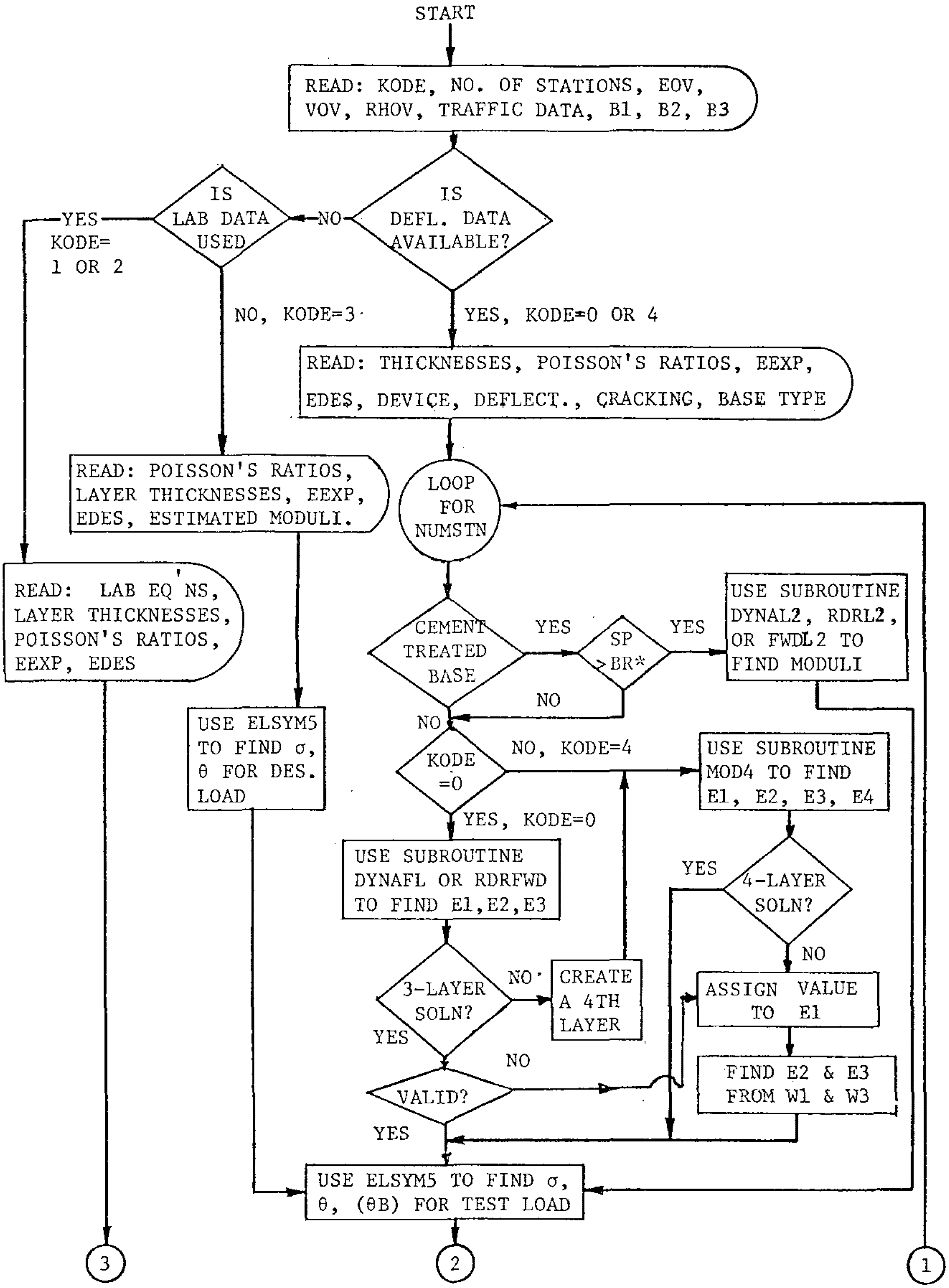


FIGURE 4. Simplified Flow Chart of OAF

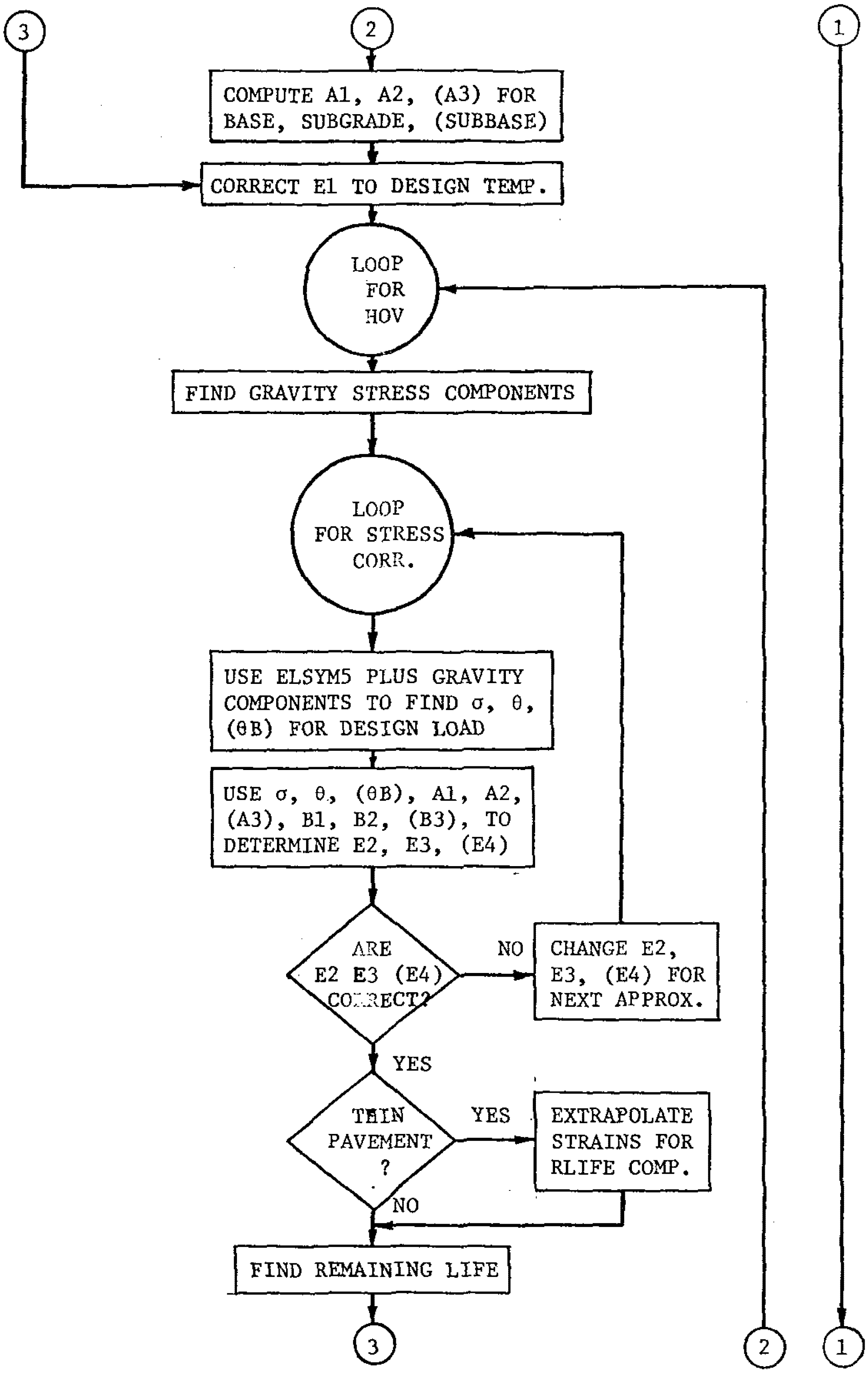


FIGURE 4. Simplified Flow Chart of OAF (continued)

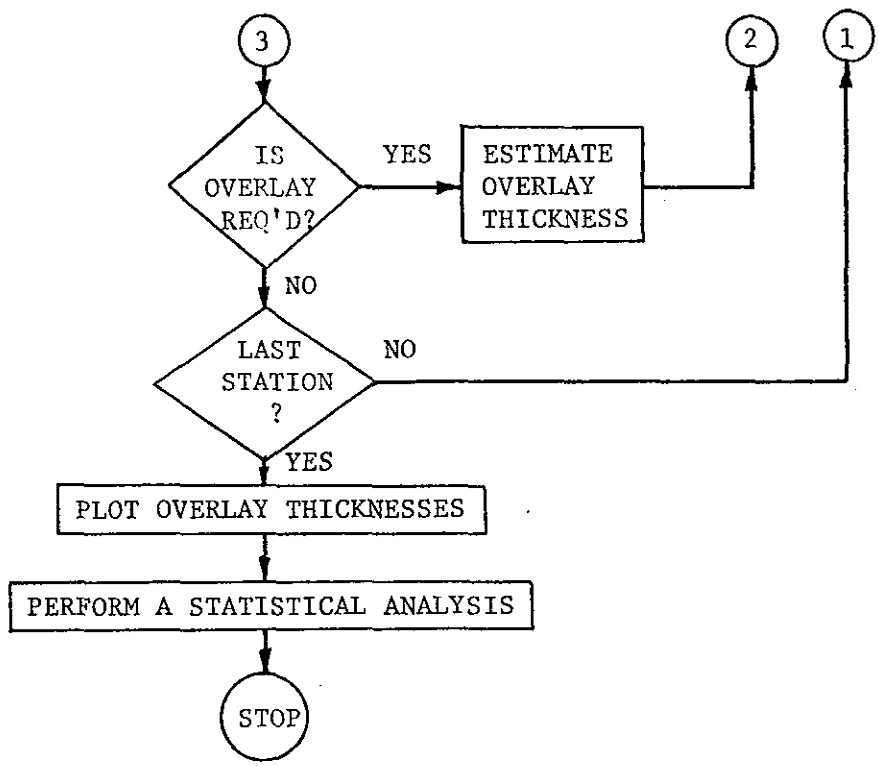


FIGURE 4. Simplified Flow Chart of OAF (continued)

A. The use of laboratory-determined layer properties in a three-layer pavement system

B. The use of laboratory-determined layer properties in a four-layer pavement system

C. The use of estimated layer properties (default values) in a three-layer pavement system

D. The use of dynamic deflection measurements in a four-layer pavement system.

The options A and B for the use of data obtained by laboratory testing are intended to serve the purpose of diagnosis and verification for those areas where laboratory test data for layer moduli is (or becomes) available. Option C, using default values, is included primarily for design purposes and may be used for the evaluation of the relative effectiveness of various base or pavement materials, and option D is offered so that the user may bypass the three-layer approximation used in the basic procedure. It should be pointed out, however, that this option is considerably more expensive and requires more computer time than any of the other options.

The computational procedures are completely computerized with the overlay thicknesses plotted in a printer graphic format as a function of locations along the roadway. On the basis of this test plot, the designer can then divide the overlay project into various sections with similar overlay requirements. A separate small computer program is used to determine the statistical significance of this division if required by the engineer.

4.2 ANALYSIS MODEL

The overlay design scheme presented in this report is based on elastic layer theory which characterizes an existing pavement as a semi-infinite layered half-space consisting of a number of homogeneous and isotropic layers with full friction between layers. These layers are assumed to be uniform in thickness and to extend to infinity in both horizontal directions. In addition, the bottom layer is assumed to extend to infinity vertically. The layer properties are represented by modulus of elasticity, Poisson's ratio and layer density, and the load is assumed to be distributed uniformly over a circular area with no horizontal or shear forces applied. A dual tire truck wheel is represented by two such circular areas with center to center distance the same as that for a typical truck wheel.

Most pavement analysis schemes use linear elasticity theory which neglects body forces, i.e., forces due to gravity, since it is assumed that the stress-strain relaitonship for paving materials is linear. However, when the layer moduli are stress dependent, as shown in equations 3-1 through 3-3, the gravity forces can no longer be neglected. Therefore, the analysis model also includes the effects of gravity forces when computing stress-dependent moduli.

Although paving materials are, in general not elastic and pavement widths fall far short of infinite extent, there is reasonable agreement between measured and computed deflection (18, 20, 21), especially when the layer theory is used in conjunction with the concept of dynamic modulus.

The layer theory makes it possible to compute stresses, strains and displacements in a pavement structure when the elastic and geometric properties of the layers are known. It is also possible, at least in principle, to compute the elastic and geometric properties of the structure from measured pavement surface deflections. The latter problem is somewhat complicated by the fact that the material properties are nonlinear, inhomogeneous transcendental equations of the measured deflections, and more than one solution may exist.

4.3 PROGRAM DESCRIPTION

The overlay design procedure has been fully computerized in the program, OAF, which utilizes seven separate programs for the required computation: ELSYM5 (17), DYNAFL, DYNAL2, RDRFWD, RDRL2, FWDL2 and MOD4. ELSYM5 (17), developed by researchers at the University of California, Berkeley, is a general one to five-layer program that is capable of handling up to nine uniformly loaded circular areas, and it is the program used to compute the stresses and strains in the original and overlayed pavement. DYNAL2 and DYNAFL are two and three-layer programs, respectively, which compute the layer moduli from measured surface deflection and the shape of the deflection basin, and have been designed specifically for the Dynaflect loading and deflection measuring geometry. RDRFWD is equivalent to the 3-layer DYNAFL for Road Rater and FWD, RDRL2 and FWDL2 are two-layer versions of DYNAL2, and MOD4 computes four-layer moduli for all three testing devices. DYNAL2 and RDRL2 were originally developed by the authors at Ohio State University and refined by RII for this application. The other programs were developed at RII expressly for this project.

The overlay analysis program offers, in addition to the basic procedure, four other options or modes of analysis. The type of analysis used is determined by the variable KODE. The basic analysis method uses in-situ evaluation of layer stiffness at each test location from measured dynamic deflections, and is specified by KODE=0 or KODE=4. KODE=0 is the basic procedure and characterizes an existing pavement as a three-layer structure. The program attempts to find stiffnesses of the layers that match the measured deflections and deflection basin, but if it fails to find a solution to the three-layer problem, it characterizes the pavement as a four-layer structure and finds moduli for these layers. KODE=4 bypasses the three-layer analysis and goes straight to the four-layer model; however, the four-layer model is considerably more expensive to use. In addition, the program can also accept the layer properties as determined by laboratory tests, or using default values. When complete laboratory test data is available, including base, subbase and subgrade stress-dependent properties (as defined by equations 4-1 though 4-6), KODE=1 or 2 is used. If estimated values (estimated layer moduli for 18 kip (80 kN) axle loading) are used, KODE=3 is specified.

The options specified by KODE=1 or 2 for the use of data obtained by laboratory testing are intended to serve the purposes of diagnosis and verification for those areas where laboratory test data for layer moduli is (or becomes) available. KODE=3, using estimated values, is included primarily for design purposes and may be used to evaluate the relative effectiveness of various base or pavement materials.

KODE=0 and KODE=4 are useable with four types of testing devices, specified by DEVICE

(a) Dynaflect, DEVICE=1
(b) Road Rater, DEVICE=2
(c) Falling Weight Deflectometer, trailer-mounted, DEVICE=3
(d) Falling Weight Deflectometer, van-mounted, DEVICE=4

The steps used in the overlay design procedure are shown schematically in the flow chart in Figure 5. This flow chart has been simplified and condensed for the convenience of the user - the actual flow of information is quite complex and would require many pages to describe all the branches and switchbacks.

The following major steps (which will be described in subsequent sections) are used in the analysis program:

1. Determine layer properties

   (a) from measured surface deflections, KODE=0 or KODE=4

FIGURE 5. Flow Chart of OAF.

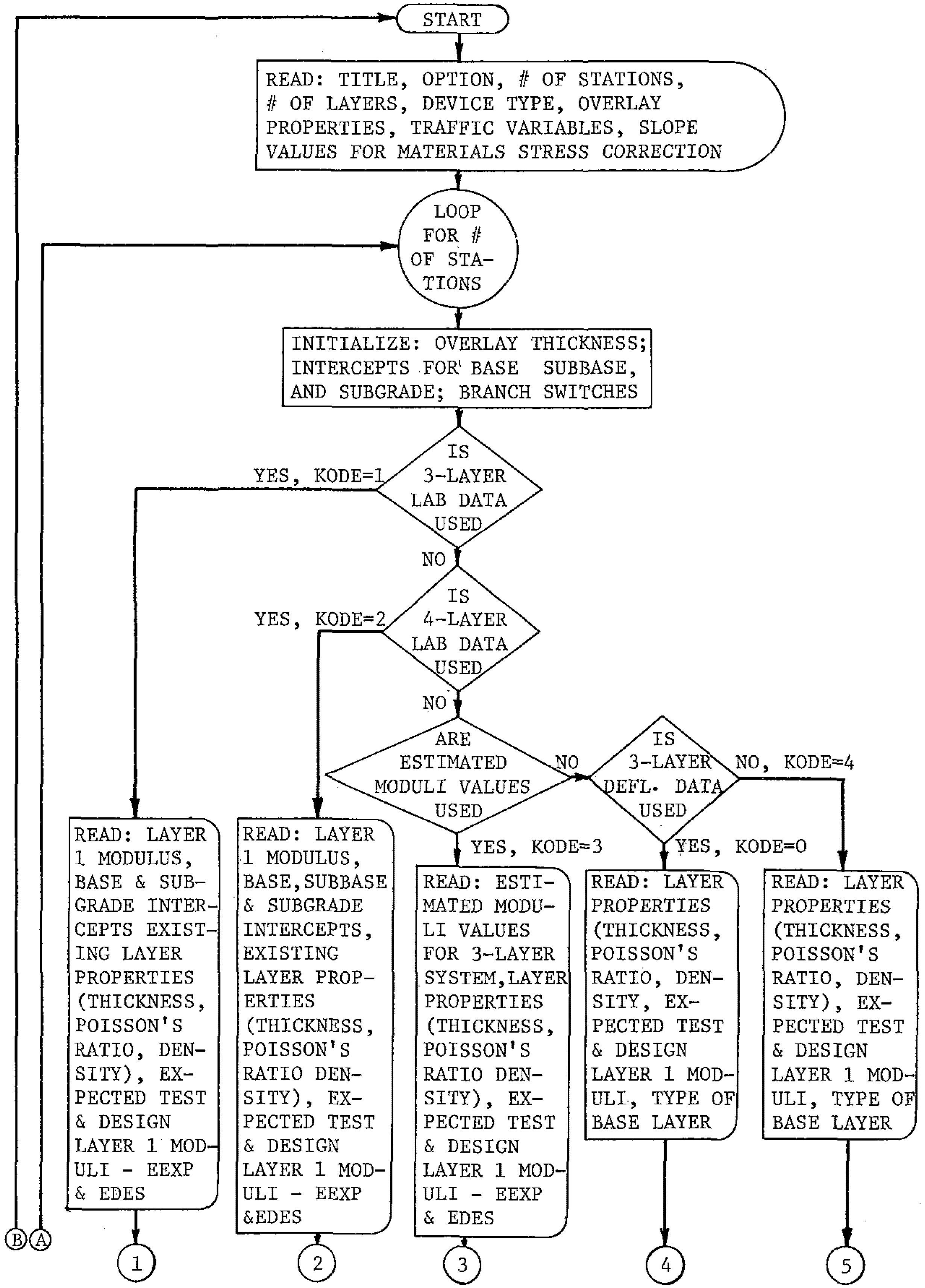


FIGURE 5. Flow Chart of OAF (continued)

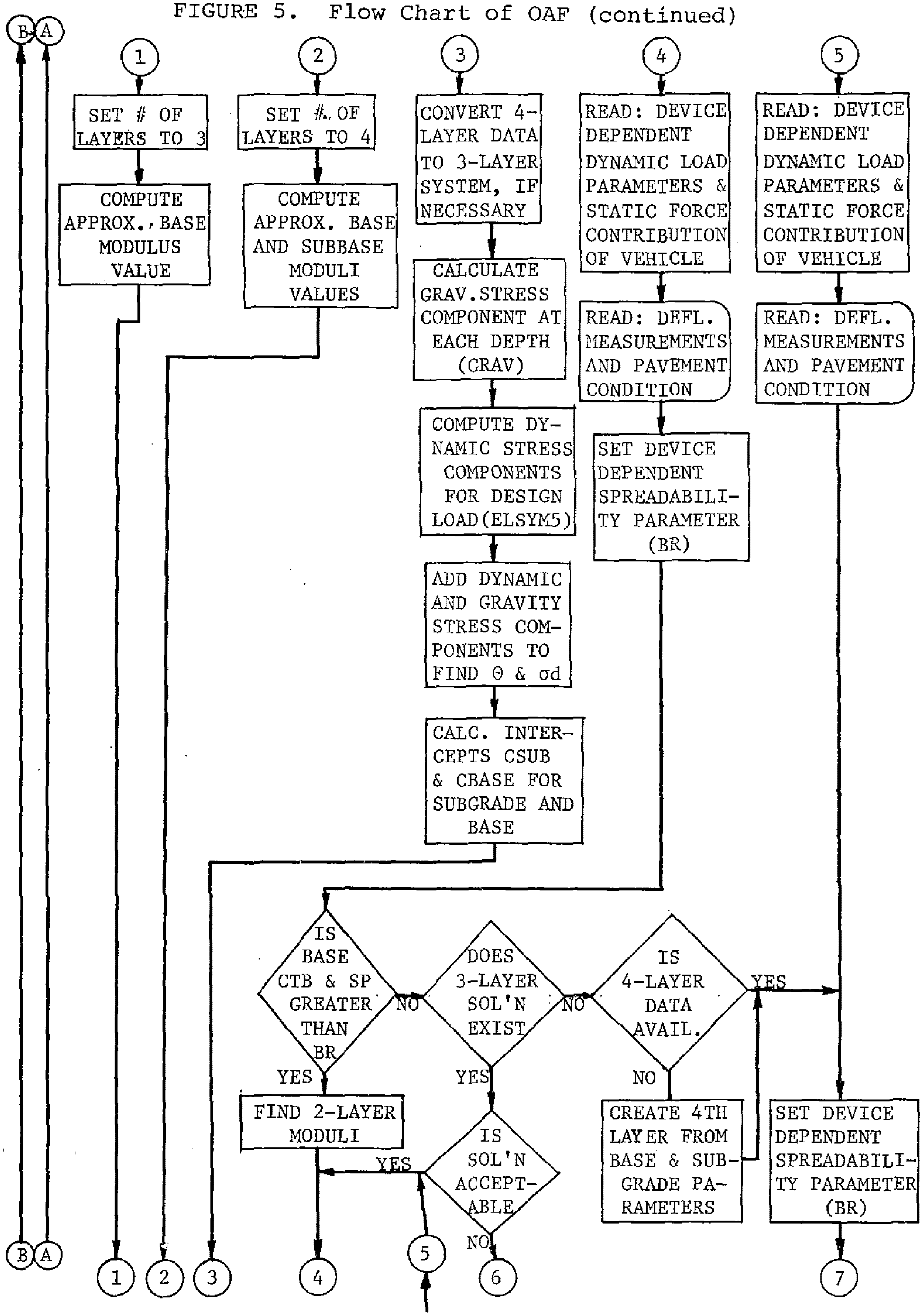


FIGURE 5. Flow Chart of OAF (continued)

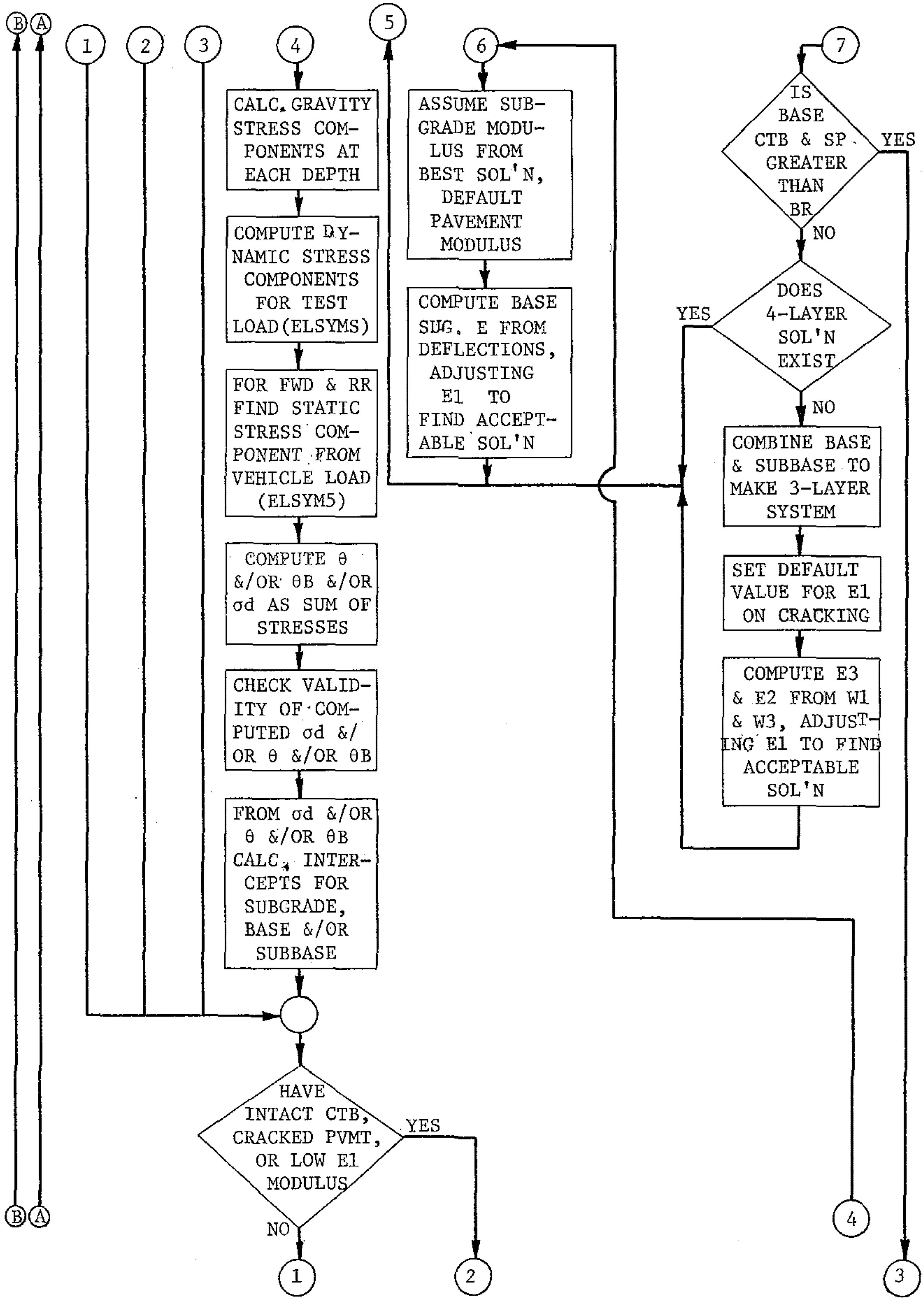


FIGURE 5. Flow Chart of OAF (continued)

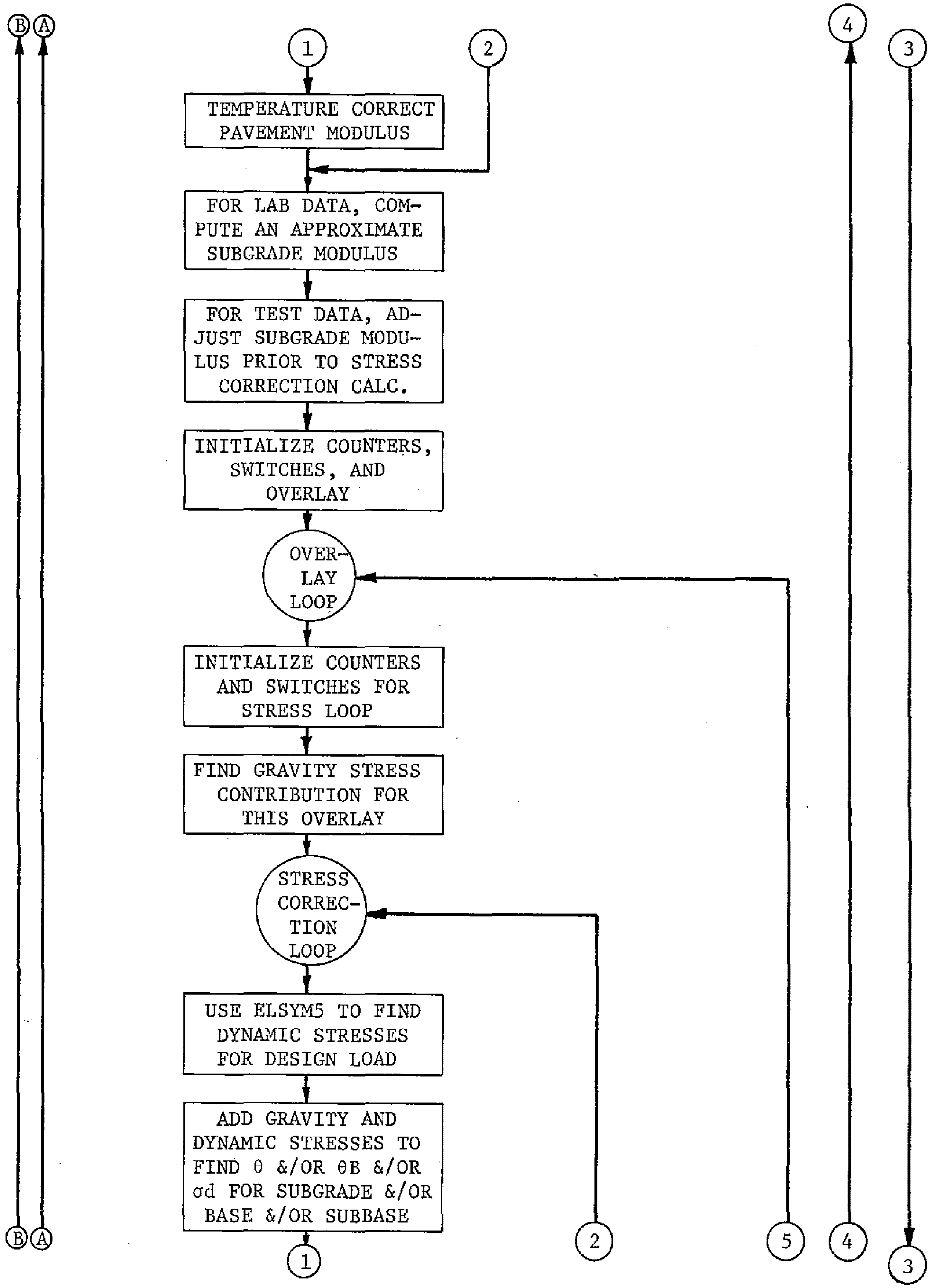


FIGURE 5. Flow Chart of OAF (continued)

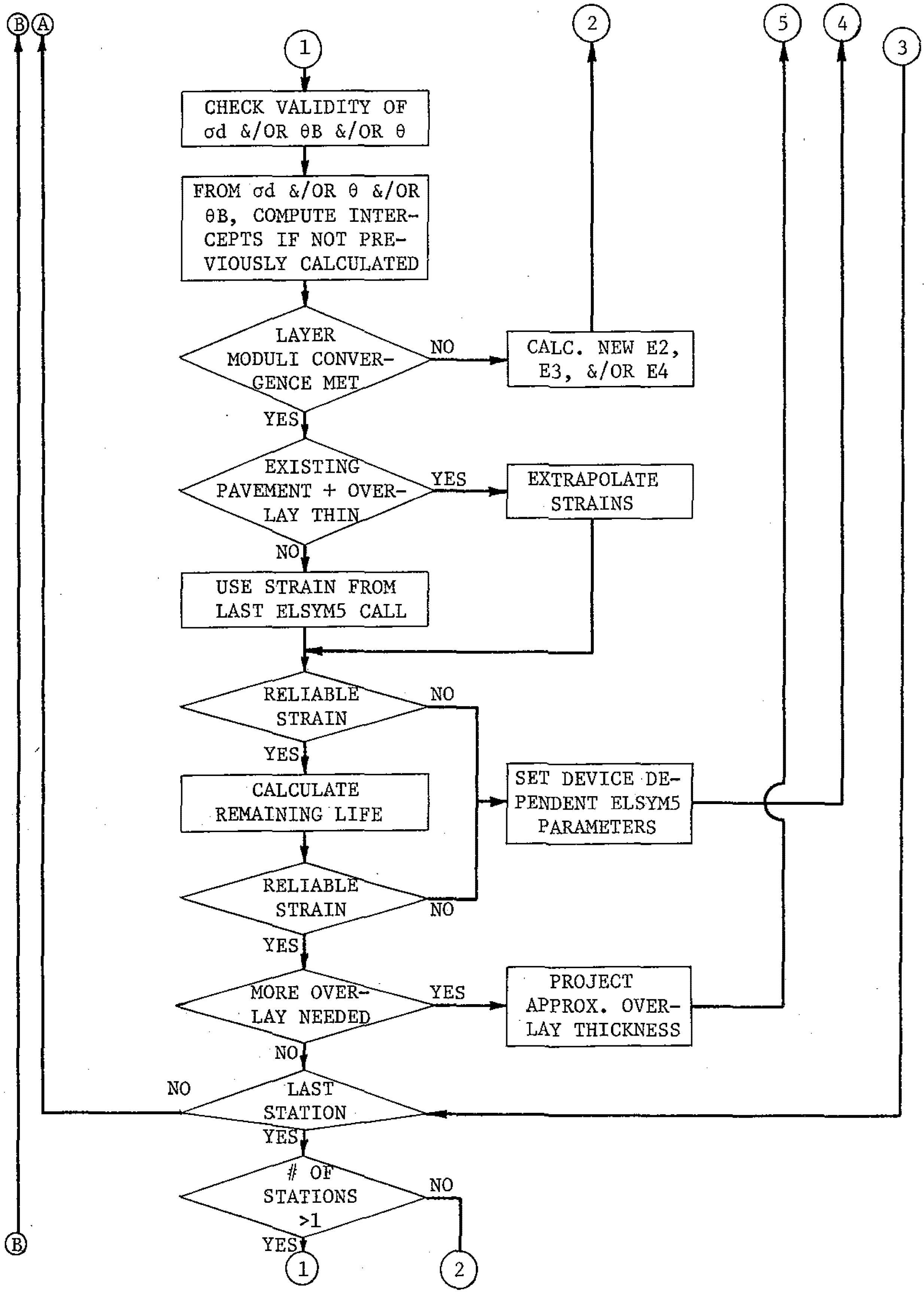
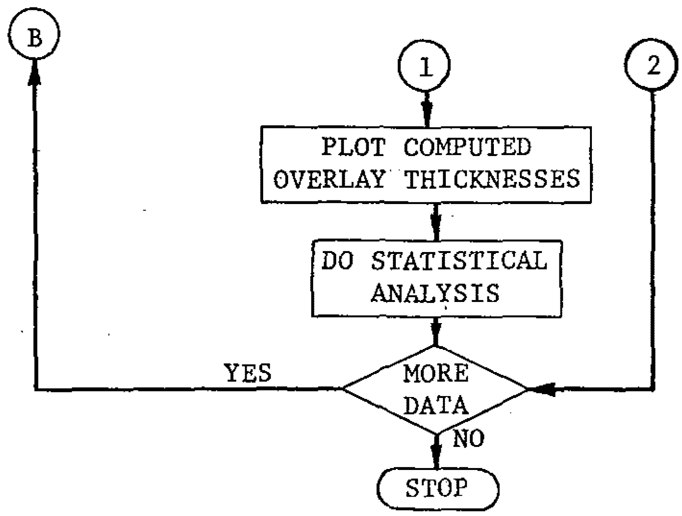


FIGURE 5. Flow Chart of OAF (continued)



(b) from laboratory test data,
KODE=1 or 2

(c) from estimated (default) values,
KODE=3

2. Adjust layer stiffnesses for temperature and stress dependency effect.

3. Compute remaining life, RLIFE.

4. Increment overlay thickness (HOV), readjust layer stiffnesses for changed stress states and recompute remaining life.

Step 4 is used and repeated until the remaining life (RLIFE) is within 25% of the design life.

Steps 1 through 4 are repeated for each test location. After the last overlay calculation, the required thicknesses are plotted as a function of test location and a statistical analysis performed. The statistical analysis computes the average overlay thickness along with the 67, 77, 87 and 97 percentile values. The program does not attempt to group the overlay project into sections having similar overlay needs - this is left for the design engineer.

Once the user has delineated his sections, he may wish to test the statistical significance of this division using a separate small program, TVAL. Appendix D, Volume 2 (2) describes the input requirements for this program.

4.3.1 Determination of Layer Properties From Measured Surface Deflections (KODE=0 or 4, DEVICE=1, 2, 3 or 4)

The following input data (described in Chapters 2 and 3, Volume 2 (2)) is required for this mode of analysis:

(a) Surface deflection measurements

(b) Base type, i.e., granular or cement-treated

(c) Layer thicknesses

(d) Poisson's ratio of all layers

(e) Modulus of pavement asphalt at test temperature and design temperature, EEXP and EDES

(f) Modulus of overlay asphalt, EOV

(g) Whether Class 2 or Class 3 (22) cracking exists.

The Dynaflect applies a 1000 lb. (4.4 kN) load to the pavement through two four-inch (102 mm) wide steel wheels covered with hard rubber tires, which are spaced 20 inches (508 mm) on center. Measurements made on a variety of surfaces show that except for soft subgrade, the total contact area remains approximately constant with a value of 5 sq. in. 323 mm^2), giving an equivalent circular area of radius 1.26 inches (32 mm). Since the wheel separation is much greater than this radius, the superposition principle is used to represent the Dynaflect loading by a 200 psi (1.38 MPa) uniformly distributed load over a single circular area with 1.26 inches (32 mm) radius.

The Road Rater applies a variable load to the pavement through two 4 x 7.1 inch (101.6 x 108.3 mm) steel columns, 10.7 inches (271.8 mm) center to center. The load generated by the Road Rater is a function of the frequency (FR) and amplitude (AMP) which are selectable by the operator. As in the Dynaflect case, superposition principle is used to represent the Road Rater loading by a single load distributed over a single circular area with 3.067 inch (77.9 mm) radius.

Two types of Falling Weight Deflectometer exist: van-mounted and trailer-mounted. Both apply a load over a single circular area with radius 5.9 inches (150 mm), so superposition is not needed.

The existing pavement is represented by either a three-layer elastic half-space composed of an asphalt layer, a base layer, and a semi-infinite subgrade layer, or a four-layer elastic half-space composed of an asphalt layer, a base layer, a subbase layer, and a semi-infinite subgrade layer. The layer thicknesses and Poisson's ratios are assumed to be known, and the layer stiffnesses are computed from measured surface deflections.

Since some pavements are composed of more than three layers, it is necessary to combine some of the existing layers into one layer to fit the three-layer model. The following scheme is used:

Layer 1 - H_1 will be equal to the sum of all existing asphalt layer thicknesses and will include surface course, leveling course, asphaltic concrete or asphalt-treated base, and previous overlays, if any.

Layer 2 - If the existing pavement is constructed with:

a) Granular base and subbase, H_2 will be the sum of base and subbase thicknesses.

b) Cement-treated base on lime stabilized subbase, H_2 will be the sum of base and subbase thicknesses.

c) Cement-treated base on granular subbase, H_2 will be the base thickness.

d) Granular base on lime-stabilized subbase, H_2 will be the granular base thickness.

This thickness assignment is done internally in the program; the user reads in the layer thicknesses as they actually exist.

In the event that an existing pavement is composed of only three layers, i.e., no subbase, and a four-layer analysis is needed (KODE=0) or requested (KODE=4), the program creates an artificial subbase layer but with stress-independent modulus, and does the following thickness assignments:

(a) Base layer thickness equal to two-thirds of the as-built base thickness;

(b) Subbase layer thickness equal to one-third of the as-built base thickness plus 6 inches (152 mm).

The artificial subbase layer is created because often existing bases become contaminated with, and intrude into the subgrade layer, creating a layer that is a mixture of soil and granular material, and since granular materials and soils generally have opposite stress-dependency relationships, stress-independent modulus is assumed.

While the creation of the subbase layer is somewhat arbitrary, it is felt that the resulting analysis has more validity than the default mode described later in this section.

As can be seen from the flow chart in Figure 5, pavements with cement-treated bases are analyzed somewhat differently from pavements with granular bases. For pavements with granular bases, subroutine DYNAFL or RDRWD, depending on DEVICE type, is used to compute the in-situ layer stiffnesses from measured deflections and the shape of the deflection basin, as defined by the spreadability, SP:

$$SP = 100 * \left(\sum_{i=1}^{N} W_i \right) / NW_1 \qquad (4\text{-}1)$$

where W_i are the measured deflections, W_1 is the maximum deflection, and N is the number of sensors.

When layer moduli are used to compute the surface deflections, the results are unique; i.e., specifying layer properties results in a specific deflection value at a particular distance from the load. This, however, is not the case for the inverse procedure of computing the moduli from surface deflections, since various combinations of layer moduli can result in the same deflection. Up to four solutions, or combinations of layer moduli, may satisfy the deflection requirements for a three-layer model and up to eight may exist for a four-layer model, but some of these may represent unrealistic conditions from an engineering viewpoint. The program does not attempt to find all the possible solutions; instead, care has been taken to assure that the solution found is acceptable from engineering conditions. However, it should be kept in mind that the analytical model assumes that the layer thicknesses are constant throughout the project. If the as-built thicknesses vary from the assumed (design) thicknesses, this variation is reflected not only in the effective layer stiffnesses, but also in the shape of the deflection basin, or the degree of correspondence between measured and computed deflections.

The analysis method also assumes that the measured deflections are precise; however, the readout resolution of the Dynaflect meter is, at best, $\pm$ .01 mils (.00025 mm) for deflection values below 1 mil (.025 mm), and $\pm$ .03 mils (.0008 mm) for deflections between 1 and 3 mils (.025 and .08 mm)(other testing devices have similar resolution ranges). This resolution problem, coupled with operator-induced errors, contributes to the discrepancy between the two sets of deflections, and may alter the effective layer 1 and layer 2 stiffnesses significantly, particularly for pavements with relatively low (0.5 to 0.6 mils (.013 to .015 mm)) maximum deflections.

The analysis model (layer theory) assumes that the surface layer is continuous, i.e., capable of transmitting horizontal stresses. This is not the case when cracks occur in the area occupied by the loading wheels and deflection measuring sensors. Depending on their size and tightness, the presence of cracks can have significant effect on the effective layer stiffnesses.

Because of the above consideration, two basic strategies are used to determine layer stiffnesses for the three-layer model:

(a) matching measured and computed W1, W2, SP

(b) matching measured and computed W1, W3, SP

If both schemes result in a solution (these will not always be the same, particularly because of the limited resolution in deflection measurements), the one that produces E1 (effective layer 1 modulus) closest to EEXP is chosen. Some measured deflection data, however, will be inconsistent with the three-layer analysis model; in that case, the program tries to find a solution by relaxing the convergence criterion consistent with the expected errors resulting from readout resolution. If this approach also fails, a four-layer analysis is tried. In this case, if the existing pavement was not constructed with a subbase, this layer is created using the scheme outlined previously in this section.

Normal convergence is achieved if the measured and completed deflections differ by less than 3×10^{-6} inches (7.6×10^{-5} mm) and spreadabilities by less than 0.15. When the convergence criteria are relaxed, these values change to 1.5×10^{-6} inches (3.8×10^{-4} mm) and 0.75, respectively. The latter values are consistent with the readout resolution of the testing devices considered in OAF.

As was stated earlier, the solutions found are not unique i.e., up to four solutions may exist; and when the resolution and reproducibility of deflection measurements are considered, many more solutions are possible. Since the layer properties are complex nonlinear functions of the measured deflections, a great deal of computer time would be required to find all possible solutions. Therefore, instead of finding all possible solutions, the moduli subroutines attempt to find solutions that realistically represent the pavement condition. However, some solutions may still be unrealistic; consequently, a check is made to determine if the solution is acceptable.

The following criteria are used in judging the acceptability of a solution:

(a) for a pavement with Class 2 or 3 cracking (22), the computed asphalt stiffness must be at least 70,000 psi (482 MPa), and the computed base stiffness must not exceed 65% of the computed asphalt stiffness.

(b) for an uncracked pavement, the temperature corrected asphalt stiffness E(1) must be at least 100,000 psi (689 MPa), and the computed base stiffness must not exceed 65% of the temperature corrected asphalt stiffness.

The above criteria have been chosen based on several considerations. Firstly, an uncracked existing pavement should require less overlay than a cracked pavement. Secondly, the modulus of most asphalt mixes approaches asymptotically a constant value of approximately 80,000 to 100,000 psi (552 to 689 MPa) with increasing temperature (see Figure 5). Thirdly, for small values of existing asphalt layer modulus, the critical strain of an overlaid pavement increases with increasing A.C. modulus before starting to decrease, leading to the erroneous conclusion that an existing pavement with stiffer asphalt modulus requires more overlay than a weaker pavement. While the value of the A.C. modulus for maximum strain varies somewhat with pavement geometry and layer properties, the critical strain is at near maximum for 70,000 psi (689 MPa). Fourthly, and this refers to the criterion for base stiffness, if the base modulus approaches the (temperature corrected - for uncracked pavements) asphalt stiffness, the neutral axis in the pavement structure approaches the layer 1-2 interface, resulting in unrealistically low critical tensile strains and unreasonable long remaining life. This is discussed more fully in Section 4.3.5.

If a solution has been rejected, the following strategy is used to find an acceptable solution:

(a) The asphalt layer modulus is set at the minimum acceptable value (from criteria (a) and (b) above) if the solution was rejected because the computed E1 was too low, or

(b) the computed asphalt layer modulus is increased by 20% if the solution was rejected because the base stiffness exceeded the allowable value, and

(c) the base and subgrade moduli are determined iteratively by matching the measured and computed W1 and W3 (first and third sensor readings), with the constraints that the subgrade modulus is not allowed to exceed 1,000,000 psi (6890 MPa), and the base modulus is not allowed to exceed the smaller of two values: 0.65* E(1) or 1500,000 psi (1034 MPa). The constraints have been selected to be consistent both with engineering judgment and the criteria used in judging the acceptability of a solution.

If the solution requires that the constraints have to be exceeded, the asphalt layer stiffness is increased, even for cracked pavements, and the iterative procedure repeated until a staisfactory solution is found. The use of this strategy is identified with the message "Default Option--3-Layer Solution Unacceptable".

The layer stiffnesses in the four-layer and analytical model are determined by matching measured and computed W1, W2, W3 and SP. In general, more than one combination of layer stiffnesses will match the above parameters; consequently, the program attempts to find several solutions by starting the iteration process from eighteen different locations in the solution space. Since this is a time-consuming process, the attempt is stopped either when a solution is found with E1 within 30% of EEXP and with a base stiffer than the subbase, or when the number of solutions exceeds nine. As in the three-layer programs, any solution that produces computed deflection parameters that match measured values consistent with readout resolution is accepted. All solutions (layer stiffness values) found are printed out; however, only the solution that results in E1 closest to EEXP and with a base stiffer than the subbase is used in the overlay determination, provided that E1 meets the criteria for acceptability discussed earlier. If E1 fails to meet the acceptability criteria, then the requirement that the base be stiffer than the subbase is dropped, and the solution with E1 closest to EEXP is used.

Although, the four-layer model results in solutions most of the time, some measured deflection data will also be inconsistent with a four-layer analytical model and no solutions result. In this case, the program prints out the message "Default Option -- No 4-Layer Solution" and a default model is used.

This default model is very similar to the one used when an unacceptable three-layer solution results, except that the iteration process is started with assigning to the asphalt layer a stiffness of EEXP or 70,000 psi (482 MPa), depending on whether the asphalt layer is uncracked or has Class 2 or Class 3 cracking (22), respectively. Again, base and subgrade stiffnesses are computed by matching W1 and W3, with the same constraints discussed previously.

The default models used in this procedure are somewhat more complicated than the model proposed by ARE (1) where the asphalt and base layers are assigned values and the subgrade modulus determined by matching measured and computed maximum deflections; however, the ARE model (1) generally results in very poor agreement between the measured and computed deflection basin shapes. Since the determination of layer properties in this procedure is based on matching the maximum deflections as well as the deflection basin shape, the default models attempt to be consistent with these concepts without undue increase in computation time.

For pavements with intact cement-treated bases, the critical strain occurs in the bottom of the base layer rather than in the asphalt layer. However, at the present time, no satisfactory distress equation exists relating tensile strain in the base layer to the number of 18 kip (80 kN) axle loads to failure. In order to overcome this difficulty, these pavements are characterized by a two-layer analytical model composed of an equivalent full-depth asphalt layer with thickness H1 + H2 resting directly on subgrade. Subroutine DYNAL2, RDRL2 or FWD2 (depending on testing device) is used to determine the effective asphalt and subgrade layer stiffness using a scheme that matches measured and computed spreadability (SP) and maximum deflection (W1) values.

When the base layer is already cracked, the critical strain moves upward toward the bottom of the asphalt layer; the program treats this case as if the base were composed of granular material, but with a stress-independent modulus.

Dynaflect measurements made on pavements with relatively intact cement-treated bases result in spreadability values in excess of 55%. This value (55%) is used to differentiate between cracked and uncracked bases and the type of analysis used, i.e., two-layer evaluation or three-layer evaluation. The branch value for Road Rater or FWD measurements is 50%. Since very little field data taken with a Road Rater or FWD, was available to the authors, the 50% branch value was derived theoretically based on Dynaflect measurements and the differences in loading and measurement geometry of these devices.

After the layer stiffnesses have been determined, ELSYM5 is used to compute the stresses σ_d and θ for the testing device load so that the constants A1, A2 and A3 can be determined from

$$A1 = EB\ \theta^{-B1} \tag{4-2}$$

$$A2 = ES\ \sigma_d^{\ B2} \tag{4-3}$$

$$A3 = ESB\ \theta^{-B3} \tag{4-4}$$

provided that the bulk stresses are greater than 1 psi (7 kPa), and from

$$A1 = EB/(.99 + .01\theta) \tag{4-5}$$

$$A3 = ESB/(.99 + .01\theta) \tag{4-6}$$

if either of the bulk stresses is less than 1 psi (7 kPa). The reasons for the above choices are discussed in Section 4.3.4 of this volume.

Since the measured surface deflections are dynamic deflections, they are functions of only the dynamic load applied by the testing device. This, however, is not true of the stresses σ_d and θ. All the testing devices considered in this procedure apply a static load in addition to the dynamic load; also, the Road Rater and the Falling Weight Deflectometer have the vehicle/trailer wheels close to the point of load application. Additionally, overburden pressure (self-weight of the layers) contributes to σ_d and θ. It is therefore necessary to include all factors when computing stresses.

As we stated earlier, the Dynaflect applies a $\pm$ 500 lb. (2.2 kN) dynamic load to the pavement through its loading wheels. In addition to this dynamic load, the trailer also exerts a 2000 lb. (8.9 kN) static load through the same wheels. Consequently, the load applied to the pavement when the deflection is maximum is not 1000 lb. (4.4 kN), but is instead 2500 lb. (11.1 kN), or the sum of static and dynamic loads. Also, the tow vehicle exerts a static load on the pavement; however, since the vehicle wheel is in excess of 9 ft. (2.7m) from the point of dynamic load application, the vehicle contribution can be neglected. Although the static load applied by the Dynaflect is constant and invariant among Dynaflects, this parameter is part of the input data in order to make the input requirements similar for all testing devices. In this correction, it should be noted that the dynamic deflections represent peak-to-peak measurements, i.e., the result of a 1000 lb. (4.4 kN) load. Therefore, the static load is specified as 1500 lb. (6.7 kN), or the difference between the maximum load and total dynamic load.

The static loads applied by the Road Rater and the FWD are somewhat more complicated -- not only are the van/trailer wheels sufficiently close so that their contribution cannot be neglected, both the static loads applied by the loading head and the vehicle wheels are a function of the dynamic load. Furthermore, the loading due to the van wheels varies from vehicle to vehicle, depending on load distribution and amount of additional weight in the vehicle. Therefore, these parameters are part of the input data and require some careful weight measurements by user agencies.

The effect of self-weight of layers (gravity forces) is generally neglected by linear elasticity theory since it is assumed that stress-strain relationship is linear, i.e., independent of stress. But when layer moduli are stress-dependent (equations 4-18 through 4-22), these forces can no longer be neglected. The vertical pressure exerted at a depth Z (in the nth layer) by the materials above this plane is given by

$$\sigma_z = \frac{1}{1728} \left(\sum_{i=1}^{n-1} \rho_i H_i + \rho_n \left(Z - \sum_{i=1}^{n-1} H_i \right) \right) \qquad (4\text{-}7)$$

where

ρ_i = density of the ith layer in pcf

H_i = thickness of the ith layer

n = layer number in which z is located

If the materials are homogenous and isotropic (an assumption of layer theory), σ_z is uniform in all horizontal directions and varies with depth only. This stress therefore results in a vertical strain ε_z but ε_x and ε_y are both zero (no bending moment).

The general relationship between stress and strain under this loading is given by

$$\varepsilon_x = \frac{1}{E} (\sigma_x - \nu (\sigma_z + \sigma_y)) = 0 \tag{4-8}$$

$$\varepsilon_y = \frac{1}{E} (\sigma_y - \nu (\sigma_z + \sigma_x)) = 0 \tag{4-9}$$

where ν is Poisson's ratio. These equations may be solved for σ_x and σ_y with the following results

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu} \sigma_z \tag{4-10}$$

Granular materials are generally tested in the laboratory under a triaxial state of stress, i.e., a cylindrical specimen is subjected to hydrostatic confining pressure and a dynamic vertical stress. The modulus of resilience M_r is computed from measured vertical dynamic strain and M_r is related to the bulk stress θ by

$$M_r = A1(\sigma_z + 2\sigma_r)^{B1} = A1\theta^{B1} \tag{4-11}$$

i.e., the constants A1 and B1 are determined. Here σ_r is the hydrostatic confining pressure and σ_z is the total vertical stress, but σ_z generally does not include gravity forces. However, the body forces add approximately 0.3 psi (2 kPa) to the vertical stress (for an 8 inch (203 mm) high cylinder) and θ is generally greater than 10 psi (68.9 kPa) so that the omission is negligible. Cohesive soils are also tested in the laboratory under a triaxial state of stress similar to that used for granualr materials, and the modulus determined similarly. But, instead of relating M_r to θ, a correlation is made between M_r and σ_d, as shown by equation 4-20. In the laboratory tests, σ_d is defined as

$$\sigma_d = \sigma_z - \sigma_r \tag{4-12}$$

again, body forces are neglected in the laboratory analysis. In this case, however, the omission may be of greater significance since some researchers report data for σ_d from 1 or 2 psi to 20 to 30 psi (7 or 14 to 13.8 to 276 kPa).

In a layered system, the bulk stress is invariant under coordinate transformations, consequently whether θ is defined in terms of

$$\theta = \sigma_x + \sigma_y + \sigma_z \tag{4-13}$$

or
$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \tag{4-14}$$

(where σ_x, σ_y, σ_z are stress in the x, y, z directions and

σ_1, σ_2, σ_3 are principal stress)

θ has the same value. This is, however, not true for σ_d, and there is not universal agreement on the definition of σ_d. Some authors define σ_d as

$$\sigma_d = \sigma_z - 1/3\ (\sigma_x + \sigma_y + \sigma_z) \tag{4-15}$$

while others define it as

$$\sigma_d = \sigma_3 - \sigma_1 \tag{4-16}$$

where σ_3 is the minor principal stress and

σ_1 is the major principal stress.

There is also a discrepancy in the definition used by the various layer programs that consider stress-dependent subgrade moduli.

It should be noted that in a laboratory test, the deviator stress is the difference between total vertical stress and the applied hydrostatic confining pressure. Therefore, if the results of laboratory tests are to be applied to layer theory, a definition for σ_d should be consistent with laboratory data. In the layer case the total vertical stress is given by σ_z but the horizontal confining pressure varies with direction, i.e., $\sigma_x \neq \sigma_y$. In this analysis σ_d is defined as

$$\sigma_d = \sigma_z - 1/2\ (\sigma_x + \sigma_y) \tag{4-17}$$

which is identical to Equation 4-12 for the lab case.

In summary, this section deals with the determination of in-situ layer properties from NDT deflection measurements. The basic analysis model assumes that the pavement can be modeled as a three-layer structure, but when the deflection data is inconsistent with this model, or when so specified by the user, a four-layer model is used. Pavements with intact cement-treated bases are analyzed using a two-layer model and cracked cement-treated bases are assumed to act as granular bases, but with stress-independent moduli. The layer properties are back-calculated from layer theory using the measured surface deflections, as well as the shape of the deflection basin.

Considerable care has been taken to insure that the solutions found are realistic from an engineering view, even though the solutions are not unique.

Granular materials and subgrade soils are assumed to have moduli that depend on the state of stress existing in the pavement structure, and that this state of stress is the result of static and dynamic forces applied by the testing devices, as well as gravity forces arising from the self-weight of layers. Inherent in this procedure is also the assumption that the materials in the pavement structure behave similarly to materials in laboratory tests, i.e., that the slopes of the temperature and stress relationships remain the same.

4.3.2 Determination of Layer Properties from Laboratory Test Data, KODE=1 or 2

When complete laboratory test data is available for materials characterization, the initial sections of the program are bypassed. The required data are read in and the program proceeds directly to Step 2 (Section 4.3). References to laboratory testing methods are presented in Appendix B of this report.

In this mode, the pavement is characterized either by a three-layer or a four-layer analytical model, depending on whether a subbase exists. The load applied by an 18 kip (80 kN) axle is approximated by two uniformly loaded circular areas which represent one half of the dual tire axle; the contribution of the other wheel is assumed to be negligible because of the distance separating the wheels. In this representation, a 4.5 kip (20 kN) vertical load is uniformly distributed over each circular area having a radius of 4.37 in. (111 mm), resulting in 75 psi (517 kPa) contact pressure. These areas are spaced 13.17 inches (335 mm) center-to-center, corresponding to the spacing between tires of an average truck wheel.

The layers in the three-layer model are defined in Section 4.3.1; the four-layer model is composed of the following layers:

(1) Asphalt layer with thickness H1 as defined in Section 4.3.1

(2) Base layer with thickness H2 equal to base thickness

(3) Subbase layer with thickness H3 equal to subbase thickness

(4) Subgrade layer with semi-infinite thickness.

In addition to the layer thicknesses, the following information is required for input:

(a) Poisson's ratio and density of all layers

(b) Modulus of overlay asphalt at design temperature, EOV

(c) Modulus of existing asphalt at design temperature, EDES

(d) Constants A1 and B1 to determine the base modulus from

$$E_B = A1\ \theta^{B1} \tag{4-18}$$

or from

$$E_B = A1(.99 + .01\theta) \tag{4-19}$$

when θ is less than 1 psi (7 kPa)

(e) Constants A2 and B2 to determine the subgrade modulus from

$$E_S = A2\ \sigma_d^{B2} \tag{4-20}$$

(f) If subbase is used, the constants A3 and B3 to determine the subbase modulus from

$$E_{SB} = A3\ \theta^{B3} \tag{4-21}$$

or from

$$E_{SB} = A3(.99 + .01\theta) \tag{4-22}$$

when θ is less than 1 psi (7 kPa)

In the above equations, θ is the bulk stress (sum of the principal stresses) and σ_d is the deviatoric stress.

4.3.3 Determination of Layer Properties from Estimated Values, KODE=3

When neither deflection measurements nor complete laboratory test data is available, the user may estimate the layer moduli based on experience and published data. In this mode, the pavement is characterized by the three-layer analytical model. In addition to layer thicknesses, the following input information is required:

(a) Poisson's ratio and density of all layers

(b) Modulus of overlay asphalt at design temperature, EOV

(c) Modulus of existing asphalt at design temperature, E(1)

(d) Base modulus for 18 Kip (80 kN) axle loading, E(2)

(e) The constant B1, equation 4-18

(f) Subgrade modulus for 18 Kip (80 kN) axle loading, E(3)

(g) The constant B2, equation 4-20.

After the above information has been read in, ELSYM5 is used to determine σ_d and θ for 18 Kip (80 kN) axle loading, the constants A1 and A2 are determined from equations 4-2 and 4-3, and the program skips to Step 3 (Section 4.3).

4.3.4 Temperature and Stress Correction, Step 2

In this step, the layer 1 stiffness is adjusted from measurement temperature to design temperature (if lab data or estimated values are specified at design temperature, no correction is needed), using the following relationship:

$$E(1) = E1 * EDES/EEXP \qquad (4-22)$$

where

E(1) = effective asphalt stiffness at design temperature

E1 = effective asphalt stiffness at test temperature

EDES = original modulus of paving asphalt at design temperature

EEXP = original modulus of paving asphalt at test temperature

which assumes that the effective (in-situ) asphalt stiffness has a temperature dependency that is parallel to that of the original paving mix when it was new.

The temperature correction procedure is bypassed if the existing pavement has Class 2 or 3 cracking (22); it is assumed that a cracked pavement will exhibit only small changes in stiffness with temperature.

Following temperature correction of layer 1, the moduli of base and subgrade are computed for the state of stress existing in the pavement under an 18 kip (80 kN) axle load at the design temperature. In this case gravity forces are included, but no additional static loads are present. Since the base, subbase and subgrade moduli depend on existing stress conditions which, in turn, depend on the layer moduli, an iterative procedure is used in the stress compensation.

The pavement is assumed to be composed of three (or four layers for KODE=0, 2, 4) layers and the 18 kip (80 kN) axle is approximated by two uniformly loaded circular areas as discussed in Section 4.3.2. The stresses σ_d and θ are computed at three horizontal locations: at the center (c) and edge (e) of the circular areas, and halfway between (m) the two load circles. The deviatoric stresses are computed at the top of the subgrade layer at the above horizontal locations and combined as shown in equation 4-23 to form an average σ_d:

$$\sigma_d = (2\sigma_c + 2\sigma_e + \sigma_m) \qquad (4\text{-}23)$$

where the subscripts c, e, m refer to the above horizontal locations.

The sum of the principal stresses is computed at the top, middle and bottom of the base layer at the same horizontal locations and averaged in the following manner:

$$\theta = (2\theta_{1c} + 2\theta_{1e} + \theta_{1m} + 2\theta_{2c} + 2\theta_{2e} + \theta_{2m} + 2\theta_{3c} + 2\theta_{3e} + \theta_{3m})/15 \qquad (4\text{-}24)$$

where the subscripts 1, 2, 3 refer to the three vertical locations and c, e, m have the same meaning as before.

If a subbase layer is analyzed separately (KODE=0, 2, 4), the subbase stress correction is identical to the one used for the base layer, except that the vertical location for stress computations are at the top, middle and bottom of the subbase layer.

The following iterative procedure is used:

(a) Initial values for the base (subbase, if present) and subgrade stiffnesses are assumed. If dynamic deflection data is used, the initial values correspond to the test device loading otherwise. values are assigned.

(b) ELSYM5 (17) is used to compute σ_d and θ.

(c) Projected base (subbase, if present) and subgrade stiffnesses are computed from equations 4-18 or 4-19, (4-21 or 4-22), and 4-20 using the above stresses.

(d) A modified Newton-Raphson procedure (26) (tailored for this problem) is used to determine new values for E_B (E_{SB}) and E_S.

Steps (b), (c) and (d) are repeated until the base (subbase) and subgrade moduli determined in Steps (c) and (d) have converged. The convergence criteria used are 2% for base, 3.5% for subbase, and 5% for subgrade, although the subbase and subgrade moduli are generally much closer when the base has converged. The relatively tight convergence criterion for the base modulus is necessary to insure consistent behavior with overlays, since relatively small changes in base modulus can have a significant effect on the critical strain.

One of the shortcomings of layer theory is that it predicts tensile stresses (θ) in the base and/or subbase layer for some combinations of layer moduli and thicknesses, particularly when the load radius is greater than layer 1 thickness. When this occurs, equations 4-18 and 4-21 have no real solutions. Also, as θ becomes small (less than 1 psi (7 kPa) the moduli values decrease rapidly, leading to very high computed deflections and strains. Various approaches have been suggested on how to deal with this problem, none of which is entirely satisfactory. The approach used in PDMAP (23) of setting σ_x and σ_y to zero whenever they become negative solves both of the above problems but is theoretically unjustifiable. This approach also creates additional difficulties: the computed base/subbase moduli are quite high, and the effectiveness (layer equivalency) of a base under a thin pavement is substantially greater than under a thick pavement. Both results are contrary to common sense and current pavement design practices; furthermore, the application of this concept to the AASHO sections (Table 3) leads to highly erratic results, with a correlation coefficient (R^2) of only 0.65.

The approach used in this procedure is to assume that the base/subbase moduli behave according to equation 4-18/4-21 for θ greater than 1 psi (7 kPa) and according to equation 4-19/ 4-22 below that value, i.e., that the base/subbase moduli are essentially stress independent below 1 psi (7 kPa). The assumption of a 1% slope (equation 4-19 and 4-22) is necessary so that the intercepts A1 and A3 (equation 4-3 and 4-6) can be computed if θ becomes negative under test load. The choice of 1 psi (7 kPa) is somewhat arbitrary; it was chosen based on two considerations:

(a) to keep the minimum value of the base/subbase modulus at a reasonable value, and

(b) the rate of decrease of EB/ESB (equation 4-18/4-21) becomes quite steep below this value.

While various objections may be raised to this approach, it does have the advantages that it results in moduli values that appear reasonable, the base layer effectiveness is lower for thin pavements than for thick pavements, and the application of this concept to the AASHO data (Table 3) results in a correlation coefficient of 0.94 for the fatigue equation (3-4). A better solution to the tensile stress problem may be to redevelop layer theory using boundary conditions that prevent σ_x and σ_y from becoming negative in layers that cannot sustain tension; however, the success of this undertaking is not assured and it is considerably beyond the scope of this project.

4.3.5 Remaining Life Determination, Step 3

The design procedure is based on the "effective stiffness" concept in which the existing pavement is characterized by layers having as-built thicknesses but with altered (reduced) layer stiffnesses, i.e., a "new" pavement with in-situ layer stiffnesses as determined from dynamic testing. In the analysis model used, the new pavement is treated as having all of its life remaining; consequently, estimation of previous traffic experience is not necessary.

The remaining life of uncracked pavements having in-situ stiffness greater than 70,000 psi (482 MPa) (1) (the value used to characterize asphalt layers having Class 2 (22) cracking) is determined by computing the horizontal tensile strain (ε) at the bottom of the existing asphalt layer (with ELSYM5) (17) and using this strain to compute N_f (number of equivalent 18 kip (80 kN) axle loadings to failure, or remaining life) from:

$$N_f = 7.56 \times 10^{-12} (1/\varepsilon)^{4.68} \qquad (4\text{-}25)$$

Pavements that exhibit Class 2 or Class 3 (22) cracking or have in-situ stiffness (at the test point) less than 70,000 psi (482 MPa) are assumed to have failed, i.e., are no longer able to withstand additional tensile strains but act as base material providing additional support to the overlay layer. The critical strain in these cases is at the bottom of the overlay; consequently, the horizontal tensile strain (ε) is calculated at that point. However, because of the limitations of layer theory, this strain may be lower than the strain at the bottom of the original pavement, leading to the conclusion that a thinner overlay is required as the pavement condition deteriorates (as the stiffness of the existing asphalt layer decreases), which is inconsistent with experience. In order to overcome this difficulty, the strain is computed at both locations (under the overlay and under the existing pavement) and the maximum of the two is used. Equation 4-25 is again used to compute the remaining life.

In the above remaining life calculations, the horizontal strain is computed at three horizontal locations: directly under the center of one load, at the edge of the load, and midway between the two loads, and the maximum of these three strains in the direction parallel to the direction of traffic (ε_{yy}) is used.

When the asphalt layer is thin (the definition "thin" is a function of layer stiffnesses and lower layer thicknesses and may vary from 2 to 5 inches (51 to 127 mm) layer theory predicts that the critical strain at the bottom of the asphalt layer decreases as the asphalt layer thickness decreases, leading to the conclusion that a thin pavement may have a higher fatigue life than a thicker pavement, which is contrary to experience. It has been found, however, that the relationship between log strain and layer 1 thickness is linear for thicker pavements. Therefore, in order to overcome the difficulty with thin pavements, the program checks to see if the particular pavement stiffnesses and geometry place the pavement into the "thin" category, and if so, extrapolates the strain by computing ε_{yy} under two thicker pavements and projects this value for the "thin" pavement. The extrapolated strain is compared with the value determined normally, and the larger of the two values is chosen as the design strain. The use of this extrapolation scheme may occur when computing remaining life of the original pavement, but should be encountered rarely when an overlay is present.

As was stated earlier in Section 4.3.1, when the base and/or the subgrade modulus approaches the asphalt layer modulus, the neutral axis in the pavement structure approaches the asphalt-base interface, resulting in low or compressive critical strains and unreasonably high remaining lives. These pavements may in fact have a lower remaining life with a relatively thin overlay (on the order of 1-2 inches (25-51 mm)) than with no overlay. This problem is generally not encountered for new or relatively new pavements since the base modulus is at least five times lower than the asphalt modulus. With the "effective modulus" concept, however, the ratio of asphalt to base modulus may approach unity for older and/or cracked pavements. It is for this reason that the constraints and acceptability criteria (discussed in Section 4.3.1) were developed.

When layers having stress-dependent moduli are used in the analysis, this "unreliable strain" problem becomes even more critical. Although the constraints and acceptability criteria have been developed for stress-dependent layers, it is not always possible to anticipate the extent of change in layer moduli from design load to test road. Therefore a check is made on the reliability of the remaining life calculations whenever the computed asphalt layer modulus is less than EEXP. This check involves placing an overlay over the pavement and computing the remaining life of the overlaid pavement. The amount of overlay is projected based on the difference between remaining life and design life, but is at least 1 inch (25.4 mm). If the remaining life with overlay is greater than the remaining life without overlay (RL(1)), the remaining life of the original pavement is reliable and the program proceeds to the next step. If the check shows that the remaining life is unreliable, the following strategy is used:

(a) KODE=0 or 4 (field deflection data available). If RL(1) is greater than the design life, or if RL(1) is less than the design life and the solution was obtained using a default model (described in Section 4.3.1), the default model is again used to determine the base and subgrade stiffnesses, but with the asphalt layer stiffness increased by 25%.

(b) KODE=1, 2 or 3 (lab data or estimated moduli). If RL(1) is infinite (compressive critical strain), this data set is omitted from analysis since no further action is possible. This condition will generally be encountered when an error has been made in the input data.

For all other cases the program proceeds to the next step.

4.3.6 Determination of Required Overlay Thickness, Step 4

If the remaining life determined in the previous step is less than the design life, an overlay thickness is projected based on the difference between remaining life and the design life. If the reliability check was used in the previous step, the information on remaining life vs. overlay thickness is utilized for the projection.

As a result of adding an overlay to the existing pavement. the state of stress experienced by the base and subgrade layers changes. It is, therefore, necessary to readjust these layer stiffnesses to correspond to the changed stresses; the procedure outlined in Step 2, Section 4.3.4 is used. Once the layer stiffnesses corresponding to the new stresses have been found, the critical horizontal tensile strain is computed, and the remaining life is determined from equation 4-25.

If the remaining life of the projected overlay thickness is not within 25% of the design life, a new overlay thickness is projected based on the remaining life computed from the previous two iterations. Since the relationship between overlay thickness and log RLIFE is approximately linear, this procedure generally converges in three iterations. The 25% criterion has been chosen in order to reduce the number of iterations without significant loss in accuracy in interpolating the required overlay thickness.

CHAPTER 5
MATERIAL CHARACTERIZATION

The parameters required in the overlay design procedure are the thickness of each layer and related material properties. Only three material properties are required to define the structural capacity of existing pavement structures. These material properties are: (i) Poisson's ratio; (ii) density and; (iii) modulus, and are determined in such a manner that they reflect, within the limits of elastic layer theory, the structural capacity of existing pavement structures. It should be pointed out that the density is the in-place wet density.

As it was indicated, the standard design procedure includes a primary design and four optional schemes. The primary design is based on in-situ evaluation of pavement layer properties from measured surface deflections for a three-layer pavement system or a four-layer system if the three-layer model does not result in a solution. The four other options offered include:

A. The use of laboratory-determined layer properties in a three-layer pavement system;

B. The use of laboratory-determined layer properties in a four-layer pavement system;

C. The use of estimated layer properties (default values) in a three-layer pavement system;

D. The use of measured surface deflections in a four-layer analysis, bypassing the three-layer analysis.

In general, whenever field testing is conducted with loads that differ from design load, or when an overlay is placed on an existing pavement, the stresses experienced by the base, subbase and subgrade layers change. It is, therefore, necessary to correct the in-situ stiffness to account for the different stress states and if field testing is conducted when the asphalt layer temperature is different from the design temperature, the temperature dependency of the asphalt layer modulus has to be considered.

The material characterization procedures for these options are as follows.

5.1 THICKNESS REQUIREMENTS

This overlay design procedure requires that all layer thicknesses be known, which can generally be obtained from construction records. Where such records are not available, pavement cores might be needed to determine these thicknesses.

The design scheme characterizes the existing pavement as a three-layer structure composed of an asphalt layer with thickness H_1, a base layer with thickness H_2, and subgrade which is semi-infinite in depth, or a four-layer structure (consisting of an asphalt layer with thickness H1, a base layer with thickness H2, a subbase layer with thickness H3, and a subgrade layer with semi-infinite depth) if the three-layer analysis fails to find a solution for the layer stiffness. Since at least some pavements are composed of more than three or four layers, it is necessary to combine some of the existing layers into one layer to fit the model.

The following scheme is used:

Layer 1 - H_1 will be equal to the sum of all existing asphalt layer thicknesses and will include surface course, leveling course, asphaltic concrete or asphalt-treated base and previous overlays, if any.

Layer 2 - If the existing pavement is constructed with:

(a) Granular base and subbase, H_2 will be the sum of base and subbase thicknesses;

(b) Cement-treated base on lime-stabilized subbase, H_2 will be the sum of base and subbase thicknesses;

(c) Cement-treated base on granular subbase, H_2 will be the base thickness;

(d) Granular base on lime-stabilized subbase, H_2 will be the granular base thickness.

The above thickness assignments are done internally in the program; the user inputs the layer thicknesses with as-built values.

In the event that a pavement is built with only three layers (no subbase) and a four-layer analysis is needed or requested, an artificial subbase layer is created by taking one-third of the base thickness plus six inches (152 mm) as discussed in Section 4.3.1.

5.2 POISSON'S RATIO

Materials characterization includes Poisson's ratio for each layer. These values are not assumed a priori in the program; rather, the user supplies them as part of the input data. Unless the user has specific values of Poisson's ratio for his materials, the values in Table 4 are suggested.

TABLE 4
SUGGESTED POISSON'S RATIOS OF MATERIALS

Material Type	Poisson's Ratio
Asphalt Layer	0.40
Granular Layer	0.37
Cement-Treated Layer	0.15
Subgrade	0.45

5.3 LAYER DENSITIES

Since the overlay design procedure is based on stress-dependent elastic theory, the contribution to stresses resulting from the self-weight of layers cannot be neglected (see Section 4.3.1). The contribution of gravity forces to stresses is computed by subroutine GRAV from layer thicknesses and in-situ densities, i.e., wet densities. An estimation of layer densities is adequate since the contribution from gravity loads is generally smaller than from static or dynamic loads.

5.4 DETERMINATION OF LAYER MODULI USING DEFLECTIONS

The most commonly-used method for determining structural capacity of a pavement is deflection measurements. In this analysis procedure, the pavement layer moduli are determined from the deflection basin parameters using subroutines DYNAFL, RDRFWD, or MOD4 (depending on the type of analysis requested or needed), as discussed in Section 4.3. The in-situ layer stiffnesses are asphaltic concrete modulus E_1, base modulus E_B, subbase modulus E_{SB} (if present) and subgrade modulus E_S. These in-situ stiffnesses are determined for the four different test device loadings, all of which differ from the actual field (truck) loading conditions.

Since there is overwhelming evidence indicating that the modulus of most soils and granular materials is sensitive to stress, the calculated in-situ moduli are corrected for stress effects.

For granular materials, the interrelation for in-situ moduli stress dependency are:

$$M_R = A_1 \, \theta^{B_1} \qquad (5\text{-}1)$$

and for soils,

$$M_R = A_2 \, \sigma_d^{B_2} \qquad (5\text{-}2)$$

where θ is the sum of principal stresses;

σ_d is the deviatoric stress.

The constants A_1, A_2, B_1 and B_2 are material constants determined from laboratory testing of pavement material under simulated field loading conditions. When surface deflection measurements are used, the constants A1, A2 (and A3 if subbase exists) are determined by the program, but the constants B1, B2, (and B3 for subbase) must either be determined from laboratory testing or estimated from the tables presented in Appendix B.

5.5 DETERMINATION OF LAYER MODULI USING LABORATORY TESTING

The laboratory testing procedures used to determine the moduli values should reproduce field conditions as accurately as possible. In a simulation of actual field loading conditions, the dynamic triaxial test is becoming widely accepted. In general, the dynamic modulus testing is used for asphaltic concrete and the resilient modulus test is used for granular material and subgrade.

5.5.1 Moduli of Asphaltic Concrete

The results of non-destructive testing techniques, such as Dynaflect or Road Rater, can also be utilized to arrive at an estimate of in-situ stiffness or modulus E_1. This estimate is, however, dependent on pavement thickness, frequency of load application and average pavement temperature.

In asphaltic pavements, the presence of cracks, voids, and discontinuities affect the in-situ pavement modulus. The modulus of a flexible pavement with moderate and severe distress might range from 20,000 to 80,000 psi (140 to 550 MPa) as compared to 300,000 to 500,000 psi (2000 to 3500 MPa) for uncracked pavement.

The asphaltic concrete modulus is a temperature and frequency-dependent material property. In Figure 6 modulus-temperature relationships of several asphaltic concretes are presented.

5.5.2 Base Course

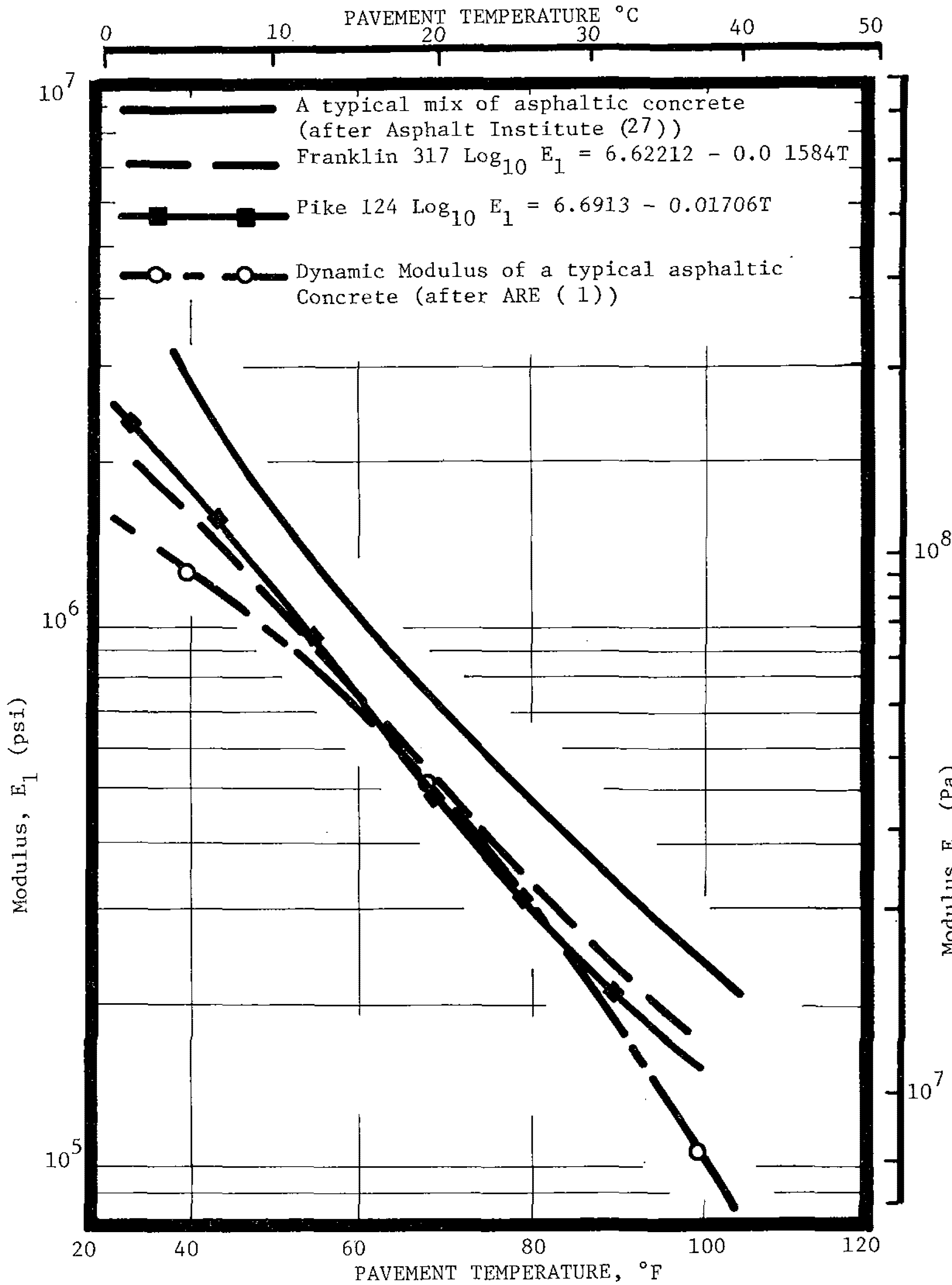


FIGURE 6. Modulus-Temperature Relationships for several asphaltic concretes.

The base course layer in a pavement structure can be generally placed into two broad categories:

(1) untreated granular base layer; and

(2) treated base layers such as asphalt base, cement-treated, lime-fly ash, etc.

The "bound" base layers (treated) are characterized similarly to pavement structures, with moduli E_i ranging from 100,000 to over 1×10^6 psi (700 to 7000 MPa) and Poisson's ration ranging from 0.20 for soil cement to 0.40 for asphaltic base. The characterization techniques and representation of material properties are similar to procedures outlined for pavement materials. The granular base course (untreated) is characterized by resilient modulus, M_R, which is dependent upon stress state. At any given point in the base course layer, the stress state can be defined by:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \tag{5-3}$$

where σ_1 = vertical stress (major principal stress)

$\sigma_2 = \sigma_3$ = horizontal stresses or minor and intermediate principal stresses.

The stress state can be determined using multi-layer elastic programs discussed in the report.

The modulus of resilience, M_R, is defined as:

$$M_R = A_1 \, \theta^{B_1} \tag{5-4}$$

where A_1 and B_1 are material constants (see Figure 7). The determination of modulus of resilience, M_R, can be carried out in the laboratory using triaxial testing procedures, and under constant confining pressure, $\sigma_3 = \sigma_2$. The axial stress, σ_1, can either be a periodic or cyclic stress function superimposed on the lateral stress as given by:

$$\sigma_1 = \Delta\sigma + \sigma_3 \tag{5-5}$$

defined by:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \tag{5-6}$$

Research studies have also indicated that the stress dependency of M_R can be represented by equation 5-7:

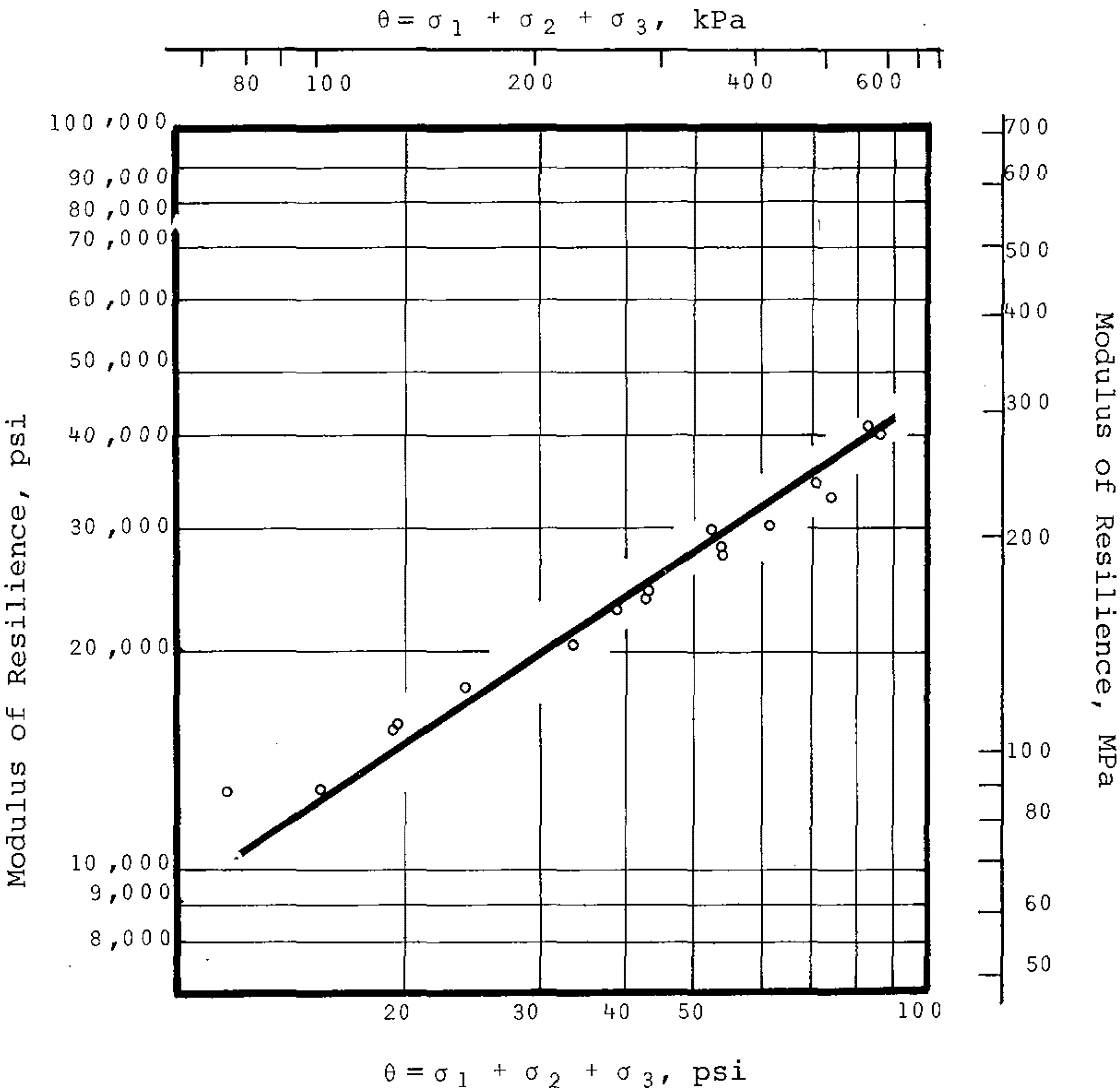


Figure 7. Typical Resilient Modulus vs. Sum of Principal Stresses for Granular Material

$$M_R = A_1 \ (\sigma_3)^{B_1} \tag{5-7}$$

where A_1 and A_2 are constants and σ_3 is the confining pressure in a triaxial test. The relationship is found to be true as long as the variation in deviator stress is not too great for a given confining pressure. It should be noted that modulus of resilience of granular base layers vary from a low value of 5,000 psi (34 MPa) for relatively low confining conditions to 80,000 to 100,000 psi (550 to 670 MPa) under high states of stresses.

5.5.3 Subbase Course

The subbase materials are generally very similar to base courses, except that they are generally somewhat weaker, i.e., with lower modulus. The overlay design program assumes that subbases are of the granular type, having a modulus of resilience that depends on the bulk stress, as given by equation 5-4. If a subbase is known to have a M_R that decreases with increasing deviatoric stress (behaves similar to subgrade) the user should treat this layer as if it were subgrade, i.e., specify that no subbase exists by not defining a layer three thickness.

5.5.4 Subgrade

In empirical and semi-empirical pavement design theories, the subgrade support has been characterized by parameters such as; California Bearing Ratio (CBR), subgrade reaction K, and soil support values. In multilayer elastic analysis of pavement systems, the subgrade support is represented by modulus E and Poisson's ratio μ. The subgrade modulus ranges from a low of 3,000 to 5,000 psi (21 to 34 MPa) for poor soils to 30,000 to 50,000 psi (210 to 340 MPa) for good soils. The Poisson's ratio is generally within a range of 0.40 to 0.45.

The laboratory testing of soils is carried out under triaxial state of stresses with confining pressure applied dynamically or as a constant pressure. The normal testing procedures require a constant applied confining pressure upon which a dynamically applied deviatoric stress is superimposed. The subgrade modulus is defined by modulus of resilience, M_R, given by:

$$M_R = A_2 \quad \sigma_d^{\pm B_2} \tag{5-8}$$

where A_2 and B_2 are material constants and σ_d is deviatoric stress, $\sigma_1 - \sigma_3$.

Some research studies on cohesive soils have also indicated that M_R decreases with increasing deviatoric stresses, σ_d, up to 10 to 15 psi (.2 to .3 MPa), beyond which the modulus M_R increases slightly or remains relatively constant. The relation between modulus M_R and stress σ_d is also dependent on soil types. For clayey soils, the parameter B_2 has a negative sign. In addition, the modulus of resilience remains relatively unaffected by the changes in confining pressures.

For sandy and silty clay soils, however, the parameter B_2 retains a small positive value, i.e., the modulus increases with increase in the deviatoric stress.

In Figure 8, the variation of modulus of resilience of typical soils with deviatoric stress is shown. The magnitude of deviatoric stress under a given pavement structure is calculated using multi-layered elastic programs.

Tables presented in Appendix B represent typical modulus of resilience relationships for various pavement component materials. The relationships have all been determined in the laboratory by repeated load triaxial tests or by a cyclic load triaxial test.

5.6 TESTING CONDITIONS

The following may be considered as typical test conditions:

1. For asphalt concrete, it is suggested that the modulus of resilience be determined by conducting tests over a range of temperatures. The modulus value used for the design of the overlay may then be selected based on the average in-situ temperature.

2. The confining pressure used in the triaxial test for base and subbase materials should be equal to the expected over-burden pressure and may range between 5 and 10 psi (.1 to .2 MPa). The deviatoric stress to be used ranges between 10 and 20 psi (.2 and .4 MPa), with the asphaltic concrete thickness equal to six inches and greater than six inches, respectively. Granular or cohesionless material should be recompacted to the field density, when undisturbed samples cannot be easily obtained, for laboratory testing.

3. Undisturbed samples should be taken where possible or recompacted to field densities for subgrade materials. The deviatoric stress to be used in the test should cover a range between 2 psi and 12 psi (.04 to .24 MPa) with the confining pressure equal to the expected overburden pressure, ranging between 2 and 5 psi (.04 to .1 MPa).

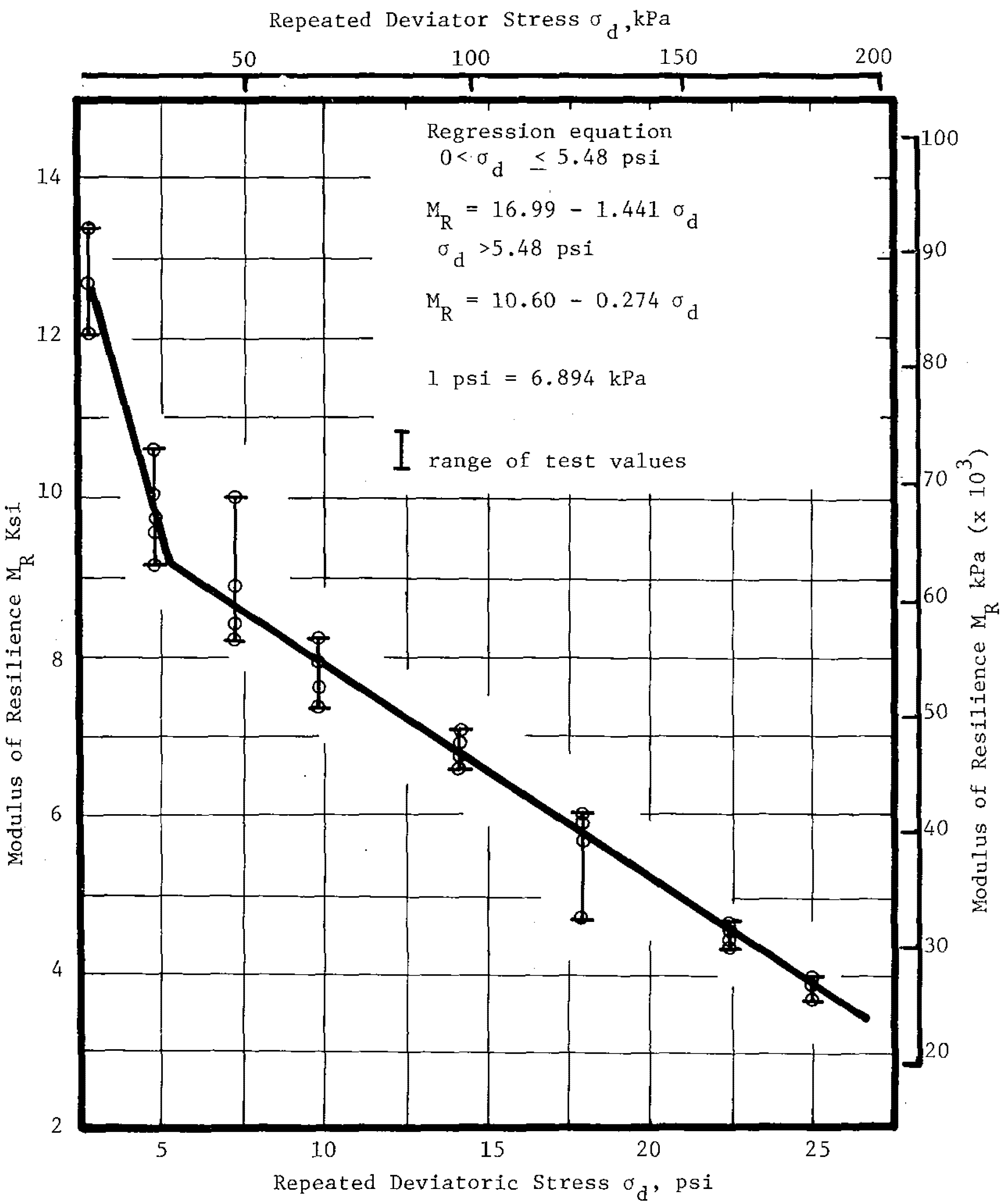


FIGURE 8. Variation of the Modulus of Resilience with Deviatoric Stress for a Subgrade

CHAPTER 6

DESIGN EXAMPLES, COMPARISON OF METHODS AND MODEL VERIFICATION

This chapter presents measured deflection data from projects in five states (Utah, Arizona, Ohio, Florida and California) and the analysis of this data using the FHWA-RII overlay design method. Also presented are analyses of the same data using the California (4), Utah (6), Louisiana (24) and Mississippi (25) overlay design procedures. This comparison of methods is not intended as direct verification of the proposed method; rather it is included to show that a great variation exists in the methods currently in use by various agencies, and that the overlay thicknesses derived from the proposed design scheme are not entirely in disagreement with existing practices.

The reader will note the omission of the FHWA-ARE Method (1) in these comparisons. As was stated in the discussion of this procedure in Chapter 2, a great deal of laboratory data for materials characterization is required by this method. The coordinated efforts of the panelists and the authors failed to unearth this information.

6.1 Analysis of Field Data

Measured Dynaflect deflection data taken on fifteen projects (representing a total of 236 individual test locations) has been analyzed using the proposed method as well as the methods used by the states enumerated above. The measured deflection data along with detailed results of these analyses are presented in Tables 7 through 33.

It should be mentioned that the deflection data in these tables has been tabulated to three significant figures in order to keep the columns the same length (some agencies report data to three significant figures when the measurements were made on the 0.03 range) and should not be interpreted as an endorsement of their accuracy. Deflection data was taken before and after placement of overlay for ten projects (Utah and Arizona), and the data for Franklin - 317 was taken after placement of an asphalt base, an asphalt leveling course and an asphalt surface course. The rest of the projects represent deflection surveys of existing pavements at various stages of deterioration and are included for comparison purposes only.

Although considerable information was available about these projects, some assumptions had to be made about these pavements:

a) Unless there was specific information that cracking was present in the region of deflection measurement, the pavement was assumed to be uncracked.

b) Poisson's ratios specified in Table 4 were used for all projects.

c) The base material for Lupton and Avondale (Arizona) is a combination of soil and sandy material, therefore it was assumed that the base stiffness is stress-independent.

d) Granular bases were assumed to have the same stress-dependency slope as the AASHO base material, i.e., B1 = 0.45 was used.

e) Except for Franklin-317 (laboratory data was available for this project) the subgrade was assumed to behave similarly to the AASHO subgrade, i.e., B2 = 1.1 was used.

f) The asphalt modulus-temperature relationship given in Figure 6 shown by the dot-dash line (after ARE) was used to adjust the asphalt layer stiffness from test temperature to design temperature for all sections except Delaware County Road 72.

h) The following design temperatures were assumed:

Utah - 60°F (16°C)
Ohio - 70°F (21°C)
Arizona - 70°F (21°C)
Florida - 72°F (22°C)
California - 80°F (27°C)

i) The Regional Factor was taken from Figure 5, Volume 2 (2)

j) The Seasonal Factor was assigned as follows:

Ohio - Table 3, Volume 2 (2)
Utah - estimated from Ohio data
Arizona - estimated from Ohio data
California and Florida - 1.0

The moduli values appearing under the column heading "Layer Stiffnesses" have been computed from the measured deflections and represent the in-situ layer stiffnesses, corrected for temperature and stress effects. The values appearing under column headings A.C., BASE, SUBB, and SUBG, represent the computed layer stiffnesses after temperature and stress correction, i.e., corrected to design conditions. If a value appears under the subbase column, this indicates that a four-layer analysis was required for that data point. In that case, the base thickness has been reduced by one-third, and a stress-independent subbase with thickness of one-third base thickness plus 6 in. (152mm) has been added.

As can be seen from these tables, A.C. stiffness shows a significant variation from test point to test point while the subgrade stiffness is relatively constant. Some of this variation is the result of variations in layer thicknesses, and part can be explained by the readout resolution problems discussed in Chapter 4, especially since the variation is greatest for projects having relatively low deflections. Of course, the deflection data represents in-service pavements with varying structural conditions, exhibiting some Class 2 and 3 cracking (22). However, the specific relationship between test locations and the existence of cracking is unknown, consequently the analysis assumed that no cracking existed. The existence of cracks in the vicinity of measuring equipment would also be expected to have a larger effect on layer 1 than on subgrade and therefore contribute to the variation and the magnitude of layer stiffnesses. The Delaware County data, taken in accordance with testing procedures outlined in the User Manual (2) (Volume 2) on a relatively new pavement with no failure areas, shows a much smaller variation in the layer properties. The stiffness values for this project are also considerably closer to the expected values than for other projects, showing the importance of careful field measurements.

The data in Table 33 was taken on a pavement constructed with a cement treated base which is currently exhibiting extensive Class 2 cracking (22). Since the base is already in a cracked condition and the spreadability is below 55%, this data was analyzed assuming that the base could be treated as a granular material, as discussed in Section 4.3.1. This assumption leads to rather high layer 1 stiffnesses since the cement treated base acts as a locally stiff material and contributes to the apparent asphalt layer stiffness. However, the overlay requirements determined from this model are in reasonable agreement with current practice in California, showing that the model has some merit as an analysis scheme.

6.2 Comparison of Methods

The summary of the computed overlay requirements is presented in Table 5. As is apparent from this table, the average overlay thickness (average of all tests) determined by OAF and by the California (4) Method is identical, even

TABLE 5. SUMMARY OF OVERLAY THICKNESSES

PROJECT		No. of tests	A.C. Thick	AVERAGE OVERLAY THICKNESS (in.) OAF	CA.	LA.	MISS.	UTAH
American Fork	-B	13	3.	2.71	0.16	7.08	0.80	0.30
(Utah)	-A	3	4.	0.00	0.00	0.40	0.00	0.00
Coalville	-B	7	5.75	2.51	1.81	5.93	8.31	0.84
(Utah)	-A	3	13.15	0.73	0.00	3.40	5.23	0.87
Cove Fort	-B	12	4.	3.45	3.12	9.38	6.07	7.27
(Utah)	-A	7	8.2	1.31	3.77	4.86	6.36	2.54
Juab County	-B	20	5.	2.05	0.67	5.48	6.33	3.48
Utah)	-A	16	7.	2.30	3.11	6.59	9.38	5.90
Spanish Fork	-B	17	5.	1.04	0.00	5.11	0.52	0.00
(Utah)	-A	9	9.2	0.00	0.00	0.00	0.00	0.00
Franklin 317	-301	8	6.	2.39	1.48	4.89	6.23	2.84
(Ohio)	-402	19	7.25	0.96	0.11	3.69	3.32	0.97
(Ohio)	-404	19	8.5	0.00	0.00	1.27	0.48	0.02
Dead River	-B	1	7.25	2.3	3.7	5.8	9.0	0.9
(Arizona)	-A	1	10.5	0.0	0.0	4.0	2.9	0.0
Crazy Creek	-B	1	4.	4.3	4.3	9.5	9.7	4.6
(Arizona)	-A	1	6.25	0.0	0.0	2.8	4.2	0.4
Avondale	-B	1	4.	0.0	3.7	5.4	9.0	0.6
(Arizona)	-A	1	6.	0.0	0.4	5.2	4.9	2.6
Benson	-B	1	7.75	5.2	2.4	4.8	7.2	2.9
(Arizona)	-A	1	9.5	0.0	0.0	2.8	2.8	0.0
Lupton	-B	1	4.	1.5	2.1	4.7	7.6	2.7
(Arizona)	-A	1	7.5	0.0	0.0	6.0	1.6	2.1
I-75-Fla.		22	4.5	1.15	0.24	3.31	4.11	1.10
Delco Rd. 72-OH		17	5.5	0.62	0.45	5.32	3.08	1.89
02-616-515-97-CA		19	4.8	0.57	5.81	6.27	9.29	1.08
03-Butler CTB		15	3.	0.40	1.24	4.67	2.69	0.00
AVERAGE OF ALL TESTS				1.28	1.30	4.71	4.39	1.77

NOTE: 1 in. = 25.4 mm

though the California (4) procedure totally ignores any temperature effects. The Utah (6) procedure predicts a slightly higher overlay thickness than does OAF, but most of this increase is from a relatively small percentage of test locations where the maximum deflection and the shape of the deflection basin indicate reduced subgrade support. Since maximum deflection is very sensitive to support values whereas subgrade has a relatively small influence on critical strains, this difference is to be expected. The Louisiana (24) and Mississippi (25) procedures result in substantially thicker overlays than the other three methods. Both of these procedures have been patterned after the Utah (6) method, except that they have been developed based on local experience. The temperature effect has been ignored in the Mississippi (25) method whereas a Temperature Adjustment Factor (used to adjust measured deflection from test temperature to 60°F (16°C)) is incorporated in the Louisiana (24) design. This TAF, however, is inconsistent with those used by other states and appears more appropriate for airport pavements than for thinner highway pavements and is partially responsible for the thick overlays. Another explanation for the increased overlay thicknesses with these procedures is that both Mississippi and Louisiana have an abundance of weak subgrade, resulting in high maximum deflection.

6.3 Model Verification

It was hoped that the above data would clearly indicate the validity of the design procedures. There is, however, no concrete evidence to indicate that the overlay thicknesses predicted by any of the above methods is appropriate. Franklin 317 in Ohio has been in service for nine years and is presently in good condition after an estimated three million equivalent axle loads (approximately 60% of its design life). It is therefore probable that the 1.27 inches (32.2 mm) predicted by Louisiana (24) or the 0.48 inches (12.2 mm) predicted by Mississippi (25) is excessive. Also, Delaware County Road 72 has been in service for five years and is currently in excellent condition with approximately 35% of its design life used. Again, it is probable that the 0.62 inches (15.7 mm) overlay predicted by OAF is adequate. The remainder of the overlaid pavement sections analyzed have not been in service long enough to form any conclusions.

A successful overlay design method should be consistent with before and after overlay measurements. It should be able to predict that the required overlay thickness after an overlay has been placed will decrease by the amount of actual overlay, e.g., if the before overlay measurements indicated that a three inch (76 mm) overlay is needed and a two inch

(51 mm) overlay is built, after overlay measurements should indicate the need of an additional one inch (25 mm) overlay. If the overlay effectiveness factor is defined as the ratio of decrease in predicted overlay (from before and after measurements) divided by the actual overlay, then the effectiveness factor can be used to judge the success of the method.

TABLE 6
EFFECTIVENESS FACTOR

Method	EF	Range
OAF	1.11	.51 to 2.97
California	.93	.08 to 1.91
Louisiana	1.64	-.37 to 6.68
Mississippi	1.57	-.07 to 2.51
Utah	.91	-1.00 to 1.87

The results of this analysis are presented in Table 6 and represent the weighted average (weighted by the number of test locations after overlay) of the before and after overlay data; however, the data from Juab County - Utah has been omitted since all five design methods call for a greater overlay after the pavement was overlaid than before overlay i.e., a negative EF. It is therefore probable that some error exists in this data. For the purpose of this analysis, it was assumed that when the before overlay measurements predicted a lower overlay than was actually built, and the after overlay measurements predicted that no overlay was needed, then the effectiveness factor for that method is 1.0. (For example, the California (4) method predicts for American Fork 0.16 inches (4 mm) before overlay and 0.00 inches (0.00 mm) after overlay; therefore the effectiveness factor is 1.0). Without this assumption, most of the projects would have to be omitted from the analysis.

As may be seen from the above table, the effectiveness factor for OAF, California (4) and Utah (6) are close to the expected 1.0, and the range of variation is not unreasonable. Louisiana (24) and Mississippi (25) both result in substantially higher effectiveness factors and have more variability than the other methods.

As was stated earlier, the computed asphalt layer stiffness (Tables 7 through 33) show a large variation, and in a few cases the asphalt is stiffer than steel; nevertheless the values are reasonable in a great majority of the cases. In most instances the high asphalt moduli result when the measured deflections are small. For instance, the after overlay deflections for American Fork - Utah (Table 8) are totally inconsistent with the authors' experience with over

100,000 Dynaflect measurements on all kinds of pavements - it is very probable that those deflection values should be multiplied by a factor of three.

Although the evidence is circumstantial, the results of the above analysis indicate that the proposed FHWA-RII method is successful in evaluating the in-situ layer properties and in detemining the required overlay thicknesses.

TABLE 7. AMERICAN FORK TO MAIN ST. BEFORE OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 3-5-76 SEASONAL FACTOR: .82 TEST TEMPERATURE: 36°F
A.C. THICKNESS: 3 in. EEXP:1,375,000 psi EDES:750,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.69×10^6 EAL ADJUSTED DESIGN LIFE+: 3.31×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
5	.940	.550	.288	.186	.105	534.	18.0	--	8.3	3.14	0.0	8.0	1.3	0.5
10	.930	.550	.350	.255	.105	158.	46.4	--	10.1	4.06	0.0	8.0	1.2	0.5
15	.900	.350	.150	.105	.042	238.	10.5	--	11.6	4.70	0.0	7.8	1.0	0.3
20	.870	.600	.420	.270	.108	633.	131.3	--	9.3	0.00	0.0	7.6	0.8	0.1
25	.860	.460	.204	.114	.044	657.	11.4	--	9.2	2.75	0.0	7.5	0.8	0.0
30	.970	.560	.350	.216	.120	191.	25.8	--	9.8	4.55	0.3	8.1	1.6	0.6
35	1.200	.570	.258	.150	.126	251.	9.6	--	7.3	4.94	1.8	9.1	3.4	1.9
40	.700	.360	.180	.120	.048	524.	18.1	--	11.8	2.88	0.0	6.6	0.0	0.0
45	.680	.380	.174	.108	.042	918.	15.5	--	11.1	1.78	0.0	6.5	0.0	0.0
50	.460	.246	.138	.105	.040	516.	39.4	--	19.5	1.94	0.0	4.7	0.0	0.0
55	.480	.255	.156	.108	.039	398.	39.1	--	19.3	2.51	0.0	4.9	0.0	0.0
60	.630	.370	.186	.132	.046	996.	22.7	--	11.8	1.49	0.0	6.1	0.0	0.0
65	.790	.480	.276	.192	.156	1601.	18.0	--	8.3	0.90	0.0	7.1	0.3	0.0
AVERAGE										2.71	0.16	7.08	0.80	0.30
STANDARD DEVIATION										1.59	0.50	1.28	0.97	0.53

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 8. AMERICAN FORK TO MAIN ST. AFTER OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 9-21-77 SEASONAL FACTOR: 1.12 TEST TEMPERATURE: 60°F
A.C. THICKNESS: 4 in. EEXP: 725,000 psi EDES: 750,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.69×10^6 EAL ADJUSTED DESIGN LIFE+: 4.53×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.280	.250	.180	.150	.120	34510.	15.9	--	24.4	0.00	0.0	0.0	0.0	0.0
2	.350	.300	.250	.220	.160	26513.	12.6	--	19.7	0.00	0.0	0.0	0.0	0.0
3	.460	.390	.280	.220	.160	7646.	319.0	--	15.8	0.00	0.0	1.2	0.0	0.0
AVERAGE										0.00	0.00	0.40	0.00	0.00
STANDARD DEVIATION										0.00	0.00	0.69	0.00	0.00

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 9. COALVILLE TO ECHO JUNCTION BEFORE OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 5-2-76 SEASONAL FACTOR: 0.88 TEST TEMPERATURE: 70°F
A.C. THICKNESS: 5.75 in. EEXP: 500,000 psi EDES: 750,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 10.5×10^6 EAL ADJUSTED DESIGN LIFE+: 13.9×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
11060	.980	.660	.480	.288	.198	750.	29.6	--	9.5	1.08	1.5	5.7	8.0	0.3
11039	1.050	.720	.540	.350	.246	2134.	1.9	--	16.2	0.00	2.1	6.0	8.6	0.8
11033	1.050	.720	.600	.420	.297	750.	111.1	--	7.9	0.00	2.1	6.0	8.6	0.8
11025	.820	.540	.420	.222	.156	490.	112.4	--	10.7	0.22	0.0	5.0	6.4	0.0
11020	.820	.490	.360	.168	.120	225.	49.8	--	11.1	3.49	0.0	5.0	6.4	0.0
11012*	1.260	.740	.460	.237	.156	77.	13.7	--	7.6	6.51	3.5	6.9	10.1	2.0
11005*	1.260	.780	.490	.258	.162	99.	15.1	--	7.4	6.28	3.5	6.9	10.1	2.0
AVERAGE										2.51	1.81	5.93	8.31	0.84
STANDARD DEVIATION										2.92	1.45	0.78	1.53	0.86

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 10. COALVILLE TO ECHO JUNCTION AFTER OVERLAY-UTAH

REGIONAL FACTOR:1.5 TEST DATE: 7-21-77 SEASONAL FACTOR: 1.35 TEST TEMPERATURE: 91°F
A.C. THICKNESS: 13.15 in. EEXP: 185,000 psi EDES:750,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 10.5×10^6 EAL ADJUSTED DESIGN LIFE+: $21.3\ \times10^6$ EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.680	.480	.230	.170	.130	403.	8.9	--	25.5	0.00	0.0	3.1	4.7	0.6
2	.690	.480	.220	.160	.130	377.	8.4	--	26.0	0.00	0.0	3.2	4.8	0.6
3	.800	.460	.280	.200	.150	100.	30.7	--	14.4	2.20	0.0	3.9	6.2	1.4
AVERAGE										.73	0.00	3.40	5.23	0.87
STANDARD DEVIATION										1.27	0.00	0.44	0.84	0.46

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 11. COVE FORT TO SHINGLE CREEK BEFORE OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 11-5-74 SEASONAL FACTOR: .97 TEST TEMPERATURE: 39°F
A.C. THICKNESS: 4 in. EEXP:1,290,000 psi EDES:750,000 psi
BASE THICKNESS: 8 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.40×10^6 EAL ADJUSTED DESIGN LIFE+: 3.49×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
5	2.070	1.200	.450	.237	.174	219.	6.2	--	4.7	5.48	5.1	10.9	9.0	10.1
55	1.560	.920	.420	.210	.150	218.	11.2	--	5.6	5.09	3.4	9.6	6.5	7.7
105	1.590	1.080	.560	.350	.204	488.	9.2	--	4.9	3.39	3.6	9.7	6.7	7.8
155	1.680	1.110	.490	.279	.222	358.	8.9	--	4.9	4.21	3.9	9.9	7.2	8.2
205	1.680	.970	.540	.297	.231	100.	16.8	--	5.1	5.83	3.9	9.9	7.2	8.2
255	1.170	.740	.370	.168	.089	961.	5.6	--	14.2	1.36	1.6	8.2	3.8	5.1
305	1.740	1.400	.530	.360	.261	271.	11.2	--	4.5	4.78	4.1	10.1	7.5	8.6
355	1.110	.780	.460	.231	.138	646.	16.1	--	6.7	2.13	1.2	8.0	3.3	4.7
405	1.620	1.170	.590	.350	.204	623.	7.1	--	4.9	2.83	3.7	9.8	6.9	7.9
455	1.500	1.080	.480	.240	.159	766.	5.1	--	7.0	2.24	3.2	9.4	6.1	7.5
505	1.410	.960	.530	.297	.213	750.	7.6	--	5.7	2.23	2.8	9.1	5.6	6.9
555	1.080	.690	.288	.123	.056	749.	7.9	--	11.5	1.87	1.0	7.9	3.0	4.5
AVERAGE										3.45	3.12	9.38	6.07	7.27
STANDARD DEVIATION										1.56	1.26	0.92	1.83	1.69

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 12. COVE FORT TO SHINGLE CREEK AFTER OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 8-6-76 SEASONAL FACTOR: 1.38 TEST TEMPERATURE: 86°F
A.C. THICKNESS: 8.2 in. EEXP: 250,000 psi EDES: 750,000 psi
BASE THICKNESS: 8 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.40×10^6 EAL ADJUSTED DESIGN LIFE+: 4.98×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.170	.850	.480	.290	.230	488.	13.7	--	11.2	0.00	1.6	4.0	4.7	1.2
2	1.380	.980	.500	.300	.220	429.	8.6	--	10.8	0.34	2.7	4.8	6.3	2.4
3	1.680	1.200	.620	.350	.170	409.	5.2	--	10.8	0.80	3.9	5.7	8.0	4.0
4	1.620	1.260	.760	.470	.300	661.	5.5	--	9.5	0.00	3.7	5.6	7.7	3.7
5	1.200	.890	.590	.350	.250	374.	21.0	--	9.2	0.22	1.8	4.1	4.9	1.3
6	1.440	.960	.540	.360	.290	132.	16.4	--	7.5	3.78	3.0	5.0	6.6	2.8
7	1.380	.920	.580	.350	.270	100.	19.6	--	7.0	4.06	2.7	4.8	6.3	2.4
AVERAGE										1.31	2.77	4.86	6.36	2.54
STANDARD DEVIATION										1.80	0.87	0.66	1.25	1.07

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 13. JUAB COUNTY LINE TO LEVAN BEFORE OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 11-11-76 SEASONAL FACTOR: .96 TEST TEMPERATURE: 61°F
A.C. THICKNESS: 5 in. EEXP: 725,000 psi EDES: 750,000 psi
BASE THICKNESS: 9 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 12.01×10^6 EAL ADJUSTED DESIGN LIFE+: 17.30×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
2	.780	.600	.420	.297	.261	802.	68.2	35.4	9.3	0.00	0.0	5.5	6.4	3.3
6	.480	.370	.282	.219	.180	1465.	92.9	--	14.3	0.00	0.0	3.3	2.0	0.3
10	.460	.370	.255	.195	.144	808.	105.9	--	14.3	0.00	0.0	3.1	1.6	0.1
14	.930	.660	.450	.350	.282	750.	31.9	--	8.5	1.56	1.3	6.3	8.0	4.7
18	.840	.630	.430	.350	.270	165.	90.5	38.2	9.0	3.64	0.4	5.8	7.1	3.7
22	.700	.600	.420	.297	.252	4153.	22.4	--	11.2	0.00	0.0	5.0	5.4	2.5
26	.660	.560	.420	.350	.285	1465.	123.1	--	10.3	0.00	0.0	4.8	4.8	2.1
30	.980	.730	.520	.420	.350	750.	33.8	--	7.5	1.53	1.8	6.5	8.5	5.4
34	.740	.550	.360	.264	.216	269.	52.3	--	9.7	3.58	0.0	5.3	5.9	3.0
38*	.700	.450	.258	.192	.150	74.	49.5	--	12.7	5.06	0.0	5.0	5.4	2.5
42	.620	.480	.350	.264	.198	225.	67.9	--	10.3	2.90	0.0	4.5	4.3	1.7
46	.990	.750	.500	.390	.297	565.	52.2	27.7	7.5	1.73	1.8	6.6	8.6	5.5
50	.880	.740	.530	.430	.350	660.	99.4	--	7.4	0.00	0.7	6.0	7.5	4.4
54	.970	.770	.570	.510	.460	750.	49.3	--	7.0	1.02	1.7	6.5	8.4	5.3
58*	.900	.570	.350	.288	.258	70.	38.8	--	9.7	5.72	0.9	6.2	7.7	4.5
62	.720	.520	.360	.282	.228	750.	46.5	--	10.3	0.85	0.0	5.1	5.7	2.7
66	.830	.600	.380	.297	.267	750.	33.3	--	9.8	1.42	0.0	5.7	7.0	3.8
70	.650	.540	.370	.297	.261	624.	110.4	--	9.9	0.00	0.0	4.7	4.7	2.0
74	1.500	1.080	.670	.530	.430	100.	25.0	--	5.4	6.53	4.8	8.5	11.7	9.0
78*	.740	.470	.270	.195	.150	70.	33.5	--	19.2	5.37	0.0	5.2	5.9	3.0
AVERAGE										2.05	0.67	5.48	6.33	3.48
STANDARD DEVIATION										2.20	1.18	1.20	2.33	2.01

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 14. JUAB COUNTY LINE TO LEVAN AFTER OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 10-12-77 SEASONAL FACTOR: .98 TEST TEMPERATURE: 74°F
A.C. THICKNESS: 7 in. EEXP: 425,000 psi EDES: 750,000 psi
BASE THICKNESS: 9 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 12.01×10^6 EAL ADJUSTED DESIGN LIFE+: 17.65×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.080	.910	.680	.540	.420	527.	60.5	--	6.7	0.00	2.5	6.2	9.3	5.1
2	.780	.630	.500	.370	.300	2482.	23.4	--	11.9	0.00	0.0	4.8	6.4	2.4
3	.980	.750	.540	.470	.420	750.	29.7	--	9.2	0.00	1.8	5.7	8.5	4.2
4	.990	.740	.45	.300	.240	434.	26.1	--	9.4	1.74	1.8	5.8	8.6	4.3
5	.980	.700	.460	.300	.240	769.	16.6	--	10.9	0.23	1.8	5.7	8.5	4.2
6	1.050	.770	.540	.480	.420	750.	20.5	--	9.3	0.26	2.3	6.0	9.0	4.7
7	.950	.600	.350	.250	.190	100.	27.6	--	9.8	5.24	1.5	5.6	8.2	4.0
8	1.210	.820	.520	.380	.290	100.	24.6	--	7.2	5.53	3.4	6.8	10.2	6.1
9	1.200	.830	.480	.350	.250	141.	23.0	--	7.6	5.15	3.3	6.7	10.1	6.1
10	1.290	1.050	.560	.440	.350	1119.	4.9	--	11.1	0.00	3.8	7.0	10.6	6.8
11	1.170	.900	.590	.440	.350	154.	28.6	--	6.4	4.54	3.1	6.5	9.9	5.8
12	1.280	.980	.500	.380	.300	632.	10.3	--	8.9	1.37	3.7	7.0	10.6	6.7
13	1.560	1.200	.800	.620	.420	116.	21.6	--	4.8	5.57	5.1	7.8	12.0	8.4
14	1.560	1.260	.800	.590	.460	602.	15.0	--	6.1	1.54	5.1	7.8	12.0	8.4
15	1.620	1.260	.860	.640	.480	148.	20.6	--	4.7	5.27	5.4	8.1	12.2	8.7
16	1.590	1.200	.880	.580	.460	956.	4.9	--	7.8	0.32	5.2	8.0	12.1	8.5
AVERAGE										2.30	3.11	6.59	9.89	5.90
STANDARD DEVIATION										2.41	1.57	0.98	1.67	1.91

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 15. SPANISH FORK TO PROVO BEFORE OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 5-6-75 SEASONAL FACTOR: 0.80 TEST TEMPERATURE: 41°F
A.C. THICKNESS: 5 in. EEXP: 1,225,000 psi EDES: 750,000 psi
BASE THICKNESS: 7 in. BASE TYPE: GRAN SLAG BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.67×10^6 EAL ADJUSTED DESIGN LIFE+: 3.22×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
0*	.830	.520	.270	.252	.213	76.	31.0	--	11.4	4.32	0.0	6.4	1.9	0.0
5	.490	.370	.219	.144	.111	1031.	42.8	--	14.3	0.00	0.0	4.0	0.0	0.0
10	.580	.420	.234	.138	.072	906.	21.6	--	12.7	0.00	0.0	4.9	0.0	0.0
15	.660	.470	.249	.144	.076	755.	17.3	--	11.6	0.38	0.0	5.5	0.5	0.0
20	.710	.530	.350	.240	.195	672.	48.1	31.6	9.9	0.00	0.0	5.8	0.8	0.0
25	.820	.570	.380	.258	.216	611.	26.2	--	8.5	0.73	0.0	6.4	1.8	0.0
30	.660	.440	.237	.168	.144	176.	43.0	--	11.9	2.80	0.0	5.5	0.5	0.0
35	.640	.500	.350	.219	.180	1994.	13.8	--	10.6	0.00	0.0	5.3	0.3	0.0
40	.780	.570	.390	.264	.219	781.	29.2	--	8.6	0.29	0.0	6.2	1.5	0.0
45	.560	.370	.210	.144	.117	122.	58.3	--	14.8	2.66	0.0	4.7	0.0	0.0
50*	.480	.288	.182	.132	.114	161.	65.4	--	19.5	1.65	0.0	4.0	0.0	0.0
55	.680	.580	.270	.186	.159	750.	20.8	--	10.6	0.32	0.0	5.6	0.6	0.0
60*	.450	.285	.171	.129	.114	603.	61.2	--	18.2	0.00	0.0	3.7	0.0	0.0
65	.600	.420	.255	.174	.138	255.	47.6	--	12.2	1.72	0.0	5.0	0.2	0.0
70*	.460	.297	.156	.069	.051	990.	31.8	--	20.1	0.00	0.0	3.8	0.0	0.0
75	.530	.420	.273	.180	.141	456.	74.0	--	12.3	0.00	0.0	4.4	0.0	0.0
80	.700	.500	.297	.240	.135	177.	44.0	--	10.2	2.78	0.0	5.7	0.8	0.0
AVERAGE										1.04	0.00	5.11	0.52	0.00
STANDARD DEVIATION										1.36	0.00	0.90	0.65	0.00

\+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING

NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 16. SPANISH FORK TO PROVO AFTER OVERLAY-UTAH

REGIONAL FACTOR: 1.5 TEST DATE: 10-12-77 SEASONAL FACTOR: .98 TEST TEMPERATURE: 57°F (
A.C. THICKNESS: 9.2 in. EEXP: 800,000 psi EDES: 750,000 psi
BASE THICKNESS: 7 in. BASE TYPE: GRAN SLAG BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.67 x10^6 EAL ADJUSTED DESIGN LIFE+: 3.93 x10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.300	.270	.200	.150	.120	3578.	2.8	--	404.2	0.00	0.0	0.0	0.0	0.0
2	.230	.180	.110	.060	.040	1243.	24.1	--	47.3	0.00	0.0	0.0	0.0	0.0
3	.260	.240	.170	.140	.120	3669.	10.4	--	33.4	0.00	0.0	0.0	0.0	0.0
4	.280	.250	.190	.170	.160	2007.	27.8	--	26.3	0.00	0.0	0.0	0.0	0.0
5	.260	.210	.130	.090	.080	1105.	48.5	--	30.5	0.00	0.0	0.0	0.0	0.0
6	.270	.220	.150	.100	.080	1022.	62.5	--	27.6	0.00	0.0	0.0	0.0	0.0
7	.300	.260	.180	.140	.130	1724.	53.3	--	23.1	0.00	0.0	0.0	0.0	0.0
8	.270	.220	.150	.100	.090	863.	80.1	--	26.8	0.00	0.0	0.0	0.0	0.0
9	.300	.270	.180	.120	.100	750.	89.6	--	22.9	0.00	0.0	0.0	0.0	0.0
AVERAGE										0.00	0.00	0.00	0.00	0.00
STANDARD DEVIATION										0.00	0.00	0.00	0.00	0.00

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 17. FRANKLIN 317 - A.C. BASE (301) - OHIO

REGIONAL FACTOR: 1.0 TEST DATE: 10-15-71 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 70°F
A.C. THICKNESS: 6 in. in. EEXP: 500,000 psi EDES: 500,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 0.8
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.84×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1175	1.140	.900	.530	.280	.180	826.	3.4		11.7	0.31	2.0	5.2	6.9	3.2
1165	.920	.630	.330	.190	.120	532.	7.2		13.5	1.43	0.3	4.2	4.8	1.8
1160	1.120	.780	.470	.270	.180	260.	17.5		9.7	3.58	1.9	5.1	6.7	3.1
1155	1.020	.710	.340	.180	.100	476.	5.5		13.7	1.91	1.2	4.7	5.8	2.4
1150	.920	.680	.380	.220	.150	620.	9.2		11.8	0.96	0.3	4.2	4.8	1.8
1140	.900	.630	.300	.180	.100	538.	6.8		14.4	1.37	0.1	4.1	4.6	1.6
1130	1.440	.920	.430	.220	.160	100.	10.7		8.2	6.30	3.6	6.2	8.9	5.2
1125	1.200	.890	.600	.350	.220	261.	24.5		8.5	3.29	2.4	5.4	7.3	3.6
AVERAGE										2.39	1.48	4.89	6.23	2.84
STANDARD DEVIATION										1.93	1.23	0.73	1.51	1.21

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 18. FRANKLIN 317 - LEVELING COURSE (402) - OHIO

REGIONAL FACTOR: 1.0 TEST DATE: 10-18-71 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 67°F
A.C. THICKNESS: 7.25 in. EEXP: 500,000 psi EDES: 500,000 psi
BASE THICKNESS: 4 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 0.8
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.84×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1300	.870	.710	.440	.260	.140	714.	3.6		15.2	0.00	0.0	4.2	4.3	1.5
1290	1.110	.850	.550	.330	.220	519.	3.2		11.2	0.86	1.8	5.3	6.6	3.2
1280	.750	.550	.330	.200	.130	591.	5.4		17.4	0.00	0.0	3.5	2.9	0.7
1270	.700	.520	.360	.240	.180	767.	9.4		14.0	0.00	0.0	3.2	2.3	0.4
1260	.700	.510	.320	.220	.140	151.	39.7		14.1	2.60	0.0	3.2	2.3	0.4
1250	.810	.580	.380	.240	.180	552.	7.2		13.2	0.14	0.0	3.9	3.6	1.1
1240	.770	.580	.360	.260	.130	482.	13.1		12.7	0.40	0.0	3.6	3.2	0.8
1230	.780	.540	.320	.190	.130	195.	23.2		13.5	2.96	0.0	3.7	3.3	0.9
1220	.740	.560	.360	.220	.130	301.	31.1		13.2	1.16	0.0	3.4	2.8	0.7
1210	.660	.480	.300	.180	.120	308.	28.9		15.2	1.06	0.0	3.0	1.8	0.1
1200	.800	.570	.340	.200	.130	499.	5.7		15.7	0.48	0.0	3.8	3.5	1.0
1190	.760	.550	.320	.190	.120	484.	7.3		15.3	0.50	0.0	3.6	3.1	0.8
1180	.900	.670	.430	.250	.140	678.	2.5		23.1	0.00	0.1	4.3	4.6	1.7
1170	.770	.580	.340	.180	.100	513.	6.3		15.8	0.33	0.0	3.6	3.2	0.8
1160	.870	.660	.420	.240	.130	420.	9.4		11.7	1.13	0.0	4.2	4.3	1.5
1150	.700	.510	.320	.210	.120	266.	31.5		14.4	1.47	0.0	3.2	2.3	0.4
1140	.750	.530	.320	.190	.120	245.	22.2		13.8	2.25	0.0	3.5	2.9	0.7
1130	.630	.460	.300	.180	.130	198.	43.0		15.7	1.62	0.0	2.7	1.5	0.0
1125	.890	.680	.430	.260	.160	402.	12.0		11.1	1.22	0.1	4.3	4.5	1.6
AVERAGE										0.96	0.11	3.69	3.32	0.97
STANDARD DEVIATION										0.90	0.41	0.59	1.18	0.73

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 19. FRANKLIN 317 - SURFACE COURSE (404) - OHIO

REGIONAL FACTOR: 1.0TEST DATE:11-11-71 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 73°F
A.C. THICKNESS: 8.5 in. EEXP:465,000 psi EDES:500.000 psi
BASE THICKNESS: 4in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 0.8
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.84×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1300	.680	.530	.330	.250	.150	397.	25.9		14.4	0.00	0.0	2.7	2.1	0.1
1290	.600	.470	.310	.250	.150	338.	49.6		15.6	0.00	0.0	2.1	1.1	0.0
1280	.460	.360	.230	.170	.130	414.	52.8		21.3	0.00	0.0	1.0	0.0	0.0
1270	.470	.380	.260	.240	.160	542.	197.0		18.3	0.00	0.0	1.0	0.0	0.0
1260	.460	.340	.240	.190	.130	472.	51.3		21.1	0.00	0.0	1.0	0.0	0.0
1250	.490	.380	.240	.200	.140	388.	81.5		18.9	0.00	0.0	1.1	0.2	0.0
1240	.440	.320	.210	.150	.110	338.	58.0		22.4	0.00	0.0	0.7	0.0	0.0
1230	.520	.390	.240	.160	.110	383.	34.3		19.5	0.00	0.0	1.5	0.4	0.0
1220	.480	.370	.230	.180	.120	281.	81.2		19.6	0.00	0.0	1.0	0.2	0.0
1210	.480	.360	.240	.180	.120	308.	70.8		19.7	0.00	0.0	1.0	0.2	0.0
1200	.530	.410	.270	.210	.130	759.	13.7		19.7	0.00	0.0	1.6	0.5	0.0
1190	.540	.420	.280	.210	.140	419.	47.4		18.1	0.00	0.0	1.6	0.5	0.0
1180	.560	.430	.260	.190	.120	458.	26.3		17.9	0.00	0.0	1.8	0.7	0.0
1170	.460	.370	.240	.190	.130	485.	109.8		19.9	0.00	0.0	1.0	0.0	0.0
1160	.710	.590	.420	.320	.210	393.	110.6		12.5	0.00	0.0	3.0	2.5	0.2
1150	.460	.370	.240	.190	.130	485.	109.8		19.8	0.00	0.0	1.0	0.0	0.0
1140	.500	.390	.260	.190	.130	293.	89.9		18.7	0.00	0.0	1.3	0.2	0.0
1130	.440	.340	.240	.200	.140	389.	191.7		20.4	0.00	0.0	0.7	0.0	0.0
1125	.550	.430	.290	.230	.130	423.	87.6		16.1	0.00	0.0	1.7	0.6	0.0
AVERAGE										0.00	0.00	1.27	0.48	0.02
STANDARD DEVIATION										0.00	0.00	0.63	0.71	0.05

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS   * CLASS 2 OR 3 CRACKING

NOTE: 1 psi = 6.895 kPa   1 in. = 25.4 mm

TABLE 20. DEAD RIVER BEFORE OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 5-22-74 SEASONAL FACTOR: .68 TEST TEMPERATURE: 83°F
A.C. THICKNESS: 7.25 in. EEXP: 280,000 psi EDES: 500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 3.29×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.458	1.206	.876	.692	.524	117.	--	--	5.1	2.27	3.7	5.8	9.0	0.9

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 21. DEAD RIVER AFTER OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 5-22-74 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 59°F
A.C. THICKNESS: 10.5 in. EEXP: 750,000 psi EDES: 500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^{6} EAL ADJUSTED DESIGN LIFE+: 4.84×10^{6} EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.750	.712	.600	.508	.356	412.	--	--	7.4	0.00	0.0	4.0	2.9	0.0

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 22. CRAZY CREEK BEFORE OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 3-25-75 SEASONAL FACTOR: .65 TEST TEMPERATURE: 43°F
A.C. THICKNESS: 4 in. EEXP: 1,180,000 psi EDES: 500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 3.16×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.596	1.230	.810	.616	.426	109.	--	--	5.4	4.27	4.3	9.5	9.7	4.6

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 23. CRAZY CREEK AFTER OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 8-26-75 SEASONAL FACTOR: 1.29 TEST TEMPERATURE: 97°F
A.C. THICKNESS: 6.25 in. EEXP: 125,000 psi EDES: 500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 6.24×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.860	.718	.598	.470	.333	350.	--	--	8.2	0.00	0.0	2.8	4.2	0.4

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 24. AVONDALE BEFORE OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 5-31-74 SEASONAL FACTOR: .94 TEST TEMPERATURE: 91°F
A.C. THICKNESS: 4 in. EEXP: 180,000 psi EDES: 500,000 psi
BASE THICKNESS: 8 in. BASE TYPE: SELECT BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^{6} EAL ADJUSTED DESIGN LIFE+: 4.55×10^{6} EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.*	LA.	MISS.	UTAH
1	1.458	.990	.690	.456	.334	5779.	4.2	--	11.7	0.00	3.7	5.4	9.0	0.6

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 25. AVONDALE AFTER OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 10-22-74 SEASONAL FACTOR: .99 TEST TEMPERATURE: 56°F
A.C. THICKNESS: 6 in. EEXP: 825,000 psi EDES: 500,000 psi
BASE THICKNESS: 8 in. BASE TYPE: SELECT BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.80×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.926	.748	.576	.354	.252	239.	77.9	--	7.6	0.00	0.4	5.2	4.9	2.6

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 26. BENSON BEFORE OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 8-19-75 SEASONAL FACTOR: 1.25 TEST TEMPERATURE: 81°F
A.C. THICKNESS: 7.75 in. EEXP: 305,000 psi EDES: 500,000 psi
BASE THICKNESS: 4 in. BASE TYPE: SELECT BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84 x10^6 EAL ADJUSTED DESIGN LIFE+: 6.04 x10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.188	.668	.430	.249	.140	100.	15.7	--	8.5	5.23	2.4	4.8	7.2	2.9

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 27. BENSON AFTER OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0 TEST DATE: 3-10-76 SEASONAL FACTOR: .79 TEST TEMPERATURE: 80°F

A.C. THICKNESS: 9.5 in. EEXP: 325,000 psi EDES: 500,000 psi

BASE THICKNESS: 4 in. BASE TYPE: SELECT BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1

DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 3.84×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.742	.562	.314	.198	.131	443.	5.1	--	16.1	0.00	0.0	2.8	2.8	0.0

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING

NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 28. LUPTON BEFORE OVERLAY-ARIZONA

REGIONAL FACTOR: 1.0TEST DATE: 6-17-75 SEASONAL FACTOR: .97 TEST TEMPERATURE: 81°F
A.C. THICKNESS: 4 in. EEXP: 305,000 psi EDES:500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.71×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	1.152	.912	.664	.524	.358	241.	--	--	7.1	1.53	2.1	4.7	7.0	2.7

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 29. LUPTON AFTER OVERLAY-ARIZONA

REGIONAL FACTOR:1.0 TEST DATE:11-19-75 SEASONAL FACTOR: .97 TEST TEMPERATURE: 40°F
A.C. THICKNESS: 7.5 in. EEXP:1,250,000 psi EDES:500,000 psi
BASE THICKNESS: 6 in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 4.84×10^6 EAL ADJUSTED DESIGN LIFE+: 4.71×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.642	.534	.456	.372	.302	458.	--	--	10.4	0.00	0.0	6.0	1.6	2.1

\+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 30. I-75 SUMTER COUNTY - FLORIDA

REGIONAL FACTOR: 1.0 TEST DATE: 7-11-78 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 86°F
A.C. THICKNESS: 4.5 in. EEXP: 250,000 psi EDES: 475,000 psi
BASE THICKNESS: 22 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 13.07×10^6 EAL ADJUSTED DESIGN LIFE+: 13.07×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
270	.440	.230	.130	.100	.080	1163.	13.2		35.9	0.00	0.0	1.7	1.0	0.0
280	.480	.320	.210	.180	.150	613.	110.4	96.0	22.0	0.00	0.0	2.0	1.6	0.0
290	.670	.430	.240	.170	.150	100.	52.2		17.8	5.58	0.0	3.5	4.6	1.1
300	.680	.430	.240	.160	.140	380.	56.0		18.6	2.20	0.0	3.5	4.7	1.2
310	.870	.550	.280	.170	.130	669.	37.8		17.2	1.39	0.7	4.8	7.3	2.5
320	.720	.480	.280	.200	.150	716.	55.6		16.7	0.00	0.0	3.9	5.3	1.5
330	.590	.380	.220	.160	.140	329.	69.4		19.6	1.79	0.0	3.0	3.4	0.5
340	.610	.420	.240	.170	.140	1139.	64.5		19.8	0.00	0.0	3.1	3.7	0.6
350	.610	.360	.170	.110	.090	516.	54.1		25.3	1.14	0.0	3.1	3.7	0.6
360	.530	.350	.180	.130	.110	1110.	69.0		24.9	0.00	0.0	2.5	2.4	0.0
370	.590	.350	.220	.150	.130	2527.	46.1		26.2	0.00	0.0	3.0	3.4	0.5
380	.770	.540	.280	.190	.160	1567.	41.5		17.8	0.00	0.0	4.2	5.9	1.8
1	1.410	.900	.610	.440	.370	1160.	22.5		9.1	0.70	4.5	7.0	12.2	6.8
2	.630	.400	.220	.160	/130	382.	61.1		19.8	1.79	0.0	3.3	4.0	0.8
3	.310	.200	.110	.060	.050	2026.	108.7		46.7	0.00	0.0	0.0	0.0	0.0
271	.410	.240	.150	.110	.090	2232.	80.1		34.4	0.00	0.0	1.2	0.6	0.0
281	.540	.330	.250	.200	.170	2795.	67.8		21.4	0.00	0.0	2.6	2.6	0.1
291	.740	.450	.250	.170	.150	100.	46.4		16.5	6.00	0.0	4.0	5.5	1.6
301	.760	.380	.200	.150	.120	569.	40.6		22.6	1.74	0.0	4.1	5.8	1.7
311	.570	.330	.210	.130	.110	4020.	32.7		38.8	0.00	0.0	2.8	3.1	0.4
321	.760	.420	.230	.170	.140	851.	41.7		21.0	0.15	0.0	4.1	5.8	1.7
331	.640	.370	.200	.170	.130	276.	61.9	76.3	23.1	2.75	0.0	3.3	4.1	0.8
AVERAGE										1.15	0.24	3.21	4.11	1.10
STANDARD DEVIATION										1.74	0.96	1.38	2.58	1.46

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 31. DELAWARE COUNTY ROAD 72 - OHIO

REGIONAL FACTOR: 1.0 TEST DATE: 11-2-75 SEASONAL FACTOR: .96 TEST TEMPERATURE: 50^{o}F
A.C. THICKNESS: 5.5 in. EEXP: 1,400,000 psi EDES: 750,000x10^{3}
BASE THICKNESS: 14 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 3.06x10^{6} EAL ADJUSTED DESIGN LIFE+: 2.94 x10^{6} EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
1	.830	.670	.420	.290	.200	817.	21.6		10.0	0.00	0.0	5.0	2.3	1.3
2	.940	.750	.460	.300	.200	698.	16.8		9.3	0.05	0.1	5.7	3.5	2.1
3	.770	.600	.370	.250	.180	582.	32.6		10.7	0.00	0.0	4.7	1.7	0.9
4	.580	.430	.270	.200	.160	225.	83.2		14.2	0.00	0.0	3.4	0.2	0.0
5	.500	.400	.280	.210	.160	539.	100.7		14.3	0.00	0.0	2.8	0.0	0.0
6	.770	.590	.350	.230	.150	564.	28.5		11.3	0.00	0.0	4.7	1.7	0.9
7	.840	.650	.370	.230	.160	640.	19.0		11.2	0.08	0.0	5.1	2.4	1.4
8	.980	.760	.460	.310	.220	458.	24.2		8.6	0.79	0.4	5.8	3.8	2.5
9	.840	.630	.360	.240	.180	405.	30.4		10.5	0.63	0.0	5.1	2.4	1.4
10	.910	.760	.480	.350	.230	1169.	9.7		10.6	0.00	0.0	5.5	32.	1.9
11	1.270	.940	.550	.350	.250	238.	20.9		7.0	3.13	2.4	7.0	6.3	4.0
12	1.130	.870	.540	.360	.260	341.	24.4		7.3	1.78	1.6	6.4	5.2	3.5
13	1.020	.820	.500	.320	.210	726.	12.4		9.1	0.26	0.7	6.0	4.2	2.7
14	.870	.690	.420	.270	.190	706.	19.4		10.0	0.00	0.0	5.3	2.8	1.6
15	.850	.660	.380	.240	.160	645.	18.8		11.0	0.07	0.0	5.2	2.6	1.4
16	.930	.710	.400	.240	.140	549.	16.1		10.8	0.75	0.1	5.6	3.6	2.0
17	1.290	.910	.480	.250	.140	514.	11.0		9.5	2.93	2.4	7.1	6.4	4.6
AVERAGE										0.62	0.45	5.32	3.08	1.89
STANDARD DEVIATION										1.02	0.84	1.09	1.79	1.27

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS   * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa   1 in. = 25.4 mm

TABLE 32. 02-515-97 CALIFORNIA

REGIONAL FACTOR: 1.0 TEST DATE: 5-2-78 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 79°F
A.C. THICKNESS: 4.8 in. EEXP: 340,000 psi EDES: 410,000 psi
BASE THICKNESS: 6.0 in. BASE TYPE: GRAN BASE SLOPE(B1): .45 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 5.42 x10^6 EAL ADJUSTED DESIGN LIFE+: 5.42 x10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
0	1.410	1.080	.570	.230	.100	1451.	2.4		19.0	0.00	3.6	6.2	9.1	0.9
1	1.680	1.230	.630	.290	.150	913.	3.2		8.1	1.97	4.8	7.0	10.4	1.9
2	1.410	1.080	.600	.290	.150	1443.	2.9		10.9	0.00	3.6	6.2	9.1	0.9
3	1.500	1.170	.600	.270	.139	1141.	3.4		8.7	1.16	4.0	6.4	9.6	1.2
4	1.350	1.020	.510	.270	.080	1111.	4.3		9.2	1.11	3.3	6.0	8.7	0.7
5	1.170	.870	.390	.130	.050	1201.	4.1		15.8	0.43	2.4	5.3	7.5	0.1
6	1.320	.990	.510	.220	.100	1336.	3.2		13.4	0.19	3.2	5.8	8.5	0.6
7	1.410	1.110	.600	.260	.110	1556.	2.3		16.1	0.00	3.6	6.2	9.1	0.9
8	1.380	1.050	.540	.220	.080	1392.	2.6		18.4	0.00	3.5	6.0	8.9	0.8
9	1.650	1.290	.690	.290	.120	1304.	2.0		15.2	0.60	4.7	6.8	10.3	1.8
10	1.680	1.230	.720	.320	.150	841.	4.3		6.3	2.23	4.8	7.0	10.4	1.9
11	1.380	1.080	.600	.270	.120	1668.	2.3		17.4	0.00	5.5	6.0	8.9	0.8
12	1.440	1.080	.570	.250	.120	1276.	2.8		12.4	0.56	3.7	6.2	9.3	1.0
13	1.620	1.260	.690	.340	.200	1129.	3.4		7.1	1.34	4.5	6.8	10.2	1.7
14	1.320	1.080	.690	.370	.230	2449.	2.1		13.1	0.00	3.2	5.8	8.5	0.6
15	1.470	1.140	.660	.300	.160	1519.	2.5		11.8	0.00	3.9	6.3	9.4	1.1
16	1.500	1.170	.690	.320	.150	1554.	2.3		12.4	0.00	4.0	6.4	9.6	1.2
17	1.550	.990	.570	.240	.110	1096.	4.6		8.6	1.20	3.3	6.0	8.7	0.7
18	1.650	1.260	.690	.280	.110	1400.	1.6		39.3	0.02	4.7	6.8	10.3	1.8
AVERAGE										0.57	3.81	6.27	9.29	1.08
STANDARD DEVIATION										0.72	0.65	0.45	0.79	0.51

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

TABLE 33. 03 - BUTLER - CALIFORNIA

REGIONAL FACTOR:1.0 TEST DATE:4-27-78 SEASONAL FACTOR: 1.0 TEST TEMPERATURE: 88°F
A.C. THICKNESS: 3.0in. EEXP: 215,000 psi EDES:410,000 psi
BASE THICKNESS: 8.0in. BASE TYPE: CTB BASE SLOPE(B1): .00 SUBGRADE SLOPE(B2): 1.1
DESIGN LIFE: 2.96×10^6 EAL ADJUSTED DESIGN LIFE+: 2.96×10^6 EAL

	MEASURED DEFLECTIONS (MILS)					COMPUTED MODULI (10^3 psi)				REQUIRED OVERLAY (in.)				
STAT.	W1	W2	W3	W4	W5	A.C.	BASE	SUBB	SUBG	OAF	CA.	LA.	MISS.	UTAH
0	.660	.470	.270	.160	.110	7731.	32.4		15.6	0.00	0.0	1.6	0.0	0.0
1	1.150	.690	.340	.230	.160	3111.	18.8		10.3	0.00	0.9	3.7	2.5	0.0
2	1.110	.810	.440	.270	.190	3972.	23.1		9.0	0.00	1.3	3.9	3.0	0.0
3	1.230	.810	.390	.260	.190	2835.	14.9		8.7	0.00	2.1	4.4	3.9	0.0
4	.920	.630	.330	.210	.150	4470.	22.0		11.5	0.00	0.0	3.0	1.4	0.0
5	.780	.500	.230	.100	.070	5478.	12.6		17.5	0.00	0.0	2.3	0.5	0.0
6	.920	.540	.240	.110	.070	3052.	12.3		14.4	0.00	0.0	3.0	1.4	0.0
7	1.110	.660	.250	.110	.070	3094.	7.8		14.5	0.00	2.1	3.9	3.0	0.0
8	1.260	.840	.380	.190	.110	4215.	6.9		11.4	0.00	2.3	4.5	4.2	0.0
9	1.290	.810	.350	.200	.190	2530.	10.8		9.1	0.00	2.4	4.6	4.4	0.0
10	.970	.660	.330	.190	.120	5104.	13.3		12.0	0.00	0.3	3.3	1.8	0.0
11	1.230	.850	.440	.250	.170	4245.	11.3		9.2	0.00	2.1	4.4	3.9	0.0
12	1.260	.870	.400	.220	.160	2396.	15.0		8.5	0.00	2.3	4.5	4.2	0.0
13	1.230	.780	.320	.160	.100	3486.	7.2		12.0	0.00	2.1	4.4	3.9	0.0
14	1.020	.630	.310	.180	.120	160.	18.6		.0.0	6.04	0.7	3.5	2.3	0.0
AVERAGE										0.40	1.24	3.67	2.69	
SRANDARD DEVIATION										0.16	1.00	0.89	1.42	

+ ADJUSTED FOR SEASONAL AND REGIONAL FACTORS * CLASS 2 OR 3 CRACKING
NOTE: 1 psi = 6.895 kPa 1 in. = 25.4 mm

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APPENDIX A

PAVEMENT OVERLAY DESIGN PROCEDURES USING TWO-LAYER GRAPHICAL SOLUTIONS

A-1 BASIC ASSUMPTIONS

The graphical design procedures are an extension of the max-deflection procedures toward a formulation of a more rationalized and mechanistic design system. These graphical solutions are derived on the basis of shape of deflection basin and in-situ pavement conditions. The deflection parameters such as max-deflection and spreadability are used to estimate the effect of the shape of deflection basin on in-situ moduli and asphalt layer thickness. The basic assumptions of these design procedures are:

(1) The pavement structure is represented by a two-layer elastic system.
(2) The deflection basin defines the in-situ pavement characteristics.
(3) The pavement elastic layer system is characterized by Poisson's ratio, moduli and thickness. The in-situ response is very insensitive to the normal variations of the Poisson's ratio.
(4) The temperature and environment influence the shape of deflection basin and the pavement moduli; the adjustment of pavement max-deflection for an environment is not sufficient and doesn't take into consideration the variation of moduli with the environment.
(5) The design procedures might be divided into two approaches:
    1. Equivalent Modulus (stiffness)
    2. Equivalent Thickness

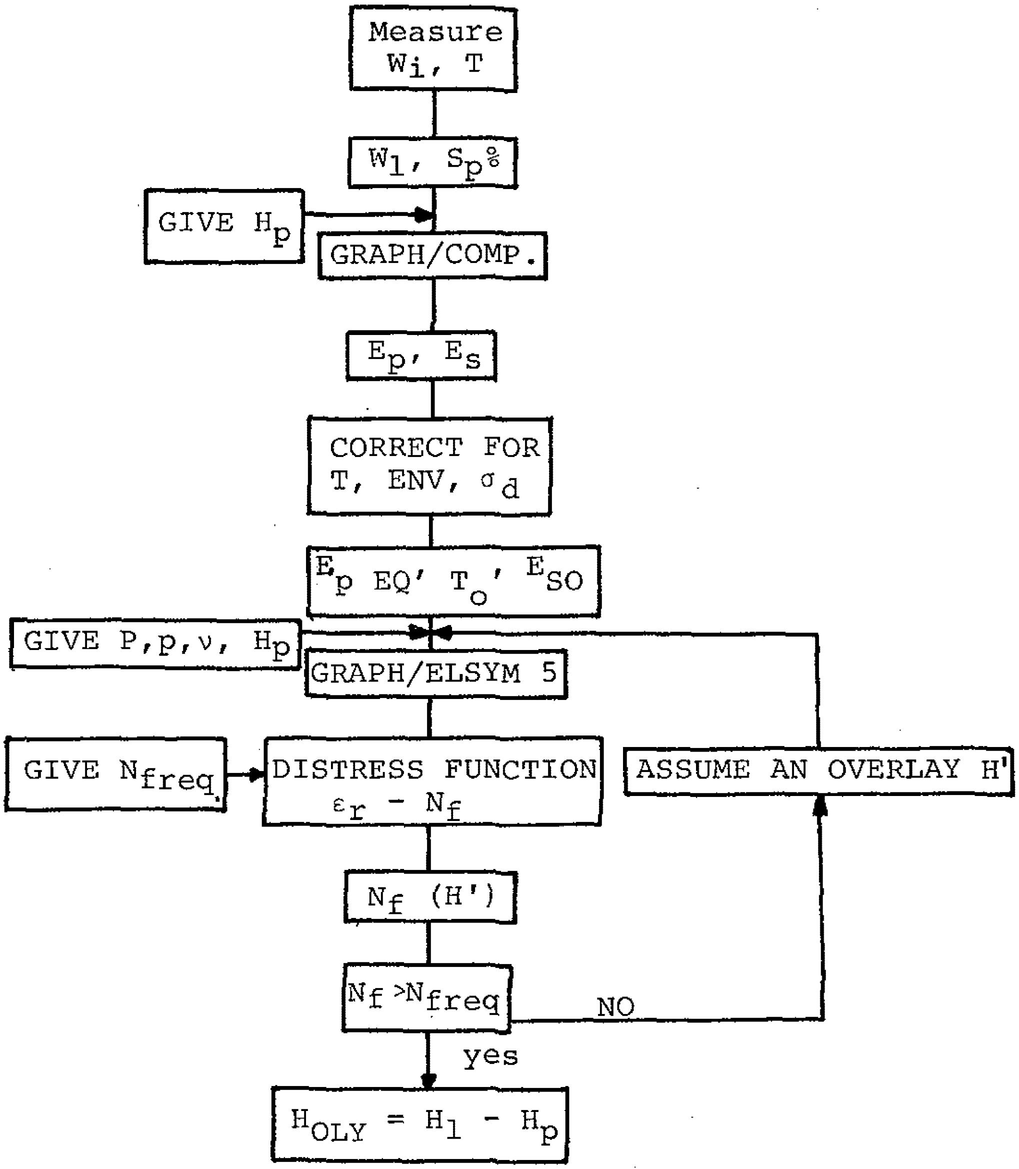
The flow charts and the description of each design approach is given in the following sections.

A-2 EQUIVALENT MODULUS CONCEPT

The equivalent modulus concept is based on the determination of in-situ stiffness using pavement deflection basin data. The flow chart for this design is shown in Figure 9. The input information required are:

(1) Field deflection data: max-deflection W_1 and spreadability SP%
(2) Estimate of pavement thickness H_p
(3) Pavement Distress Function
(4) Design Traffic Life N_{freq}

The procedure as shown in the flow chart Figure 9, estimates the pavement moduli, Ep, Es, makes appropiate environmental and stress corrections, and then by using a



SYMBOL IDENTIFICATION

W_i = measured deflections

T = average pavement temperature during test

E_p, E_s = pavement and subgrade moduli at test conditions

$E_{p\ EQ}$, T_o = equivalent pavement modulus at design temperature

E_{SO} = stress-corrected subgrade modulus

P = 1/2 axle load

p = contact pressure

ν = Poisson's Ratio

H_p = pavement thickness

N_{freq} = adjusted design life

σ_d = deviatoric stress at top of subgrade

ENV = environmental effects

FIGURE 9. OVERLAY DETERMINATION USING TWO-LAYER EQUIVALENT MODULUS CONCEPT

distress function estimates the remaining life of in-situ pavement. The temperature effect is taken into consideration not by adjusting measured deflection as in most other design schemes, but by adjusting the effective asphalt modulus from measurement temperature to design temperature. Correction for seasonal and climatic effects is done by adjusting the design life (N_{freq}) as discussed in Section 3.1.4, and the subgrade stress correction is performed in the manner described in Section A-3, Step 8.

If the remaining life is less than the estimated required life (N_{freq}), an overlay thickness is assumed and the remaining life, $N_f(H')$, determined from the reduced strains. This procedure is repeated until $N_f(H')$ is equal to, a greater than, the required life. It should be noted that in this procedure the effect of cracking in the existing pavement is reflected in the estimated in-situ modulus.

As was noted previously in Section 2.4.2, this procedure is somewhat cumbersome to implement, and since in-service pavements are rarely representable by a two-layer system, the necessary graphs are not included.

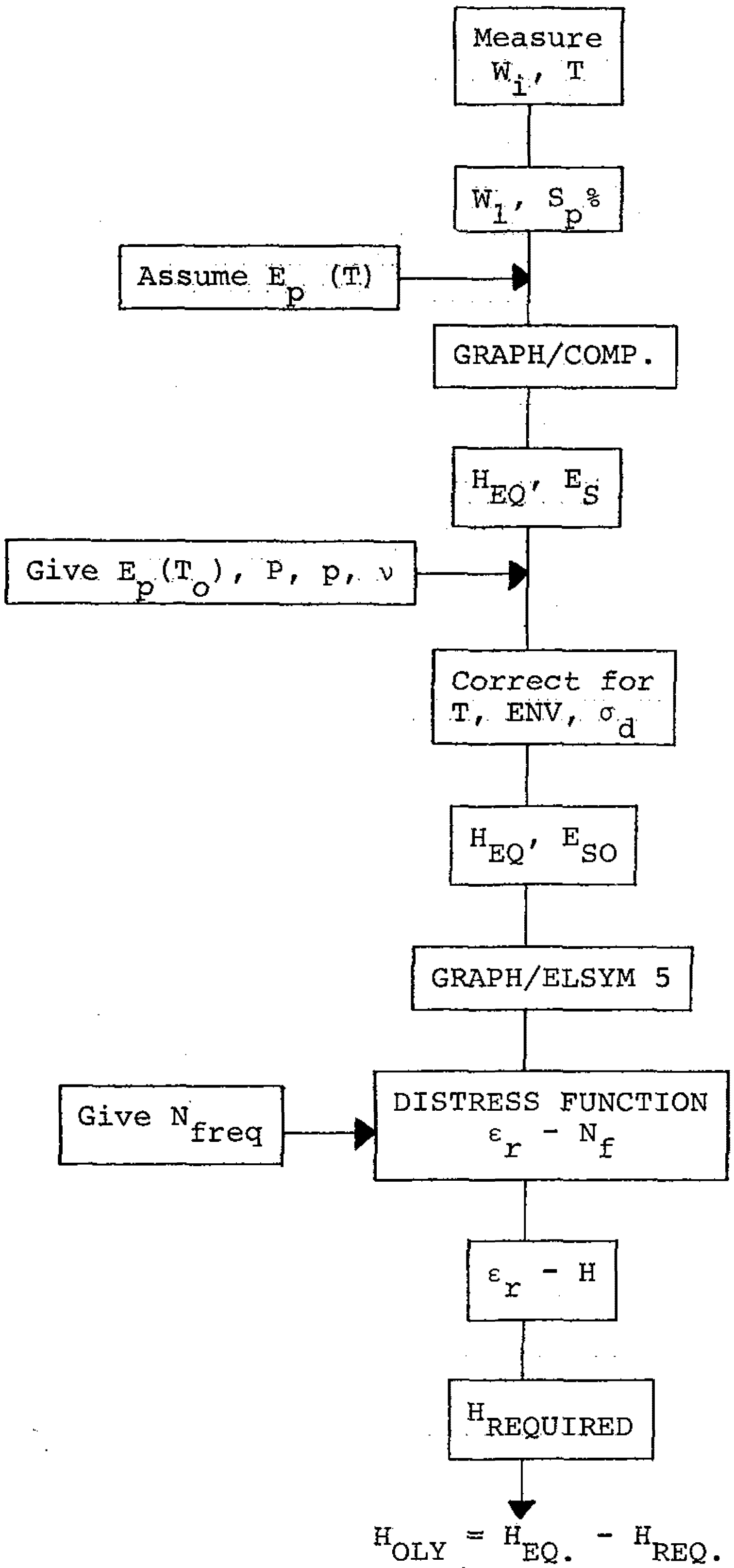
A-3 EQUIVALENT THICKNESS CONCEPT

The basic assumption and the mathematical principles in this design approach is similar to that of equivalent modulus. In this design procedure it is assumed that pavement deterioration is reflected not in the reduction of mixture properties, but rather in the effective thickness of the layer. That is, as the pavement undergoes aging and deterioration, it would retain its stiffness but would behave as if it would have been constructed with a lesser thickness.

The flow chart for the equivalent thickness concept is shown in Figure 10. In this design procedure, the following information is assumed known or available.

(1) Pavement Deflections
W1 max-deflection
SP Spreadability
(2) Pavement "as-built"
Modulus or stiffness Ep
(3) Pavement Distress Function
(4) Estimated Total Design Traffic N_{freq}

The design procedure involves two steps: namely pavement condition evaluation and overlay design. The step-by-step pavement evaluation procedures are as follows:



SYMBOL IDENTIFICATION

W_i = measured deflections

T = avg. pavement temperature during test

E_p(T) = pavement modulus at test temperature

H_{EQ} = equivalent thickness of existing pavement

E_S = subgrade modulus at test conditions

E_{SO} = stress-corrected subgrade modulus

σ_d = deviatoric stress at top of subgrade

$E_p(T_o)$ = pavement modulus at design temperature

P = 1/2 axle load

p = contact pressure

ν = Poisson's ratio

N_{freq} = adjusted design life

ENV = environmental effects

FIGURE 10. OVERLAY DETERMINATION USING TWO-LAYER EOUIVALENT THICKNESS CONCEPT

1. Using measured Dynaflect data, compute

$$S_p = 100* (\Sigma W_i)/5W_1$$

DO NOT correct for temperature

2. Using measured temperature data or a procedure such as Southgate curves (1), compute average asphalt temperature T_{av}

3. Using an asphalt modulus - temperature relationship, such as Figure 3 or 11, determine asphalt modulus $E_p(T_{av})$

4. Select from (W_1-S_p) curve shown in Figure 12 thru 23 for E_p just less than $E_p(T_{av})$, enter with W_1 and S_p and read off H_{p1} and H_{p1} and E_{s1}

5. Select W_1 - S_p curve for E_p just greater than $E_p(T_{av})$, enter with W_1 and S_p and read off H_{p2} and E_{s2}

6. Interpolate for $H_p(T_{av})$ and $E_s(T_{av})$ from

$$H_p(T_{av}) = H_{p1} + \frac{(H_{p1}-H_{p2}) * [\log E_{p1} - \log E_p(T_{av})]}{\log E_{p2} - \log E_{p1}} = \text{Heq}$$

$$E_s(T_{av}) = E_{s1} + \frac{(E_{s1}-E_{s2}) * [\log E_{p1} - \log E_p(T_{av})]}{\log E_{p2} - \log E_{p1}}$$

Example No. 1

Dynaflect measurements were made on an 8" (203mm) asphalt pavement having average temperature of 50°F (10°C). The measured values are:

W_1 = 1.11 mils (.0282mm)

W_2 = 0.87 mils (.0221mm)

W_3 = 0.53 mils (.0135mm)

W_4 = 0.30 mils (.0076mm)

W_5 = 0.19 mils (.0048mm)

The computed spreadability S_p = 54.05%

From Figure 3 the asphalt modulus is 1.05 x 10^6psi (7.24x10^9Pa)

Using Figure 20 along with W_1 = 1.11 mils (.0282mm) and S_p = 54.05%, H_{p1} = 4.45 in (113mm), E_{s1} = 22.5 x 10^3 (155 MPa)

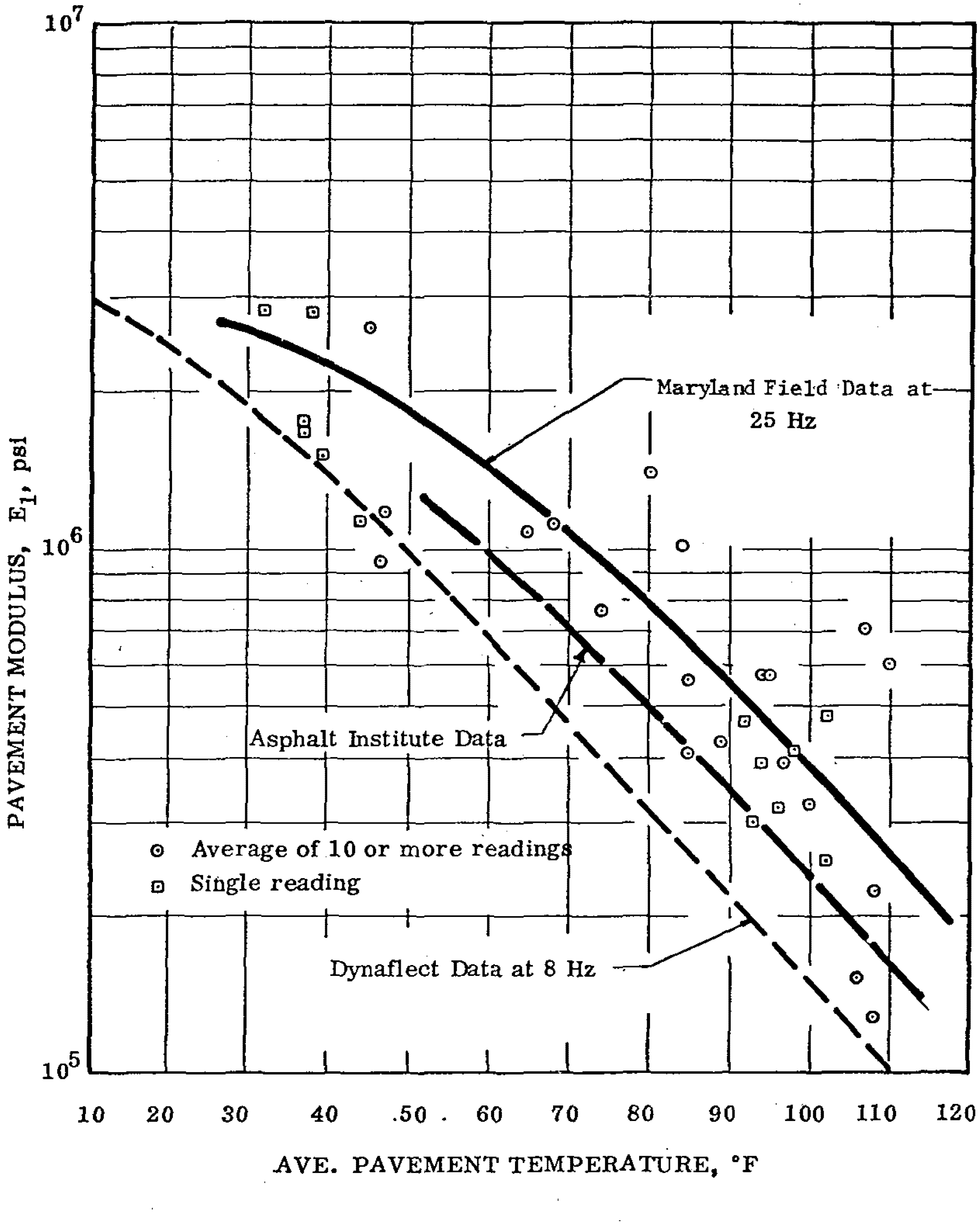


FIGURE 11. COMPARISON OF ASPHALTIC CONCRETE MODULI DETERMINED BY NDT WITH ASPHALT INSTITUTE DATA

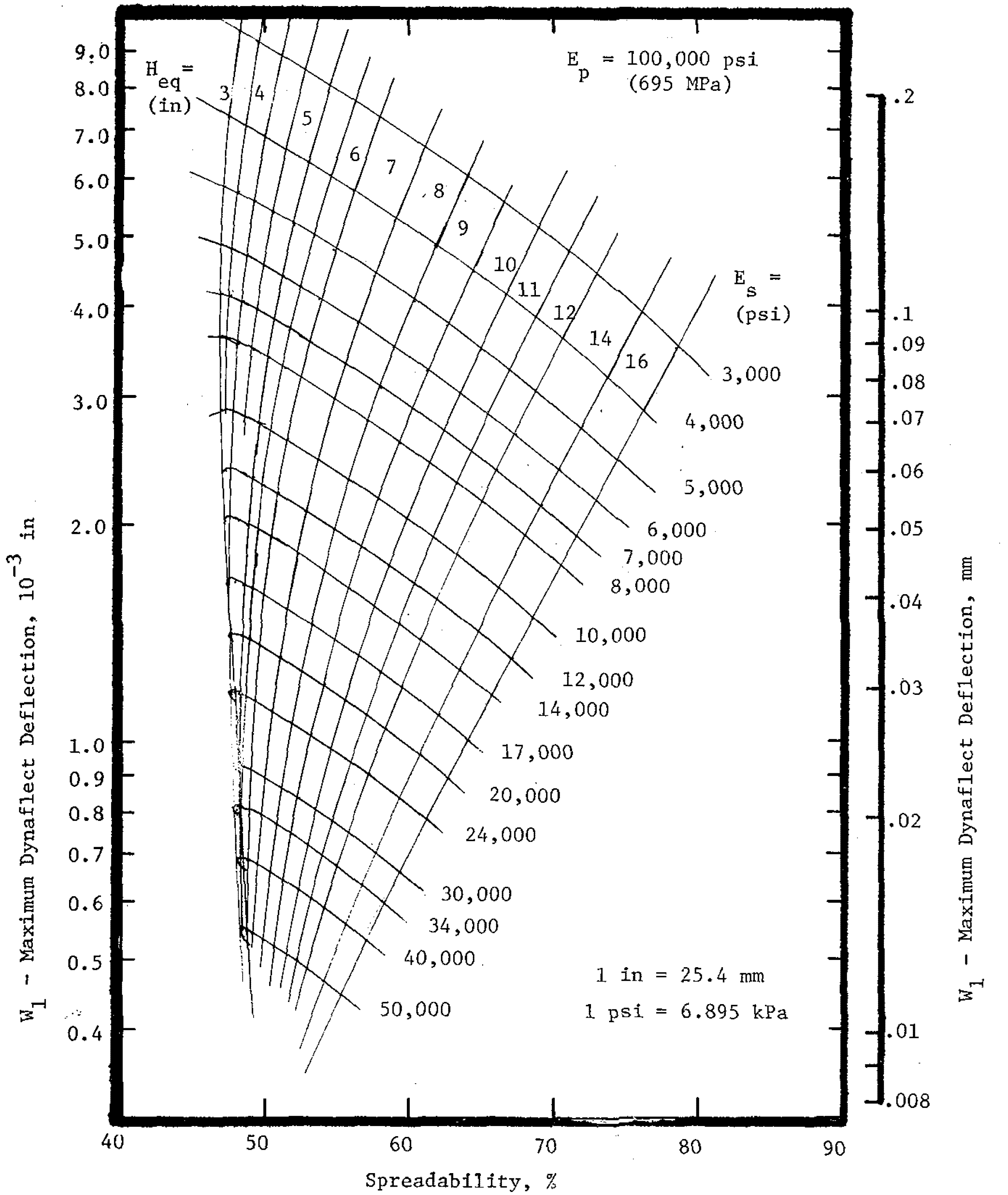


FIGURE 12. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 100,000 psi (695 MPa) MODULUS

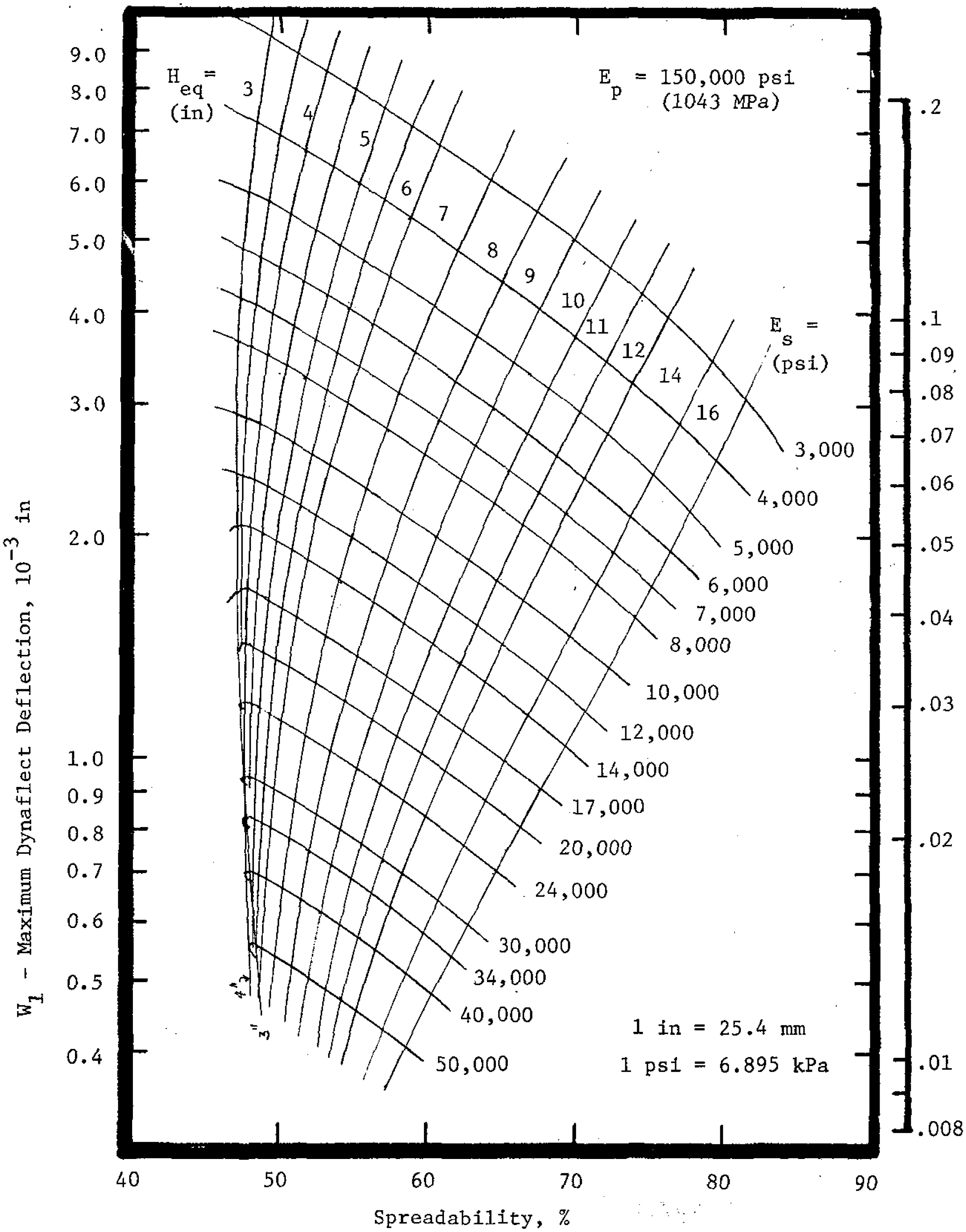


FIGURE 13. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 150,000 psi (1043 MPa) MODULUS

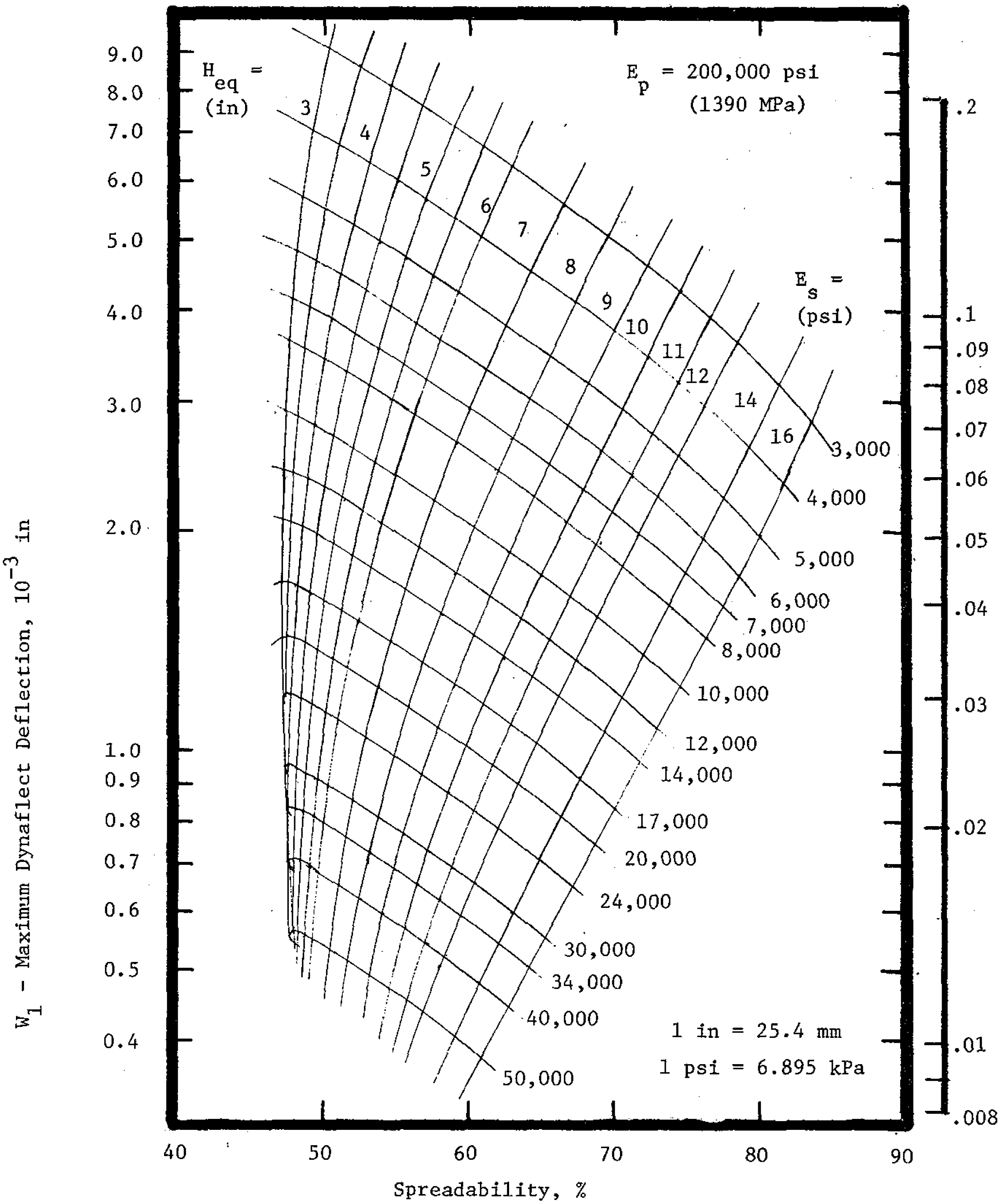


FIGURE 14. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 200,000 psi (1390 MPa) MODULUS

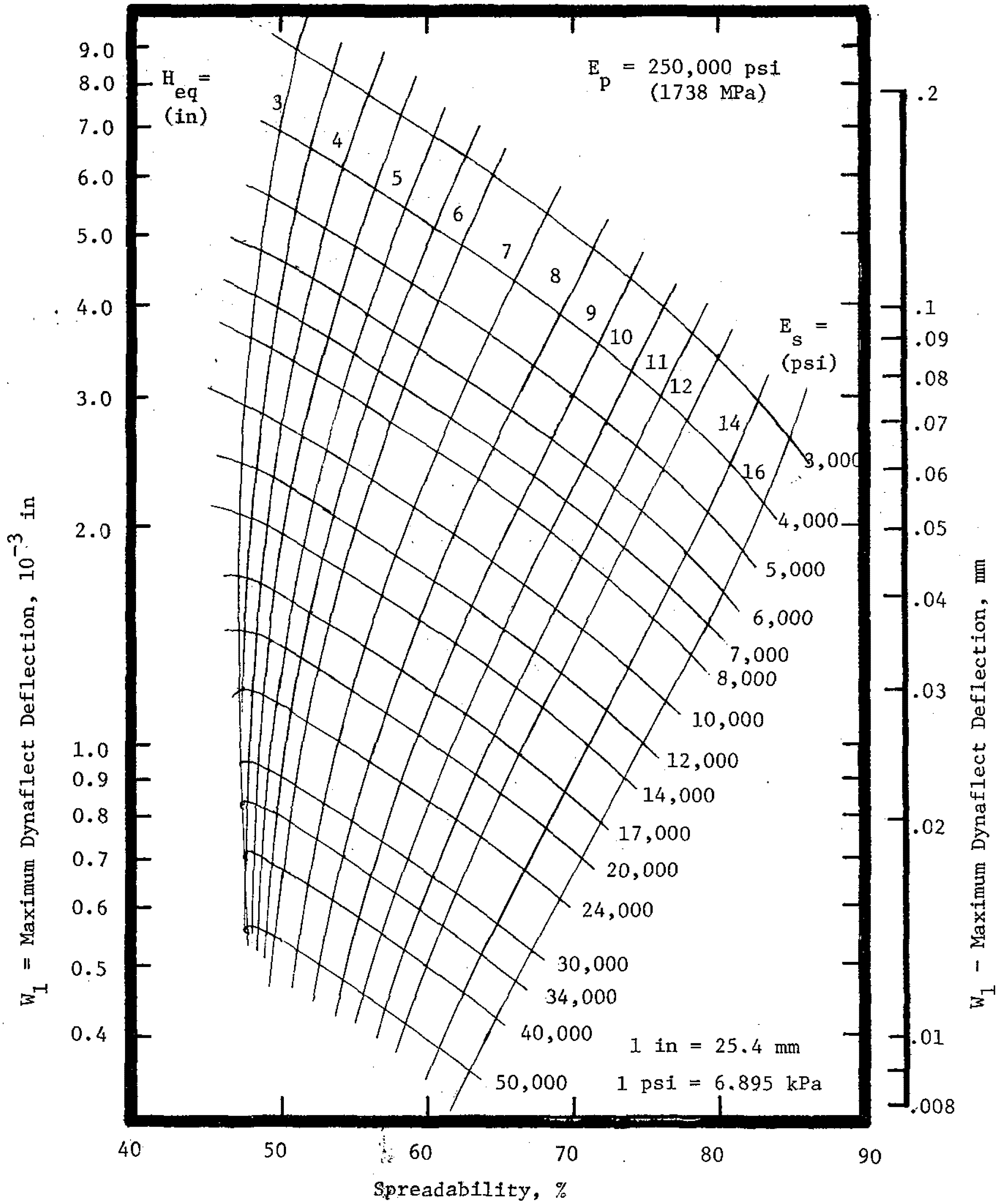


FIGURE 15. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 250,000 psi (1738 MPa) MODULUS

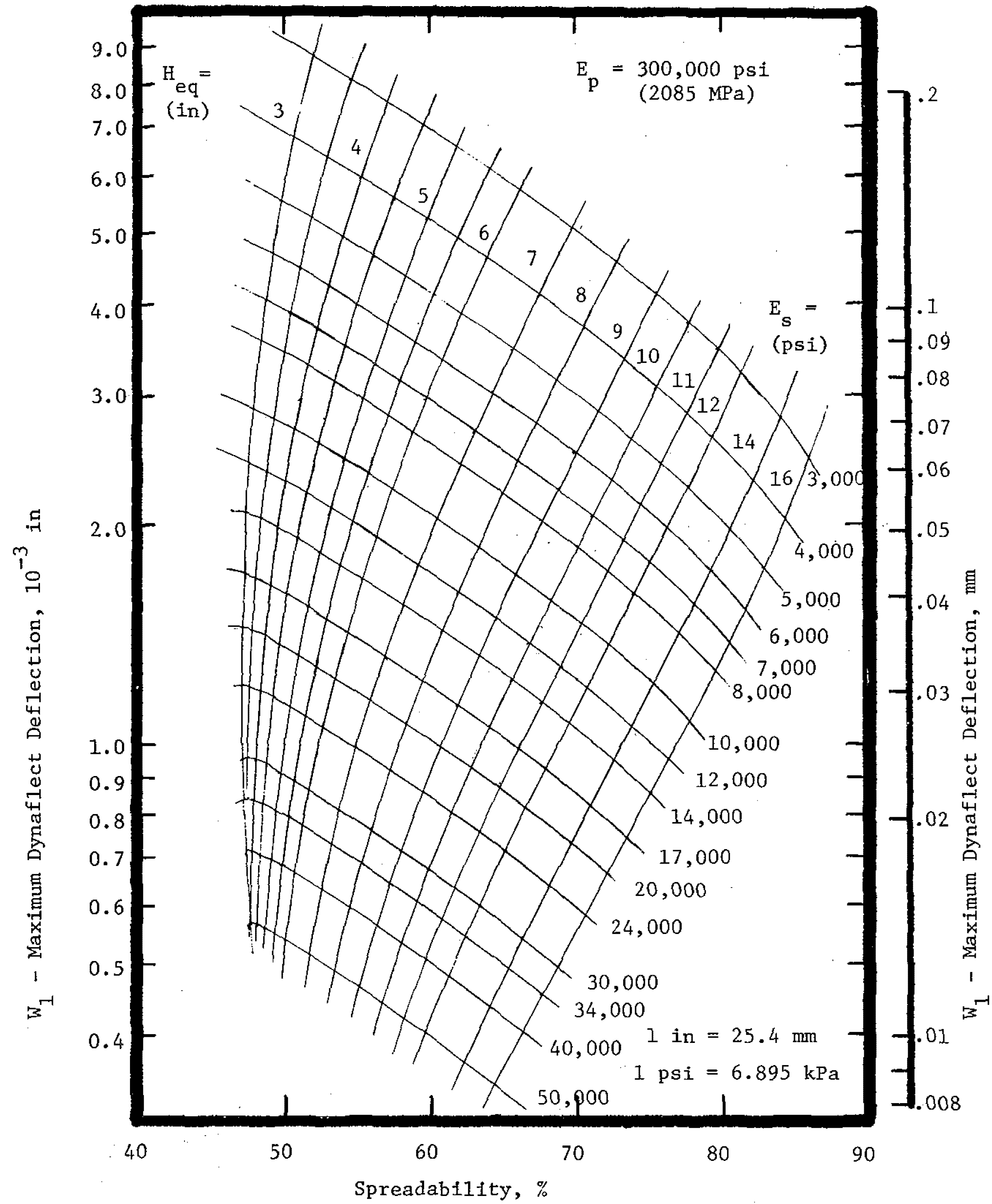


FIGURE 16. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 300,000 psi (2085 MPa) MODULUS

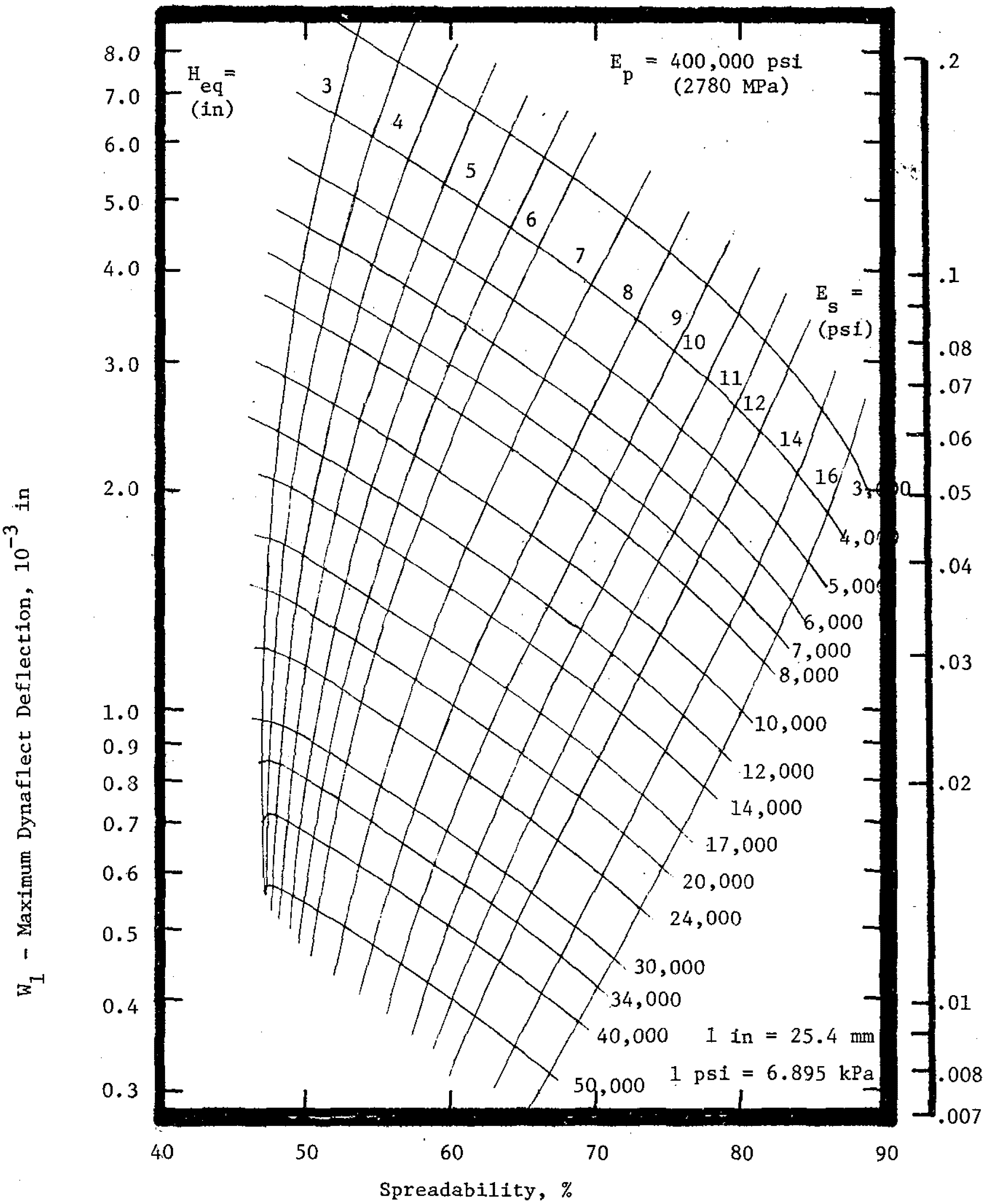


FIGURE 17. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 400,000 psi (2780 MPa) MODULUS

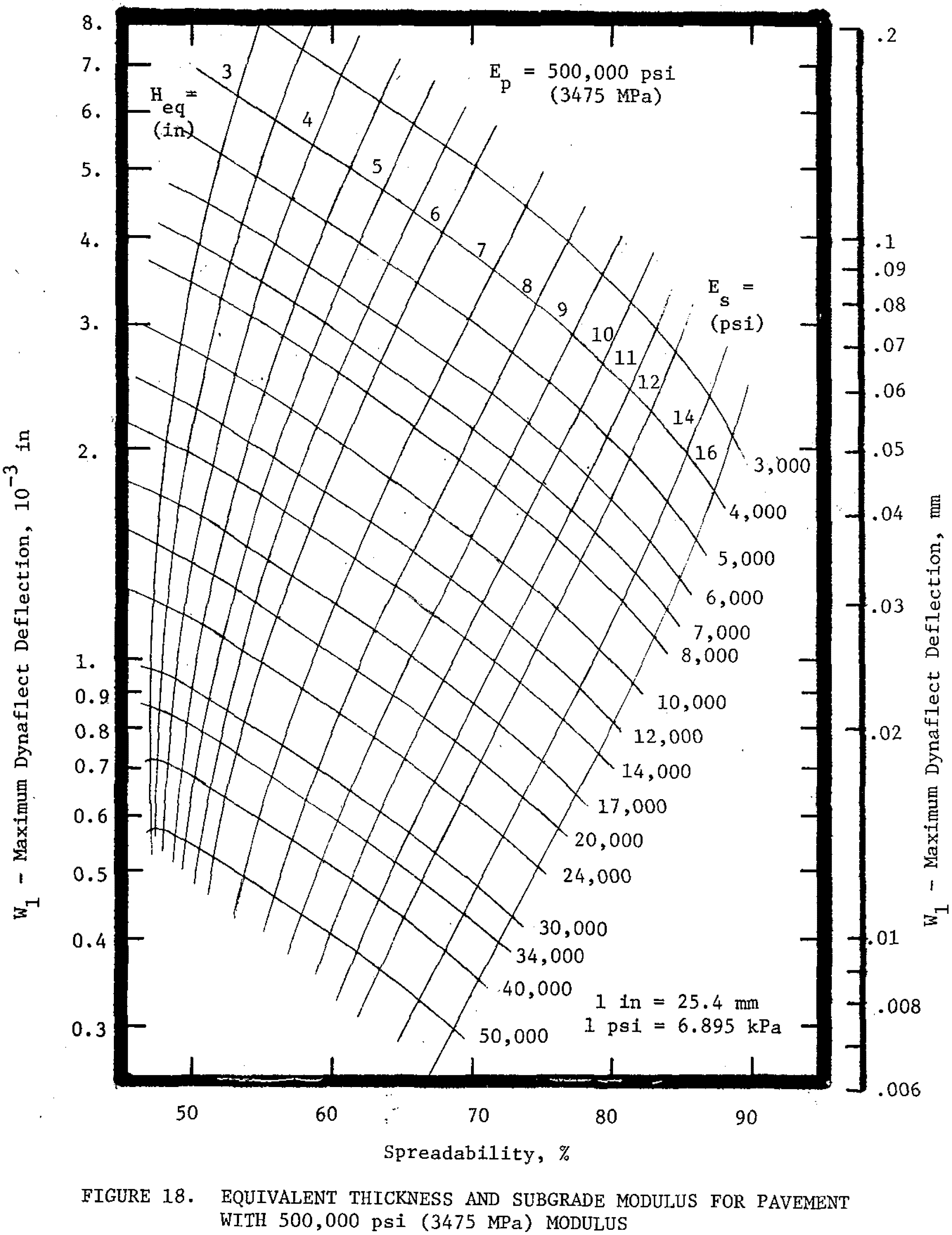


FIGURE 18. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 500,000 psi (3475 MPa) MODULUS

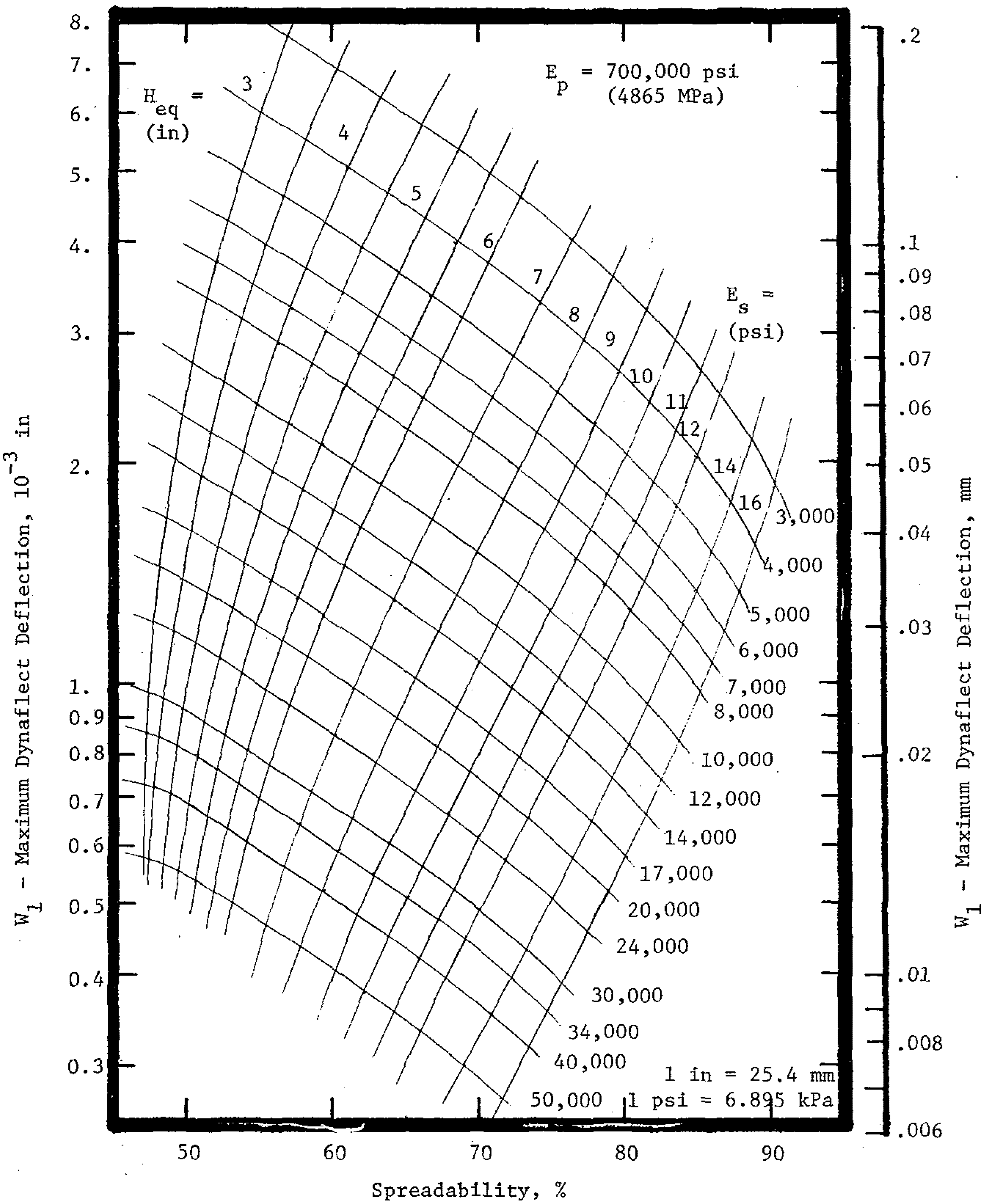


FIGURE 19. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 700,000 psi (4865 MPa) MODULUS

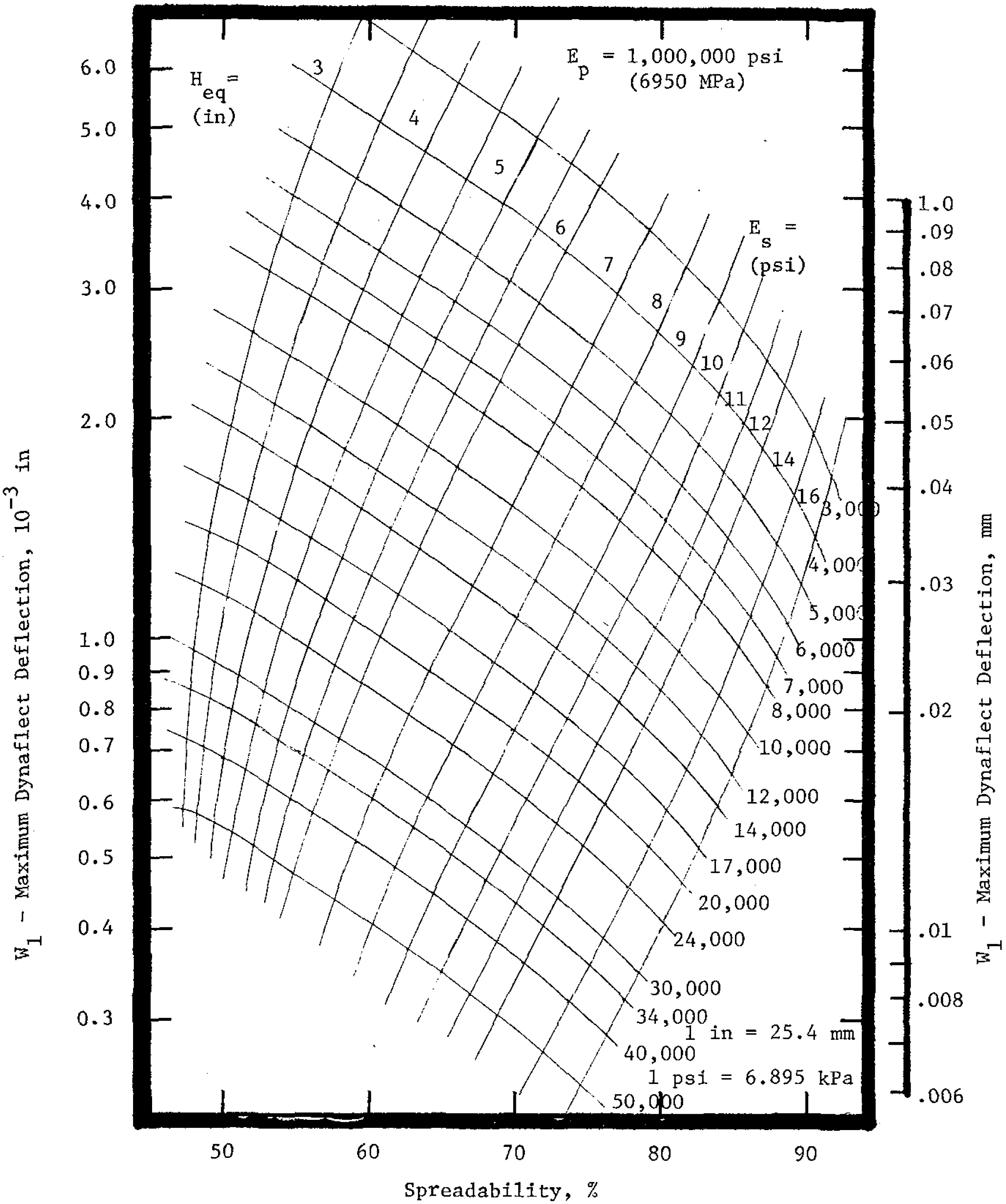


FIGURE 20. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 1,000,000 psi (6950 MPa) MODULUS

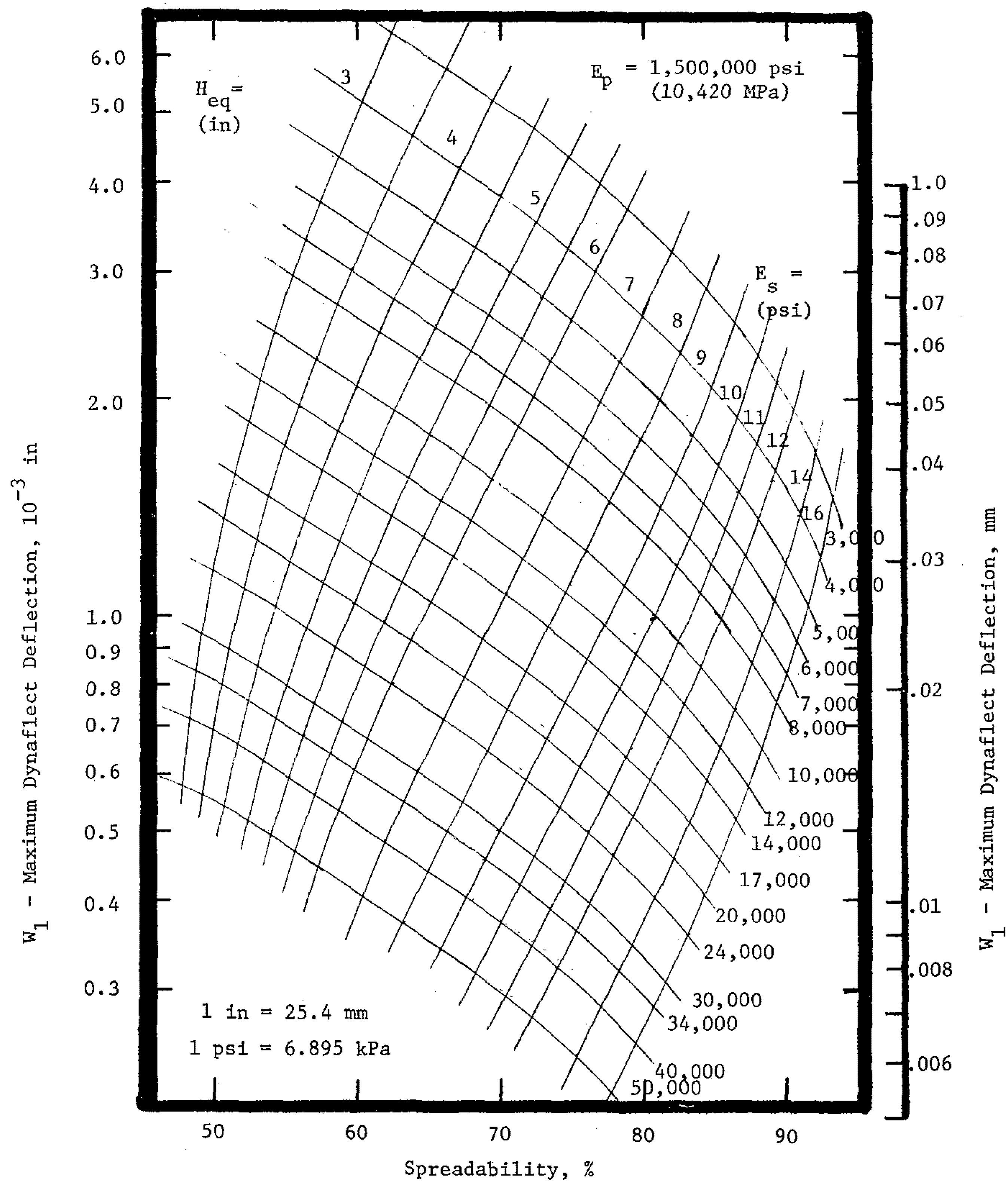


FIGURE 21. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 1,500,000 psi (10,420 MPa) MODULUS

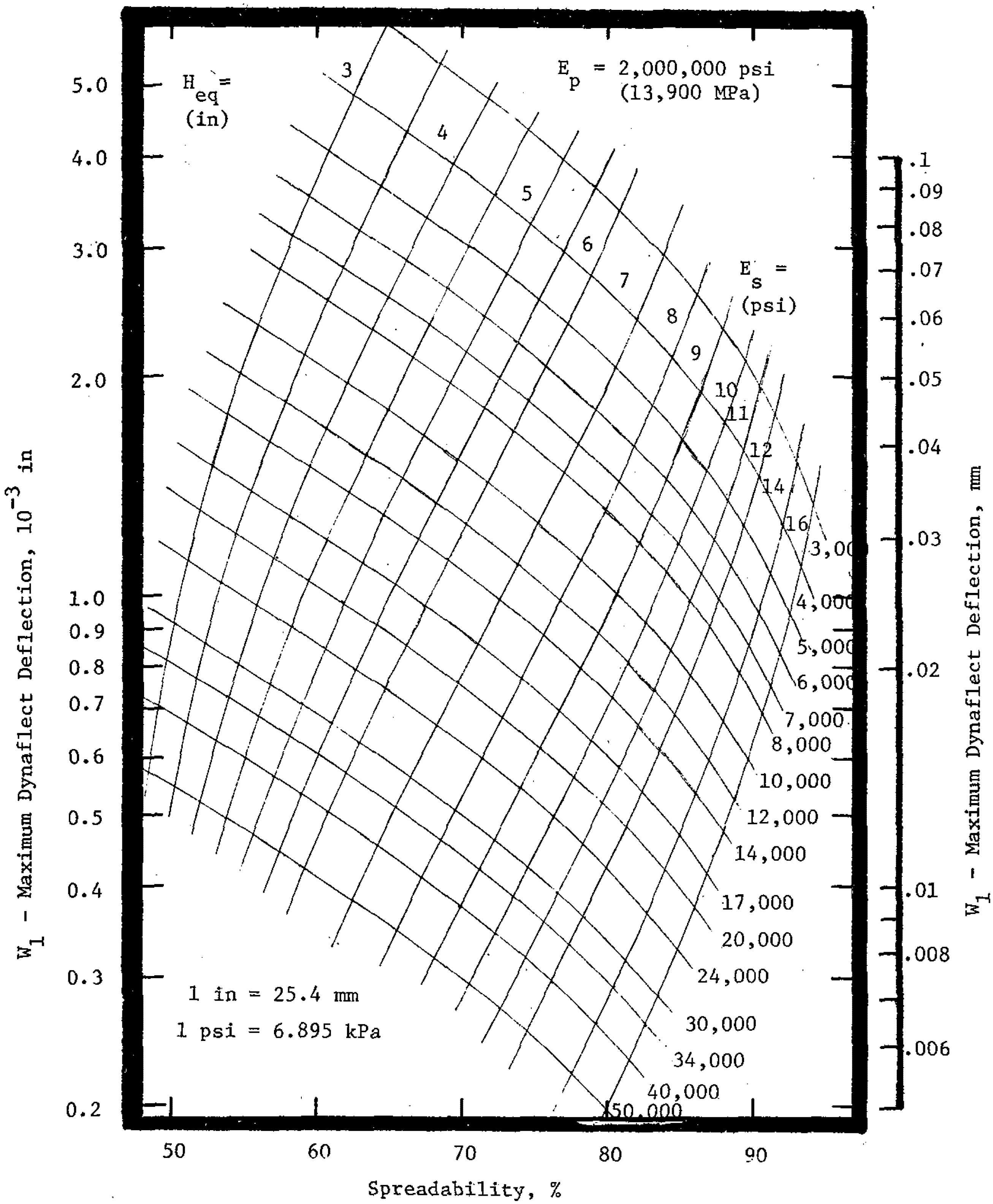


FIGURE 22. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 2,000,000 psi (13,900 MPa) MODULUS

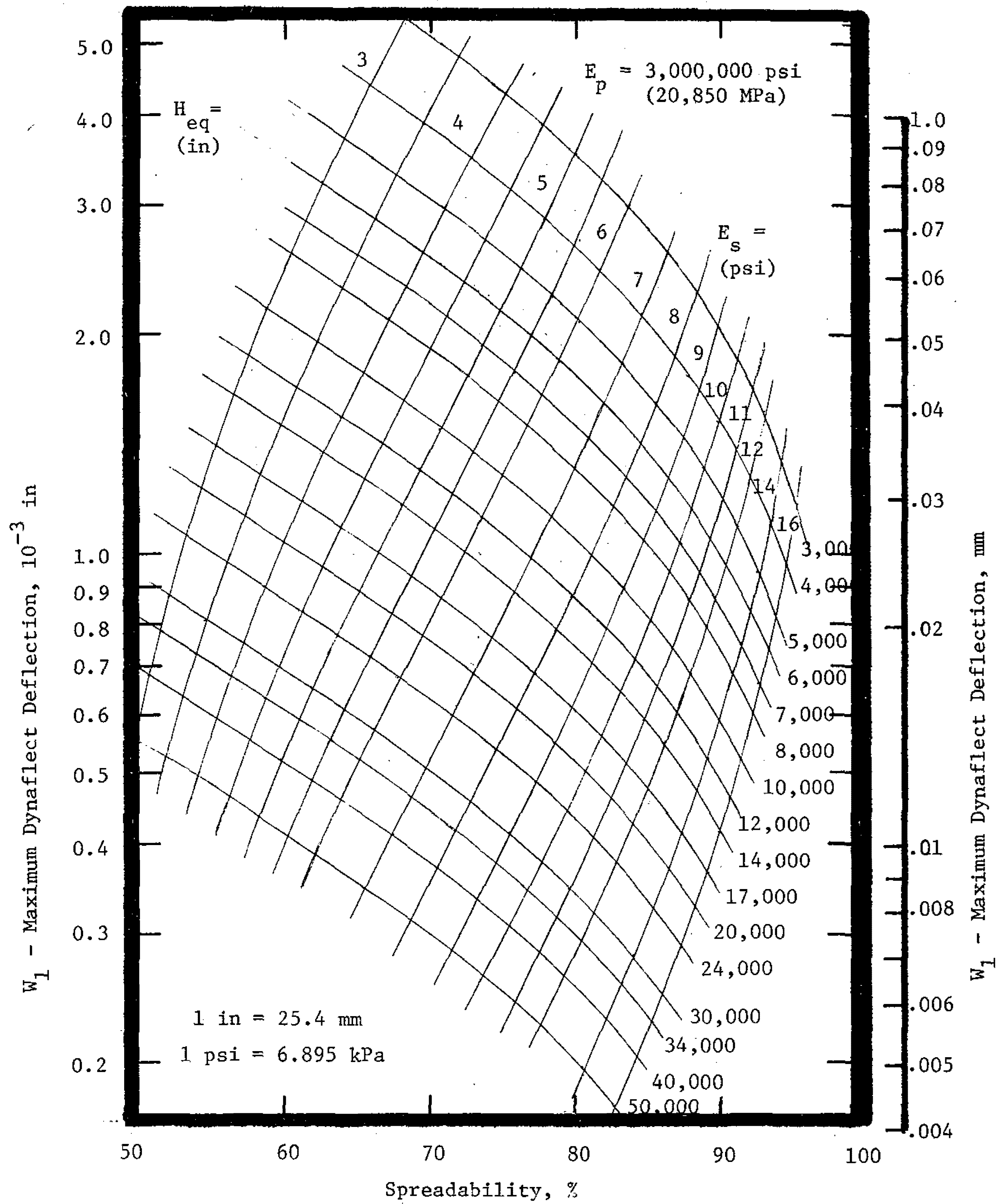


FIGURE 23. EQUIVALENT THICKNESS AND SUBGRADE MODULUS FOR PAVEMENT WITH 3,000,000 psi (20,850 MPa) MODULUS

Using Figure 21 H_{p2} = 3.85 in.(98mm), E_{s2} = 22.0 x10^3 (152MPa)

The interpolation equation yields:

Equivalent pavement thickness $H_p(T_{av})$ = 4.38 in. (111mm)

Subgrade modulus $E_s(T_{av})$ = 22.44 x 10^3psi (155MPa)

7. Repeat the above procedure (steps 1 through 6) for each location where deflection measurements were taken and then determine the 80th percentile values for H_{eq} and E_s. These latter values will be used in subsequent overlay computations.

8. Upon the completion of the pavement condition evaluation, it will be necessary to correct the support modulus E_s for stress - the procedure outlined in the FHWA-ARE (2) report (Asphalt Concrete Overlays of Flexible Pavements) and Figure 24 through 26 may be used. Note that some lab testing is required to determine E_s vs. σ_D for the particular subgrade, as shown in Figure 25. If lab testing is not possible, the relationship given in Figure 25 may be used.

   The correction procedure is as follows:

   a. Determine deviatoric stress for dynaflect from Figure 24

      (i) enter Figure 24 with total pavement thickness equal to H_{eq}
      (ii) draw a horizontal line to intersect surface modulus equal to $E_p(T_{av})$
      (iii) from this intersection, draw a vertical line to intersect subgrade modulus equal to E_s
      (iv) read off deviatoric stress value

   b. Establish relationship between subgrade modulus and deviatoric stress.

      (i) plot E_s on Figure 25 at deviatoric stress value obtained in step a
      (ii) through this point draw a line parallel to Lab curve - this becomes the analysis curve

   c. Determine deviatoric stress for 18 kip (80kN) axle loading from Figure 26.

      (i) enter Figure 26 with total pavement thickness equal to H_{eq}

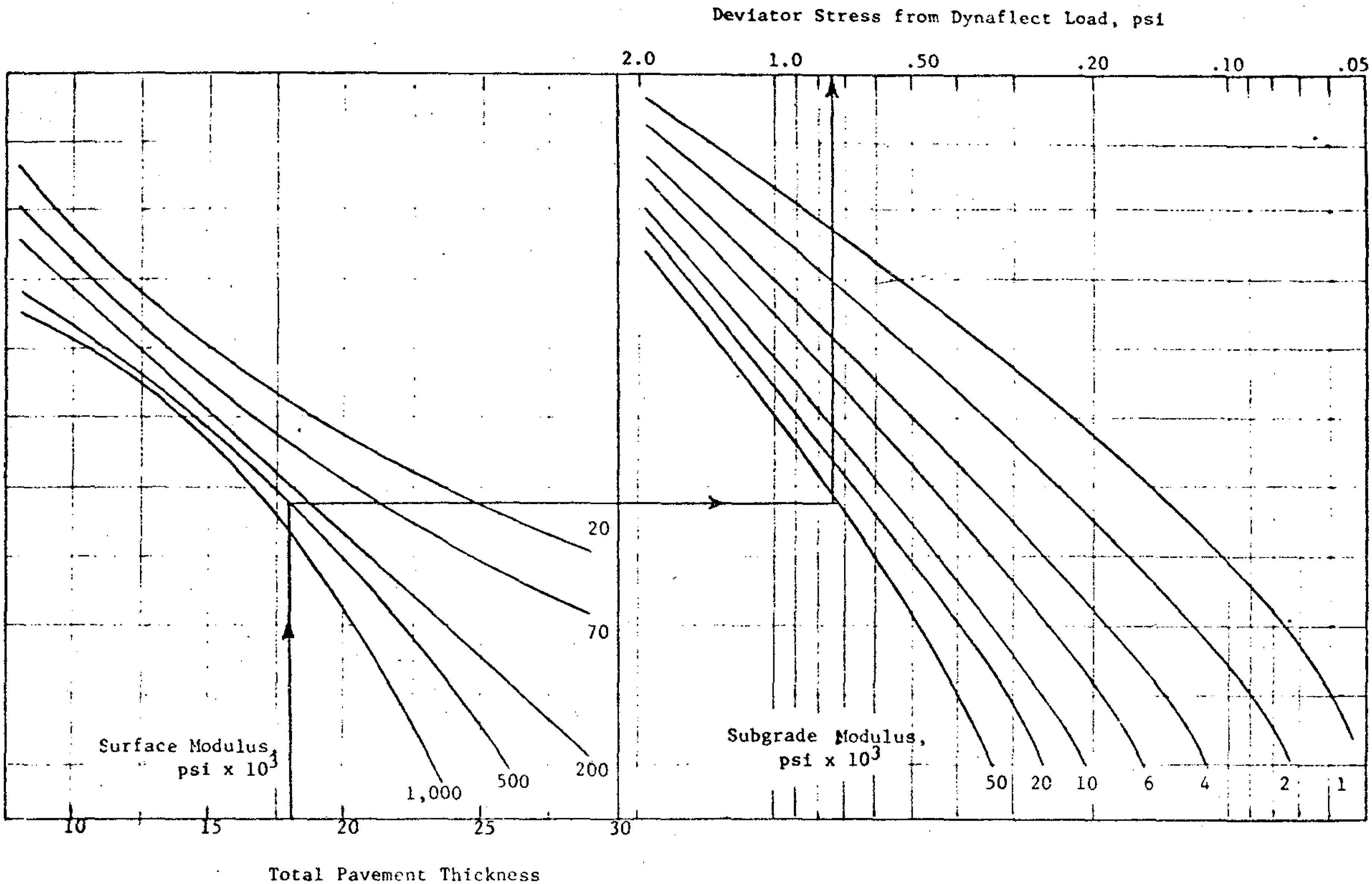


Figure 24 Analysis Chart For Subgrade Deviator Stress For Dynaflect Test Load

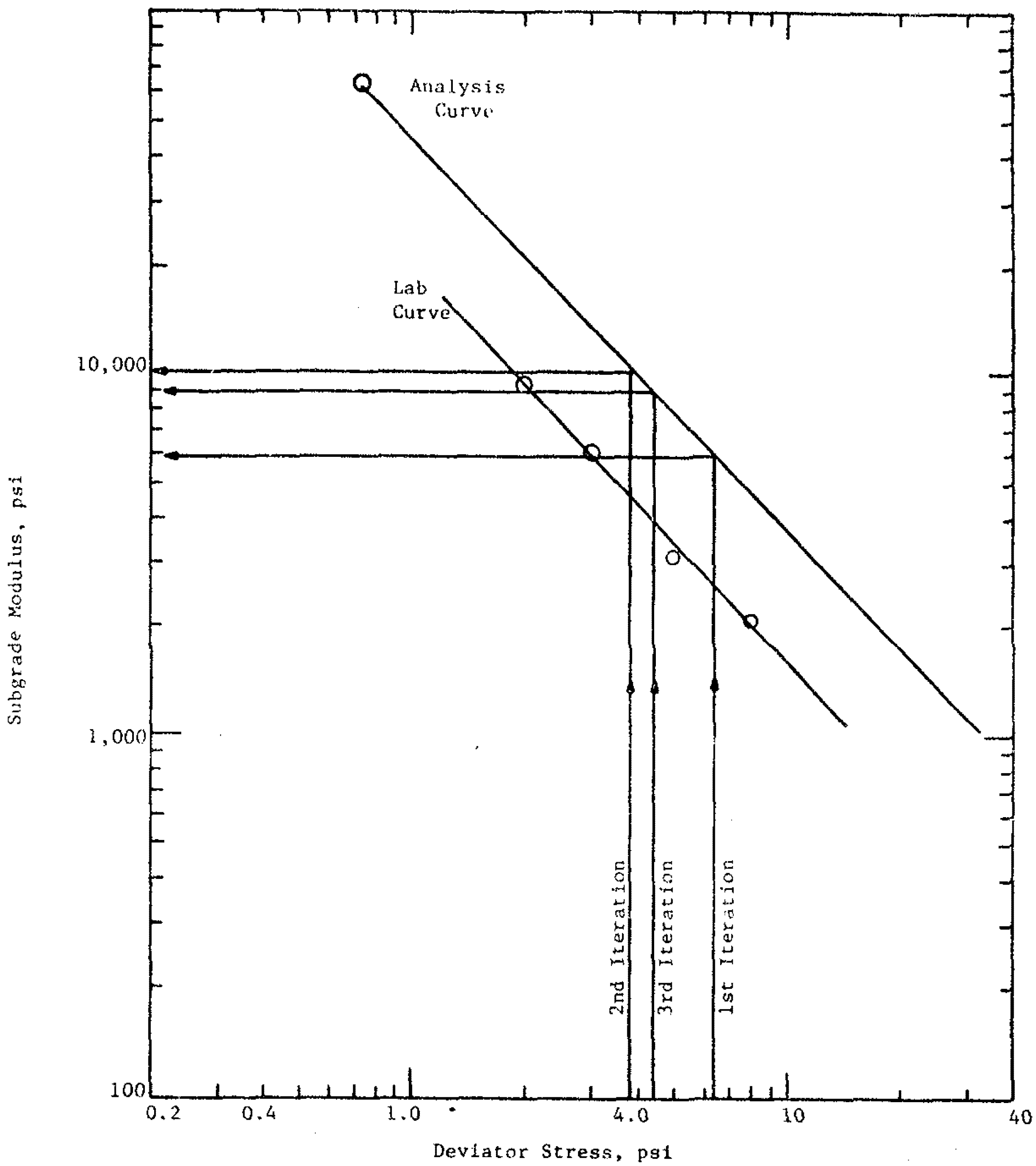


Figure 25 Relationship of resilient modulus and repeated deviator stress

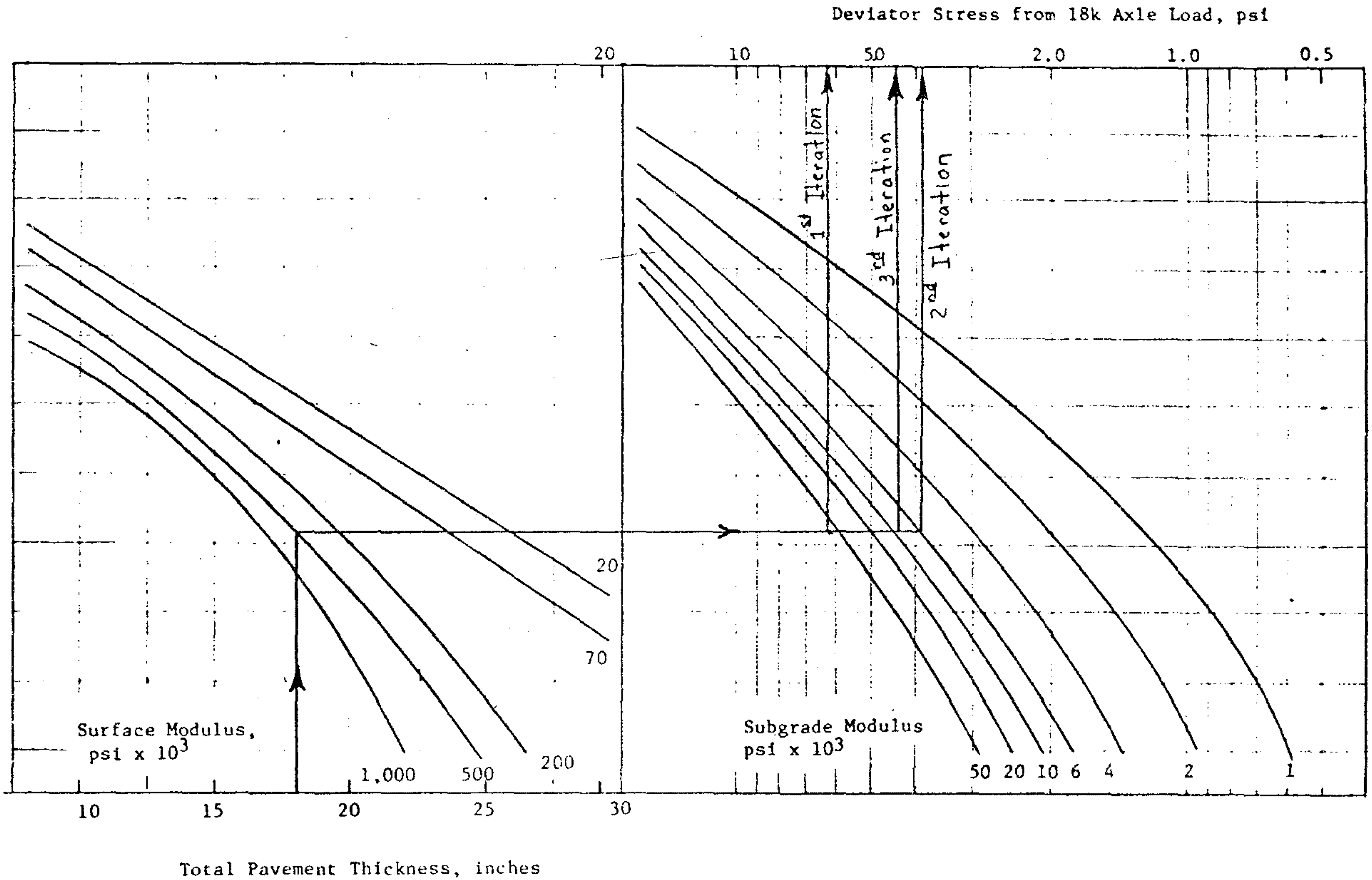


Figure 26 Analysis Chart For Subgrade Deviator Stress For 18kip Axle Design Load

(ii) draw a horizontal line to intersect surface modulus equal to design modulus (500,000 psi) (3.45×10^9 Pa)

(iii) draw a vertical line to intersect subgrade modulus equal to E_s

(iv) read off deviatoric stress value

d. Determine subgrade modulus corresponding to 18 kip (80kN) deviatoric stress (determined in step c) from Figure 25.

(i) enter Figure 25 with deviatoric stress determined in step c

(ii) draw a vertical line to intersect analysis curve

(iii) read off subgrade modulus corresponding to this deviatoric stress

e. Repeat step c and d until subgrade modulus value has converged - usually 3 iterations are adequate. This value is the design subgrade modulus E_{SD}.

9. Use Figure 27 or FHWA-ARE distress function (equation 4-10) to determine allowable tensile strain ε_r at bottom of asphalt layer.

10. Determine total asphalt thickness required (H req.) from Figure 28.

(i) on Figure 28 locate the point corresponding to ε_r and E_{SD}

(ii) interpolate between the thickness curves and determine the required thickness

11. Determine the amount of overlay required:

$$H_{OLY} = H_{req} - H_{eg}$$

If the above is zero or negative, no overlay is needed.

Example No. 2

A pavement was measured in late summer. The average pavement temperature during measurement was 98.5°F (37°C)

The measured values are:

W_1 = 1.15 mils (.0292mm)

W_2 = 0.88 mils (.0224mm)

W_3 = 0.55 mils (.0140mm)

W_4 = 0.36 mils (.0091mm)

W_5 = 0.27 mils (.0069mm)

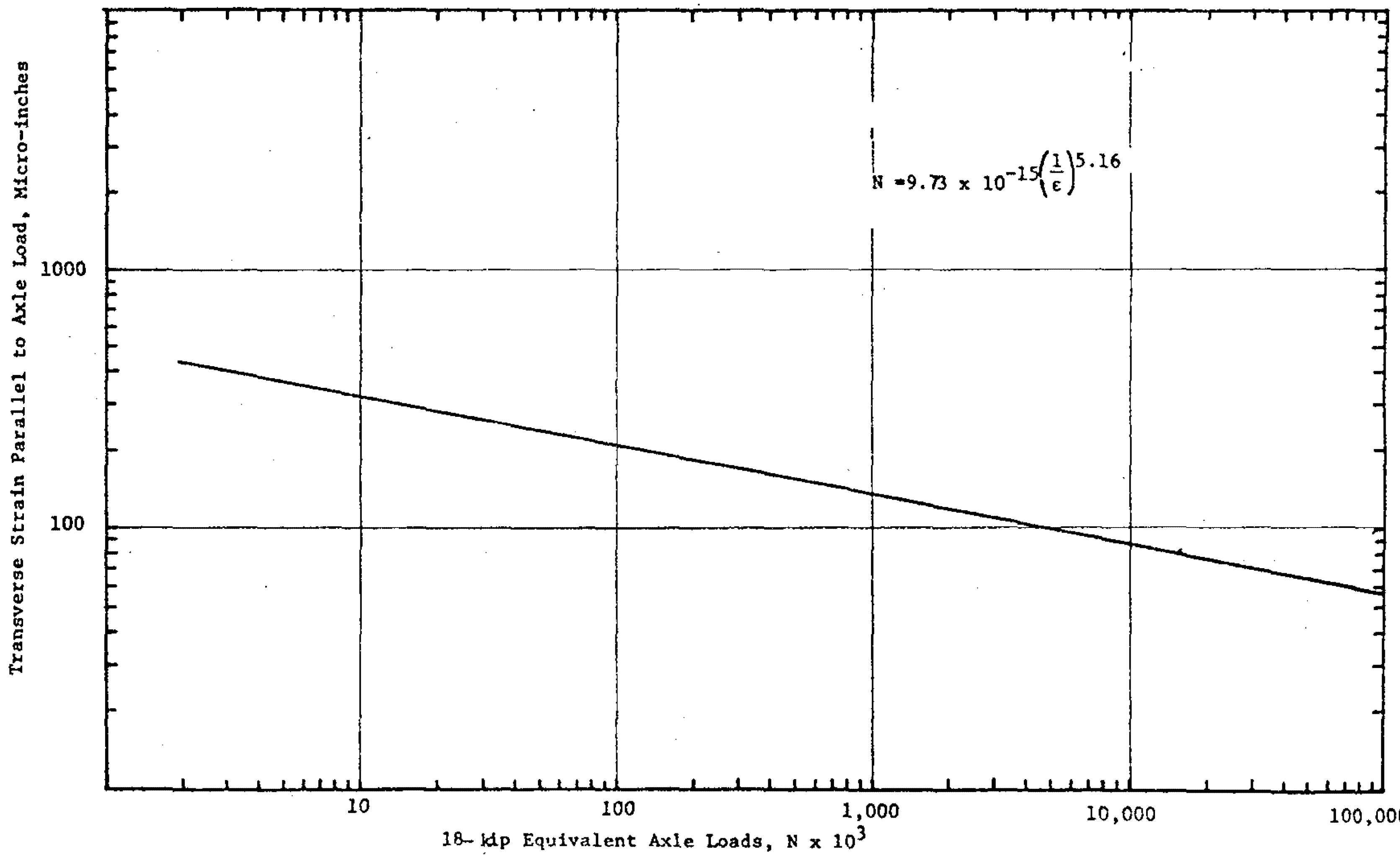


Figure 27 Fatigue Curve for 18-kip Load Applications to Time of Class 2 Cracking

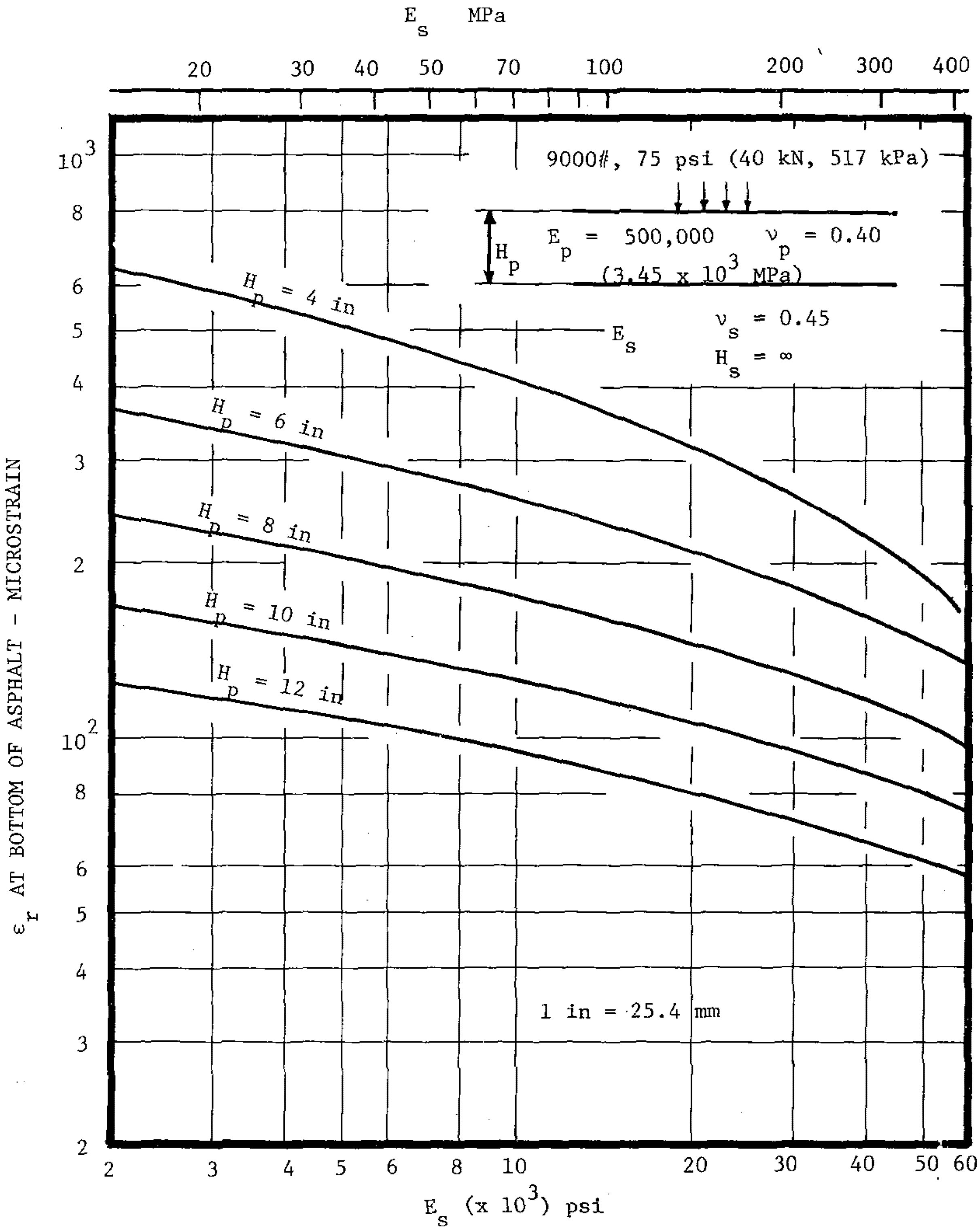


FIGURE 28. VARIATION OF ε_r WITH SUBGRADE MODULUS FOR DIFFERENT PAVEMENT THICKNESSES

The calculated spreadability is

$$Sp = 55.8\%$$

1. From Figure 10 E_p = 200,000 psi (1.38 x 10^9 Pa)
   From Figure 14 H_{eq} = 8 in. (203mm) E_s = 20,000 psi (138MPa)

2. The subgrade behaves as shown in Figure 25, Lab curve.

   a. from Figure 24 the Dynaflect deviatoric stress is 1.42 psi (9.79 MPa)
   b. plot a point at 1.42 and 20,000 on Figure 25 and draw a line thorugh this point parallel to the Lab curve - this becomes the analysis curve
   c. using Figures 25 and 26 and the procedure described above, the design subgrade modulus is obtained (after several iterations)

$$E_{sp} = 7{,}000 \text{ psi (48 MPa)}$$

3. It is desired that the road handle 4.2 million trucks over the design period. From Figure 27 the allowable tensile strain is 100 micro strain.

4. From Figure 28, for E_s of 7,000 psi (48 MPa) and ε_r of 100 micro strain the required thickness is 12 in. (305mm).

5. H_{OLY} = 12 - 8 = 4 inches (102mm)

BIBLIOGRAPHY TO APPENDIX A

1. Southgate, H.F., "An Evaluation of Temperature Distribution Within Asphalt Pavements and Its Relaionship to Pavement Deflections". Commonwealth of Kentucky, Department of Transportation, Bureau of Highways, Division of Research, April 1968.

2. Austin Research Engineers Inc., "Asphalt Concrete Overlays of Flexible Pavements Volume 2, Design Procedure." Federal Highway Administration Report No. FHWA-RD-75-76.

APPENDIX B

A GUIDE FOR SELECTION OF TYPICAL MODULI OF PAVEMENT COMPONENT LAYERS

In this appendix, the typical results of moduli of various pavement component layers such as asphaltic concrete, granular base, granular subbase, cement treated base and subgrade are presented.

The tabulated results reported herein have been gathered from review of various research reports referenced at the end of this appendix and only represent typical values for pavement moduli.

Accompanying tables (Tables 9 through 13) present typical modulus of resilience relationships for various pavement component materials. The relationships have all been determined in the laboratory by repeated load triaxial tests or by a cycylic load triaxial test.

In the first and the second columns, the pertinent properties of the material such as gradation, water content, density, etc., are presented. In column three, the number of the reference from which the data has been obtained, are listed. These references are presented at the end of the Appendix. The type of test, either repeated load triaxial test (TR) or cyclic load triaxial test (MTS) are given in column four. Column five gives the frequency (repetition rate) of the load application in counts per minute and the duration of the load, which is the length of time over which the maximum dynamic load is retained over the sample. Column six gives the number of load applications at which the modulus of resilience values have been computed. As discussed previously, modulus of resilience may be represented by:

$$M_R = K_1(\sigma_i)^{K_2}$$

where $i = 3$ or d, or

$$M_R = K_3(\theta)^{K_4},$$

depending on the material.

TABLE 34 MATERIAL CHARACTERISTICS AND MODULI DATA

GRANULAR BASE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY &DURATION	LOAD REPE-TITION	EQUATION OF STATE $f(\sigma_3 \text{or } \sigma_d)$ K_1^{+}	K_2	$f(\theta)$ K_3^{+}	K_4	REMARKS
California; well graded and angular crushed stone, 3/4 in. max.; class 2 Aggr. base.	Dry; 3% <#200	1,3, 4,5, 8&23	TR**	30 cpm 0.1 sec.	100	11000 12000	0.53	300- 4000	0.65	v*=0.38
	Dry; 5% <#200			"	"	11400 15000	0.55	3500 5000	0.63	v*=0.31
	Dry; 10% <#200			"	"	14000 15000	0.5	5000	0.57	v*=0.25
	partially sat-urated; 3% < #200		"	"	"	9000 10000	0.57	2710	0.67	v*=0.27
	partially sat-urated; 5% < #200		"	"	"	8000 9000	0.58	2300 2700	0.66	v*=0.34
	partially sat-urated: 10% < #200		"	"	"	9000 10000	0.56	2200 3000	0.66	v*=0.45
California; well graded and sub-rounded gravel; 3/4 in. max.; class 2 Aggr. base.	Dry; 3% <#200		"	"	"	10000 13000	0.53	2000 4000	0.65	v*=0.47
	Dry; 5% <#200		"	"	"	10200 11000	0.62	2800 3300	0.69	v*=0.38
	Dry; 8% <#200		"	"	"	8000 9000	0.59	"	0.70	v*=0.45

+ = psi

*v = Poisson's Ratio

**TR - triaxial repeated loading

1 in = 25.4mm 1 psi = 6.895 Kpa

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 or \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Crushed gravel base	γ_d*=138.8 W/C=4.4% compactive effort =26400 ft.lbs/cft	10	TR	120 cpm 0.1 sec	10000	5700	0.58			
Gonzales by-pass Agg. base; class 2.	γ_d=131 W/C=5.1 to 7.5%; degree of sat. ∿ 60%	3,5	"	20 cpm 0.1 sec	10000	15200	0.482			
Morro Bay base	degree of sat. ∿ 60%		"	"	"	11000	0.45			
Crushed rock base	γ_d=137.9; W/C=4.4% Compactive effort =12200 ft.lbs/cft	10	"	120 cpm 0.1 sec	"	5600	0.58			
Gravel from McHenry, Ill.	γ_d=126.7 pcf; e=0.31; dry 10% <3/8"	11 12	MTS (triaxial)	50 cpm 0.15 sec	5000			5388	0.59	
	3%<3/8"; γ_d=102.4, e=0.62		"	"	"			8228	0.53	

* γ_d = Dry unit weight in pounds per cubic foot 1 pef = 16.018 kg/m^3 1 ft. lbs/cft = kg m/m^3

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 \text{ or } \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
California; well graded and angular crushed stone, 3/4 in. max.; class 2 Aggr. base	partially saturated; 3% <#200	1,3, 4,5, 8&23	TR**	"	"	7000 10000	0.55	2000 3500	0.65	v*=0.3
	partially saturated; 3% <#200		"	"	"	"	0.59	2000 3000	0.67	v*=0.41
	partially saturated; 8% <#200		"	"	"	5000 7300	0.63	1600 1900	0.72	v*=0.46
	saturated; 3% <#200		"	"	"	9600 11000	0.54	2700 3700	0.63	v*=0.26
	saturated; 5% <#200		"	"	"	8000 10000	0.54	2400 3200	0.65	v*=0.35
	saturated; 8% <#200		"	"	"	9000 12000	0.5	3000 4000	0.6	v*=0.25
SanDiego Test Road Aggregate Base	Dry; W/C = 2.6 to 2.8%	2	"	"	"			5700	0.5	
	Field W/C = 5.93 to 6.24%		"	"	"			4300	0.5	
	Wet; W/C = 6.21 to 6.52%		"	"	"			3100	0.5	

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 \text{ or } \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Gravel from McHenry, Ill.	3%<3/8"; γ_d=107.5; e=0.54	11, 12	MTS (tri- axial)	50cpm 0.15sec	5000			10431	0.49	
	3%<3/8"; γ_d=112.1; e=0.48		"	"	"			25187	0.38	
	well graded; 20%<#4; γ_d=131.7; e=0.26		"	"	"			7781	0.60	
Crushed gravel from McHenry, Ill.	3%<3/8"; γ_d=100.8; e =0.66		"	"	"			7864	0.56	
Limestone; dolomitic from Kankakee, Ill.	10%<3/8"; γ_d=90.3; e=0.81		"	"	"			11234	0.4	
	10%< 3/8"; γ_d=103.2; e=0.59		"	"	"			5640	0.52	
	10%< 3/8"; γ_d=106.8; e=0.55		"	"	"			7296	0.54	
	3% <3/8"; γ_d=88.9 e=0.84		"	"	"			6513	0.51	

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 \text{ or } \sigma_d)$		$f(\theta)$		REMARKS
						K_1	K_2	K_3	K_4	
Limestone; dolomitic from Kankakee, Ill.	3%<3/8"; γ_d=95.9; e=0.71	11, 12	MTS (tri-axial)	50cpm 0.15sec	5000			5883	0.47	
	3%<3/8"; γ_d=99.0; e=0.66		"	"	"			8636	0.46	
	well graded; 20%<#4; γ_d=111.9; e=0.46		"	"	"			5149	0.59	
	well graded; γ_d=112.1; e=0.46		"	"	"			4733	0.61	
	CA-10(Ill.) γ_d=123.8; e=0.32		"	"	"			2598	0.65	
	CA-10(Ill.) γ_d=130.6; e=0.25		"	"	"			4186	0.6	
Granitic Gneiss; from Columbus, GA.	10% <3/8"; γ_d=89.3; e=0.87		"	"	"			34127	0.19	
	3% <3/8"; γ_d=93.0; e=0.76		"	"	"			5128	0.6	

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 or \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Granitic Gneiss; from Columbus, GA.	3%<3/8"; γ_d=97.5; e=0.71	11, 12	MTS (triaxial)	50cpm 0.15sec	5000			6819	0.53	
	3%<3/8"; γ_d=102.3; e=0.63		"	"	"			8076	0.52	
	well graded; 20%<#4; γ_d=86.3; e=0.54		"	"	"			7092	0.56	
Basalt from New Jersey	10%<3/8"; γ_d=107.5; e=0.63		"	"	"			8944	0.47	
	3%<3/8"; γ_d=95.3; e=0.82		"	"	"			4725	0.65	
	well graded; 20%<#4; γ_d=115.7; e=0.5		"	"	"			7145	0.6	
Crushed porphyritic Granite Gneiss 3%<#200	γ_d=137.4; W/C=6.5	29	TR	30 cpm 0.1	10000			3746.1	0.532	
	γ_d=130.5; W/C=							2145.8	0.703	
	γ_d=130.5							2857.5	0.632	Soaked

TABLE 34 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3$ or $\sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Crushed porphyritic Granite Gneiss - 11.25% <#200	γ_d=135.0; W/C=6.0	29	TR	30 cpm 0.1sec	10000			1976.9	0.681	
	γ_d=128.25; W/C=	"	"	"	"			3359.2	0.539	
	γ_d=128.25	"	"	"	"			2414.5	0.619	Soaked
Crushed Biotite Granite Gneiss; 3% <#200 11.25% <#200	γ_d=137.4 W/C=6.5%	"	"	"	"			1986.7	0.682	
	γ_d=135.0 W/C=6.0%	"	"	"	"			1494.8	0.718	
22% <#200	γ_d=132.9; W/C=6.1%	"	"	"	"			1491.8	0.731	
Dry Gravel		9, 14	"	"				1900	0.61	
Granular Base Colorado standard base ½" max & 8.7% <#200; std. subbase 2½" max & 7.9%<#200	W/C=2.6% W/C=6.3% W/C=8.2%	8	"	120cpm 0.2sec	10000	10618 10019 8687	0.4474 0.465 0.696			
Crushed Stone Base 3%<3/4"	(γ_d)=147pcf; W/C=4.7% 6.2%<#200	21	"	20cpm 0.1sec	10000	7300	1.01			

TABLE 35 MATERIAL CHARACTERISTICS AND MODULI DATA

GRANULAR SUBBASE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 or \sigma_d)$ K_1^+	K_2	$f(\theta)$ K_3^+	K_4	REMARKS
Morrow Bay Sub-base	Degree of sat. ∿ 60%	3,5	TR*	20cpm 0.1 sec	10000	7600	0.33			
Blend of 17% Silty sand + 83% Crushed biotite Granite Gneiss	Compacted @ γ_d=143.0 & W/C=4.6%	29	"	30cpm 0.1 sec	"			3835.5	0.534	
								3145.0	0.552	Soaked
Blend of 40% Silty fine sand + 60% no. 467 stone 75%<3/4"; 0%<#10	Compacted @ (γ_d) = 138.0 W/C=4.2%		"	"	"			2507.1	0.624	
								3825.7	0.459	Soaked
	Compacted @ (γ_d) = 138.0 W/C = 6.6%		"	"	"			1791.5	0.802	
								3902.6	0.529	Soaked
Blend of 21% sandy silt + 79% crushed biotite granite Gneiss	Compacted @ γ_d=140.2 & W/C = 6.0%		"	"	"			4982.6	0.45	
								5938.9	0.365	Soaked
Crushed Granite 83% + silty sand 17%	100% T-180;** W/C = 5.1%	8	"	33cpm 0.1 sec	"			3836	0.53	
								3145	0.55	
Sand	Dry	9, 14	"	20cpm 0.1 sec		12500	0.35	6700	0.36	

TABLE 35 MATERIAL CHARACTERISTICS AND MODULI DATA

GRANULAR BASE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3$ or $\sigma_d)$ $f(\theta)$ K_1+	K_2	K_3+	K_4	REMARKS
Sand Subbase	γ_d = 133.9 W/C = 4.52%; compactive effort 2500 ft lbs/ft^3	10	TR*	120cpm 0.1 sec	1000	6700	0.55			
Clayey Sandy Silt Blythe Test Section B	$(\sigma_d)_{max}$ = 110 OMC = 15.3%	13	"	20cpm	"	26310	0.022	30134	-0.031	

+ = psi
*TR = triaxial repeated loading
**AASHTO test specification No.
γ_d = Dry unit weight in pounds per cubic foot 1in = 25.4mm 1 psi = 6.895 kPa 1pcf = 16.018 kg/m^3
1ft.lb/cu ft = 4.88 kgm/m^3

TABLE 36 MATERIAL CHARACTERISTICS AND MODULI DATA

SUBGRADE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 \text{ or } \sigma_d)$ K_1^+	K_2	$f(\theta)$ K_3^+	K_4	REMARKS
Silty sand		18 19	TR**	35 cpm 0.1sec	10000			3126	0.37	
Silty fine sand	100% AASHTO*** T-99; W/C=13.4% 40%<#200	8	"	33 cpm 0.1 sec	10000			1856 3126	0.61 0.37	
Silty fine sand orange tan, slightly clay	Compacted @ (γ_d)=115.4 W/C = 13.0% L.L=22%; PI=6 %	29	"	30 cpm 0.1 sec	"			1855.6 3126.6	0.606 0.371	Dry Soaked
Clayey silty sand Blythe Test Section-A	γ=125; W/C = 4.4%	13	"	20 cpm 0.1sec	10000	19844	0.197	14145	0.194	Subgrade
Clayey sand	Gonzales By-pass Subgrade	3	"	20 cpm 0.1 sec	200 200 60000 60000	9000* 4800* 24500* 6600*	-0.99* 0.09 -0.8* 0.25			$0<\sigma_d<3$psi $3<\sigma_d<10$ $0<\sigma_d<3$ $3<\sigma_d<10$
F-1 type soils	$(\gamma_d)_{max}$=123.9; OMC = 7.8%		"							
	W/C = 4.5%	17				11100	0.46	6000	0.4	
	W/C = 6.0%					10900	0.42	6000	0.4	
	W/C = 8.0%					10500	0.43	5690	0.41	
	W/C = 9.7%					6100	0.54	2640	0.54	

+ = psi
* Functions of σ_d
** TR = Triaxial repeated loading
***AASHTO Test specification No.
γ_d = Dry unit weight in pounds per cubic foot

1 psi = 6.895 kPa   1 pcf = 16.018 kg/m^3

TABLE 36 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 or \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Silty Clay Subgrade	AASHO Test Rd.; γ_d=119; W/C= 15.3%; Deg. of sat.= 95%	4 8	TR	20cpm 0.25sec	10000	62000*	-1.12*			Kneading Compaction $0< \sigma_d<19$
						1500*	0.21*			$19<\sigma_d<35$
						65000*	-1.0*			Static compaction $0<\sigma_d=32$
						10*	1.11*			$32<\sigma_d<40$
		9	"	"	200	25000*	-0.77*			$0<\sigma_d<18$
						3120*	0.7*			$18<\sigma_d<38$
Silty Clay	PI=25.5	8	"	30cpm 0.1sec	10000	3200*	5.2*			
Highly Plastic Clay	PI=36.5	9	"	"	"	4150*	1.0*			
Silty Clay	A-6; E-5; CL; L.L=28.31% PI=13.7%	22	"	120cpm 0.125sec	"	66000*	-0.38*			
						49000*	-0.38*			
	A-7; E-7; Cl; L.L=41.0% PI=28.3%	"	"	"		24000*	-0.11*			
	A-4; E-6; CL to ML; L.L=26.5% PI=7.6%	"	"	"		46000*	-0.3*			
	A-4; CL L.L=28.8%; PI=10.1%	"	"	"		64000	-0.18			

* Functions of σ_d

TABLE 36 cont. MATERIAL CHARACTERISTICS AND MODULI DATA

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 \text{ or } \sigma_d)$ K_1	K_2	$f(\theta)$ K_3	K_4	REMARKS
Lean Clay	E-8; CL Soils	18	TR	30 cpm 0.1 sec	10000	26800*	-0.495*			$0<\sigma_d<12.5$
Heavy Clay	E-11; CH Soils	18 19	"	"		25000*	-0.77*			$0<\sigma_d<12.5$

TABLE 37 MATERIAL CHARACTERISTICS AND MODULI DATA

CEMENTED TREATED BASE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	EQUATION OF STATE $f(\sigma_3 or \sigma_d)$ K_1^+	K_2	$f(\theta)$ K_3^+	K_4	REMARKS
Silty sand stabilized with 60% cement	γ_d = 124.0; W/C = 10%	29	TR*	30 cpm 0.1sec	10000			2.51×10^6	-0.444	
40% Silty sand + 60% #467 stone + 2.73% cement	γ_d = 138.0; W/C = 7.5	29	"	"	100			0.30×10^6	0.399	

+ = psi
*TR = Triaxial repeated loading
γ_d = Dry unit weight in pounds per cubic foot

1 psi = 6.895 kPa
1 pcf = 16.018 kg/m^3

TABLE 38 MATERIAL CHARACTERISTICS AND MODULI DATA

ASPHALTIC CONCRETE

MATERIAL DESCRIPTION	MATERIAL CHARACTERISTICS	REF. NO.	TYPE OF TEST	FREQUENCY & DURATION	LOAD REPETITION	MR (PSI)	REMARKS
California type B, ½ in. max. med. aggr., 85 to 100 pen. asphalt		8	TR*	30 cpm 0.1 sec	100	300000 70000	70°F 90°F
Georgia standard A, 1½" max. aggr., 85 to 100 pen. asphalt		"	"	20 cpm 0.1 sec	10000	220000 100000	72°F 89°F
California Type B; 3/8 inch max. med aggr., 85 to 100 pen asphalt		"	"	30 cpm 0.1 sec	100	2500000 1500000 50000	40°F 55°F 100°F
Asphalt Institute mix IVb, ½ in. max. aggr.; 60 to 70 & 85-100 pen. asphalts		"	CT**	1 to 16 Hz	250 to 300	600000 to 2000000 150000 to 750000 50000 to 150000 1100000 to 3000000 350000 to 1300000 90000 to 450000	40°F; 1Hz 70°F; 1Hz 100°F; 1Hz 40°F; 16Hz 70°F; 16Hz 100°F; 16Hz

*TR = Triaxial repeated loading

**CT = Cyclic Load Triaxial

1 in = 25.4mm 1 psi = 6.895 kPa °F = 1.8°C + 32

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APPENDIX C
ANNOTATED BIBLIOGRAPHY

The following annotated bibliography presents brief descriptions of various flexible pavement overlay design methods which have recently been developed or are currently in use.

ANNOTATED BIBLIOGRAPHY ON THE DESIGN OF FLEXIBLE PAVEMENT OVERLAYS.

1. Kinchen, R.W. and Temple, W.H., "Asphaltic Concrete Overlays of Rigid and Flexible Pavements", Interim Report No. 1, Research Report No. 109, Lousisiana Department of Transportation, 1977.

   Required overlay thickness is based upon allowable deflections as a function of number of 18 kip (80kN) load applications and an empirically derived relationship between overlay thickness and pavement reduction in deflection. A nomograph has been prepared which denotes the required overlay for any desired life and measured representative deflection. The design method is based on the concept of "critical" deflection representing the maximum pavement deflection allowable before failure occurs in the form of fatigue cracks. The empirical relationship between cumulative 18 kip (80kN) axle loadings and allowable deflection was derived from field studies conducted in Louisiana. Rutting is not considered in the failure criteria because "asphalt overlays have been observed to crack before developing severe rutting.

   Existing pavement deflection is evaluated through use of a Dynaflect. Testing is conducted in wheel paths which exhibit the greatest amount of distress at .05 mile (80m) intervals. Air and pavement surface temperature along with visual pavement conditions are also recorded when deflection measurements are obtained. Representative deflection is calculated as the arithmetic mean of the highest and second highest measurements as an approximation of the 95th percentile value. The calculated representative deflection is normalized to 60^{o}F (15.5^{o}C) using a temperature correction factor based upon the time of measurement, surface temperature and 5 day mean air temepratur e. Deflections are not adjusted for seasonal effects.

2. Classen, A.I.M., and Ditmarsch, R., "Pavement Evaluation and Overlay Design - The Shell Method". 4th International Conference, Structural Design of Asphalt Pavements, 1977.

The procedure models the pavement as a 3-layer system and utilizes the computer program BISAR to analyze the pavement models response to load. BISAR is a multi-layer linear elastic semi-infinite half space program capable of handling multiple circular loaded areas. The program can also analyze models with unbonded pavement layers and horizontal (shear) as well as vertical loads. The overlay thickness procedure is based on fatigue life of the asphaltic layer. Fatigue life is established from corrected laboratory curves which relate strain at the bottom of the asphaltic layer to cycles to failure. Required overlay thickness is the difference between the design asphalt thickness (as established by the fatigue equation and the BISAR analysis) and the effective thickness of the existing asphalt layer.

Characterization of the model pavement structure in terms of layer thicknesses and elastic moduli is derived from pavement deflection measurements. Asphalt modulus and base thickness, E_1 and h_2, respectively, are considered as known parameters. E_1 is estimated from temperature-moduli curves and h_2 from construction reports or small diameter core drilling. A fixed mathematical relationship is assumed to exist between base modulus (E_2) and subgrade modulus (E_3). The two unknown parameters, effective asphalt thickness (h_1) and subgrade modulus (E_3), are solved by use of the deflection data and the BISAR program. Two independent deflection measurements are utilized to determine the two unknown pavement parameters.

Deflection measurements are made using a Falling Weight Deflectometer (FWD). The normal testing interval is 50 meters (164 ft.) with measurements made both in one of the wheel tracks and between wheel tracks. Duplicate measurements are made at each location with average values used. Deflection at both the center of the load (d_1) and 600 mm (23.6 in) average from the load (d_2) are measured. Hourly air and pavement surface temperature measurements are also made. Utilization of the deflection data to evaluate effective asphalt layer thickness and subgrade modulus is generally based upon the 85 percentile value of maximum deflection and the 15 percentile for Q. (Note $Q=d_2/d_1$)

The design life of the existing pavement is determined by using between wheel track deflection measurements to calculate $h_1$and E_3 or input to the fatigue strain analysis. Remaining life is the difference between design life and actual loadings experienced. The overlay calculation involves the same procedure, only wheel track deflection measurements are used to calculate h_1 and E_3. The overlay is assumed to have the same modulus as the existing asphalt layer with E_1 based upon the anticipated mean annual air temperature. The design method does not consider seasonal variation of subgrade conditions nor is the stress dependency of base and subgrade materials considered.

3. Peterson, D.E., et al., "Asphalt Overlays and Pavement Rehabilitation Evaluating Structural Adequacy for Flexible Pavement Overlays", Final Report No. 8-996, Utah Department of Transportation.

   The design method is based upon an empirically derived relationship between maximum Dynaflect deflection and terminal number 18 kip (80kN) axle loadings. When the measured deflection exceeds the required deflection associated with the future number of 18 kip (80kN) axle loadings, an overlay is needed to reduce deflection to a tolerable level. A nomograph relating required deflection, measured deflection and required structural number (SN) of overlay has been developed. A weighted structural number is computed from regional factors which consider differences in precipitation and freezing index within the state. Overlay thickness is computed by dividing the weighted structural number by the structural coefficient for the overlay material.

   Dynaflect deflection measurements are usually obtained from mid-June to November in order to minimize seasonal effects. Measurements are obtained in the outer wheel path at intervals not exceeding .1 mile (160 m). Representative deflection is the mean value plus twice the standard deviation and is corrected to a standard temperature of 60°F (15.5°C) by use of a temperature adjustment factor. The temperature adjustment factor is based on relationships between pavement surface temperature, air temperature history and pavement temperature at any depth derived by Kentucky (Havens and Southgate) and verified for Utah conditions.

   Utah also uses a computer program to compare measured values of maximum deflection, SCI and BCI with required values for these parameters (depending on number of axle loadings) to qualitatively evaluate pavement and subgrade conditions. The evaluation is based upon field observations of poor and good pavements.

4. Lister, N.W., and Kennedy, C.K., "A System For The Prediction of Pavement Life and Design of Pavement Strengthening", 4th International Conference on Structural Design of Asphalt Pavements, 1977.

Method is based upon empirical relationships between "early life" and "critical" deflections established from measurements on a wide variety of roads in the United Kingdom. "Critical" deflection is defined as the preferred time for extending the structural life of the pavement by overlaying. Critical condition is based upon rutting, rather than fatigue cracking, and occurs at a serviceability index of approximately 3.2 PSI. Authors claim that fatigue cracking is rarely seen in the United Kingdom except on very thin pavements or after severe rutting has occurred. Nomographs have been prepared relating critical deflections to cumulative number of standard axle loadings. Remaining life of the existing pavement is defined as the difference between the expected life (as defined by the critical deflection, standard axle relationship) and the traffic experienced to date. The thickness of required overlay for extension of life (beyond the current remaining life) is based upon empirical relationships between overlay thickness and reduction in deflection. The design procedure can consider three different base materials; bituminous, unbound and cement-treated.

Existing pavement deflection is evaluated using either a Benkelman Beam or a Lacroix Deflectograph, with readings obtained a 4-meter (13 ft.) intervals. Testing is usually done in the spring with methods having been developed for normalizing other deflections to standard conditions (20°C (68°F) in spring). Roads are divided into sections of 100m (328 ft.) lengths with the design deflection for each section equal to the 95 percentile value. The project design deflection is based upon the confidence level chosen for the design, (for example, the 90 percentile value of the section deflections). Latest development in the design method is a computer program capable of treating the variability of deflection in a consistent manner, so as to eliminate the risks of localized early failure or overdesign in the strengthened pavement.

5. Bhajandas, A.C., et al. "A Practical Approach to Flexible Pavement Evaluation and Rehabilitation", 4th International Conference on Structural Design of Asphalt Pavements, 1977.

Design procedure is based upon allowable deflections as a function of number of 18 kip (80kN) axle loadings and base type. Permissible deflections on crushed stone bases were developed from AASHO Road Test data, while tests on model pavements were used to develop the bituminous base relationships. Deflection 18 kip (80kN) loading relationships associated with different terminal serviceability values are developed for each base type. The relationship between overlay thickness and reduction in deflection was empirically derived. A nomograph has been established to determine overlay thickness based upon existing and desired maximum deflection.

Deflection measurements are made using a Road Rater. Corrective nomographs for surface temperature and season have been formulated to adjust deflection measurements to standard conditions (63°F (17°C) in the spring). The 95th percentile deflection value, adjusted for temperature and season, is used to represent the existing pavement. Interval length for deflection measurements was not stated.

6. Bonnot, J., et a., "Design of Asphalt Overlays for Pavements", 4th International Conference on Structural Design of Asphalt Pavements, 1977.

A description of the systematic overlay design procedure used by French Department of Highways is presented. Method is based upon modeling both the existing and overlaid pavement by a multilayer elastic model (ALIZE). Default or assumed modulus values are used in the model, except subgrade modulus which is based upon measured pavement deflection. Program is capable of considering 1 to 6 pavement layers. Overlay thickness is based upon fatigue of the asphaltic layer (limiting strain) and maximum acceptable vertical strain at the top of the subgrade. A catalogue of overlay structures has been prepared for 36 classes of old flexible pavement types (classified by deflection and layer thickness combinations), with different overlay thicknesses identified for each pavement class as a function of future traffic loadings. Method considers the stockastic nature of fatigue distress and adopts different acceptable distress risks as a function of traffic.

Existing pavement deflection is measured with the Lacroix Deflectograph. Guidelines have been prepared for collecting deflection data. The 95th percentile values is used for design purposes with a project segmented into lengths of similar deflection characteristics. Seasonal and temperature adjustment are not presented.

7. Rufford, P.G., "A Pavement Analysis and Structural Design Procedure Based on Deflection", 4th International Conference on Structural Design of Asphalt Pavements 1977.

   Design procedure is based upon allowable deflections as a function of number of standard axle loadings for various base types. Design Benkelman Beam deflection is determined from the performance of the existing pavement in relation to empirical recommended maximum deflections derived by the Transport and Road Research Laboratory. Field studies have indicated that TRRL deflections can be used to determine pavement life based upon fatique cracking in New South Wales, Australia. Benkelman Beam deflections including spreadability are used to assess the stiffness of the subgrade and the effective thickness of the existing pavement, using two layer elastic theory. The required pavement thickness to reduce the measured deflection to the design value is also calculated using the two-layer elastic theory. Overlay thickness is equal to the required pavement thickness minus the effective pavement thickness. All thicknesses are computed assuming the top layer is a base material with a specified modulus value. A stiffness equivalency ratio is used to convert base thicknesses to other material (such as asphalt concrete) thicknesses.

8. Bushey, R.W., et al, "Structural Overlays for Pavement Rehabilitation - Interim Report", California Department of Transportation, Report No. CA-DOT-TL-3128-3-74-12, July 1974.

   Method is based upon maximum deflection as related to total traffic and existing structural section. Nomographs have been developed which define the highest level of pavement deflection to which a particular pavement thickness could be subjected during its design life without developing fatique cracking. Tolerable deflections are based upon field observations of pavement performance and fatique criteria considering maximum tensile strain at the bottom of the asphaltic layer. A nomograph is presented defining the required overlay thickness necessary to reduce the existing deflection to the tolerable level as a function of traffic. The thickness nomograph is based upon field investigations to define the relationship between asphalt concrete overlay thickness and percentage of deflection reduction.

Deflection measurements are normally made using a Traveling Deflectometer, although nomographs are available for using measurements made by Dynaflect, Benkelman Beam or Dehlen Curvature Meter. Measurements are obtained on 800 to 1000 foot (240 to 300 m) test sections which are felt to be representative of 1 centerline mile (1.6 Km) of roadway. Benkelman Beam or curvature readings are obtained at 25 foot (7.6 m) intervals, with Dynaflect and Deflectometer measurements, at .01 mile (16 m) and 20 foot (6 m) intervals, respectively. Measurements are normally conducted in the spring. The 80th percentile deflection is used in the design method.

9. Asphalt Institute, "Asphalt Overlays and Pavement Rehabilitation", Manual Series No. 17, 1969.

Two methods are presented for design with one based upon deflection measurements and the other a component analyses of the existing pavement.

The deflection method is based upon reducing deflection to a tolerable level. The method usese a 2-layer elastic analysis to determine the amount of overlay required to reduce deflection. The existing pavement is represented as a one layer system whose modulus is calculated from the measured design deflection. A default value of elastic modulus is used for the top layer (the overlay). The criteria for tolerable deflection is based upon a single, empirically derived relationship between number of 18 kip (80kN) loadings and deflection. The relationship is assumed to be practical for pavements having asphalt surface and granular bases.

The manual presents a testing procedure to obtain Benkelman Beam rebound deflections. Measurements are made in the outer wheel tracks with about 20 measurements made per mile (12 per kilometer). Adjustment for temperature is presented. The manual recommends that readings also be adjusted for seasonal conditions and presents a procedure for defining the "critical" period adjustment factors.

The component analysis method is based upon characterizing the existing "equivalent" thickness of the existing pavement. Conversion factors are presented, which consider both material type and quality for use in determining the equivalent thickness. Effective thickness, in terms of asphalt concrete, is determined by multiplying each of the existing pavement layers by the appropriate conversion factors and then summarize. The required overlay thickness is equal to the difference between the

required "full depth" thickness and the effective thickness of the exisitng pavement structure. The conventional Asphalt Institute method for flexible pavement design is used to determine the required "full depth" asphalt thickness.

10. AASHO, "Interim Guide for Design of Pavement Structures", American Associating State Highway Officials, 1972.

The Oklahoma method is based upon the premise of providing a PSI of 2.0 after 20 years. Required overlay, thickness is a function of Benkelman Beal deflection, only. A nomograph is presented to determine the amount of asphaltic concrete overlay necessary to reduce deflection to the limiting value which will permit the pavement to perform satisfactorily for 20 years.

Average Benkelman Beam deflections can either be obtained from direct measurements, with adjustments derived for seasonal variations, or from a method based upon condition surveys. The method for estimating Benkelman Beam deflection is based upon empiricaly relations between depreciation of a pavement condition rating at an age of 10 years, and measured deflection.

11. Brown, J.L. and Orellara, H.E., "Utilizing Deflection Measurements to Upgrade Pavement Structures", Texas Highway Department Research Report 101-1F, Dec. 1970.

The overlay design subsystem is part of the computerized Texas Highway Department Flexible Pavement Design System (FPS). The procedure is based on the deflection-performance equation developed by Scrivner which defines expected PSI decrease as a function of surface curvature index, traffic and temperature. From Dynaflect measurements and Scrivner's deflection equation, SCI of the existing pavement is used to determine the SCI of the overlayed pavement. The overlayed pavement SCI is then used in the performance equation to predict future serviceability for a given overlay thickness.

The subsystem evaluates several AC overlay strategies and identifies the optimum design. Uncertainty in the predicted performance, as a result of variability in construction materials, is considered so that 95% or 99% confidence level designs may be selected.

12. Vaswani, N.K., "A Method for Evaluating the Structural Performance of Subgrades and for the Overlaying Flexible Pavements", Virginia Highway Research Council, Interim Report No. 3, February 1971.

Burmeister's two-layer elastic theory is used to represent existing pavement structures, with the AASHO design equation used to design new pavements. Dynaflect spreadability and Benkelman Beam maximum deflection (converted from Dynaflect deflection) are used to evaluate the subgrade modulus and effective pavement thickness. The elastic modulus of the top layer is assumed equal to that of asphalt concrete. Overlay thickness is equal to the required full depth asphalt concrete thickness (based upon soil support value and design traffic) minus the effective thickness of the existing pavement. The procedure is utilized for resurfacing flexible pavements on primary, interstate and arterial roads in Virginia.

13. Kher, R. and Phang, W.A., 4th International Conference on Structural Design of Asphalt Pavements, 1977.

An overlay design method is included with the Ontario Pavement Analysis of Costs (OPAC) design and management system. OPAC is a computerized system that compares the performance and cost of alternative flexible pavement designs. Pavement life is based upon the predicted deterioration in Riding Comfort Index (RCI) as a function of expected traffic loading and annual cyclic environmental changes. RCI deterioration is predicted thru use of 1) linear elasticity for calculating pavement response under load; 2) default or estimated modulus and layer equivalency factors for surface, base and subbase materials; and 3) empirical relationships derived from the AASHO and Brampton Road Tests. Subgrade surface deflection is calculated using a simplified procedure originally developed by Odemark and is used to predict RCI decrease. Separate functions are used for the prediction of load and environmental related RCI decrease.

The performance relationship for an overlay involves reducing the layer equivalency factors to reflect in-service deterioration of in-situ pavement materials, calculation of the equivalent pavement thickness including both existing layers and the overlay, calculation of the subgrade defelction for the overlayed structure, and prediction of RCI deterioration over the design period. The optimum overlay thickness or strategy is that which maintains the performance above an assumed terminal RCI level for minimal cost during the design period. Evaluation of pavement conditions from field tests are not required design input into the OPAC model. The reduced layer equivalency factors were empirically derived from observations of in-service pavements.

*U.S. GOVERNMENT PRINTING OFFICE : 1981 0-731-543/2507

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

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This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.