TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWATX79-177-13	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle

A RIGID PAVEMENT OVERLAY DESIGN PROCEDURE FOR TEXAS SHDPT

5. Report Date

May 1978

6. Performing Organization Code

7. Author(s)

Otto Schnitter, W. R. Hudson, and B. F. McCullough

8. Performing Organization Report No.

Research Report No. 177-13

9. Performing Organization Name and Address

Center for Highway Research
The University of Texas at Austin
Austin, TX 78712

10. Work Unit No.

11. Contract or Grant No.

Research Study 3-8-75-177

13. Type of Report and Period Covered

Interim

12. Sponsoring Agency Name and Address

Texas State Department of Highways and Public Transportation, Transportation Planning Division
P.O. Box 5051
Austin, TX 78763

14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration. Research Study Title: "Development and Implementation of the Design, Construction, and Rehabilitation of Rigid Pavements."

16. Abstract

The Texas State Department of Highways and Public Transportation (SDHPT) rigid pavement overlay design procedure was developed by evaluating, improving, modifying, and simplifying a recently developed Federal Highway Administration overlay design method.

This overlay design procedure involves fatigue cracking and reflection cracking subsystems. Linear elastic layered theory is the basic model for computing stresses and strains in the pavement system for fatigue computations. The condition and remaining life of the existing pavement are considered in the fatigue cracking analysis, and thickness designs for practically all types of asphaltic concrete and portland cement concrete overlays on rigid pavements can be obtained using this computerized method. The reflection cracking analysis, intended for use with asphaltic concrete overlays, involves the computation of strains in the overlay due to horizontal, thermal, and vertical load-associated movements in the overlay. The final overlay thickness is selected to meet both the fatigue cracking and reflection cracking criteria.

The design procedure uses four computer programs for pavement evaluation, overlay thickness design and reflection cracking analysis. A detailed User's Manual intended for use by Texas SDHPT is included in the report.

It is recommended that this design procedure be implemented for trial use as soon as possible. This design method is a useful research tool as well as a practical design procedure.

17. Key Words

pavement evaluation, pavement design, overlay, rigid overlays, flexible overlays, deflection analysis, condition survey, fatigue cracking, reflection cracking

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	414	

Form DOT F 1700.7 (8-69)

A RIGID PAVEMENT OVERLAY DESIGN PROCEDURE FOR TEXAS SDHPT

by

Otto Schnitter
W. R. Hudson
B. F. McCullough

Research Report Number 177-13

Development and Implementation of the Design, Construction and Rehabilitation of Rigid Pavements

Research Project 3-8-75-177

conducted for

Texas State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

May 1978

PREFACE

This report presents the development of a rigid pavement overlay design procedure for the Texas State Department of Highways and Public Transportation (SDHPT). A recently developed Federal Highway Administration (FHWA) overlay design procedure was the basis for the Texas procedure. By means of evaluation and modification the FHWA procedure has been adapted for Texas. This procedure, based on the most up-to-date theories and concepts, can be used to design both asphaltic concrete and portland cement concrete overlays on rigid pavements.

In order to make this report as useful and functional as possible, it is presented here in three parts. Part I gives a brief summary of the FHWA method and outlines the revised Texas SDHPT procedure. Part II deals with the evaluation of the FHWA method and the details of the modifications made for Texas. Part III is a detailed step-by-step User's Manual for the Texas SDHPT.

This is the thirteenth in a series of reports which describe work done on Project 3-8-75-177, "Development and Implementation of the Design, Construction, and Rehabilitation of Rigid Pavements."

The cooperation of the staff of the Center for Highway Research of The University of Texas at Austin as well as the assistance of the personnel of the Texas State Department of Highways and Public Transportation is greatly appreciated.

O. Schnitter

W. R. Hudson

B. F. McCullough

May 1978

LIST OF REPORTS

Report No. 177-1, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement," by Felipe R. Vallejo, B. Frank McCullough, and W. Ronald Hudson, describes the development of a computerized system capable of analysis and design of a concrete pavement slab for drying shrinkage and temperature drop. August 1975.

Report No. 177-2, "A Sensitivity Analysis of Continuously Reinforced Concrete Pavement Model CRCP-1 for Highways," by Chypin Chiang, B. Frank McCullough, and W. Ronald Hudson, describes the overall importance of this model, the relative importance of the input variables of the model and recommendations for efficient use of the computer program. August 1975.

Report No. 177-3, "A Study of the Performance of the Mays Ride Meter," by Yi Chin Hu, Hugh J. Williamson, B. Frank McCullough, and W. Ronald Hudson, discusses the accuracy of measurements made by the Mays Ride Meter and their relationship to roughness measurements made with the Surface Dynamics Profilometer. January 1977.

Report No. 177-4, "Laboratory Study of the Effect of Non-Uniform Foundation Support on CRC Pavements," by Enrique Jimenez, W. Ronald Hudson, and B. Frank McCullough, describes the laboratory tests of CRC slab models with voids beneath them. Deflection, crack width, load transfer, spalling, and cracking are considered. Also used is the SLAB 49 computer program that models the CRC laboratory slab as a theoretical approach. The physical laboratory results and the theoretical solutions are compared and analyzed and the accuracy is determined.

Report No. 177-5, "A Comparison of Two Inertial Reference Profilometers Used to Evaluate Airfield and Highway Pavements," by Chris Edward Doepke, B. Frank McCullough, and W. Ronald Hudson, describes a United States Air Force owned profilometer developed for measuring airfield runway roughness and compares it with the Surface Dynamics Profilometer using plotted profiles and mean roughness amplitude data from each profilometer.

Report No. 177-6, "Sixteenth Year Progress Report on Experimental Continuously Reinforced Concrete Pavement in Walker County," by Thomas P. Chesney and B. Frank McCullough, presents a summary of data collection and analysis over a 16 year period. During that period, numerous findings resulted in changes in specifications and design standards. These data will be valuable for shaping guidelines for future construction. April 1976.

Report No. 177-7, "Continuously Reinforced Concrete Pavement: Structural Performance and Design/Construction Variables," by Pieter J. Strauss, B. Frank McCullough, and W. Ronald Hudson, describes a detailed analysis of design, construction, and environmental variables that may have an effect on the structural performance of a CRCP is presented. May 1977.

Report No. 177-8, "Continuously Reinforced Concrete Pavement: Prediction of Distress Quantities," by John P. Machado, B. Frank McCullough, and Hugh J. Williamson, presents a general analysis of environmental, design, construction and historic pavement behavior conditions and their effects on future performance.

Report No. 177-9, "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavements," by James Ma and B. Frank McCullough, describes the modification of a computerized system capable of analysis of a continuously reinforced concrete pavement based on drying shrinkage and temperature drop. August 1977.

Report No. 177-10, "Development of Photographic Techniques for Performance Condition Surveys," by Pieter Strauss, James Long, and B. Frank McCullough, discusses the development of a technique for surveying heavily trafficked highways without interrupting the flow of traffic. May 1977.

Report No. 177-11, "A Sensitivity Analysis of Rigid Pavement-Overlay Design Procedure," by B. C. Nayak, W. Ronald Hudson, and B. Frank McCullough, gives a sensitivity analysis of input variables of Federal Highway Administration computer-based overlay design procedure RPOD1. June 1977.

Report No. 177-12, "A Study of CRCP Performance: New Construction versus Overlay," by James I. Daniel, W. Ronald Hudson, and B. Frank McCullough, the documentation of the performance of several Continuously Reinforced Concrete Pavements (CRCP) in Texas.

Report No. 177-13, "A Rigid Pavement Overlay Design Procedure for Texas SDHPT," by Otto Schnitter, W. Ronald Hudson, and B. Frank McCullough, discusses the development of the design procedure, and provides a detailed User's Manual for Texas use.

ABSTRACT

The Texas State Department of Highways and Public Transportation (SDHPT) rigid pavement overlay design procedure was developed by evaluating, improving, modifying and simplifying a recently developed Federal Highway Administration overlay design method.

This overlay design procedure involves fatigue cracking and reflection cracking subsystems. Linear elastic layered theory is the basic model for computing stresses and strains in the pavement system for fatigue computations. The condition and remaining life of the existing pavement are considered in the fatigue cracking analysis, and thickness designs for practically all types of asphaltic concrete and portland cement concrete overlays on rigid pavements can be obtained using this computerized method. The reflection cracking analysis, intended for use with asphaltic concrete overlays, involves the computation of strains in the overlay due to horizontal, thermal, and vertical load-associated movements in the overlay. The final overlay thickness is selected to meet both the fatigue cracking and reflection cracking criteria.

The design procedure uses four computer programs for pavement evaluation, overlay thickness design and reflection cracking analysis. A detailed User's Manual intended for use by Texas SDHPT is included in the report.

It is recommended that this design procedure be implemented for trial use as soon as possible. This design method is a useful research tool as well as a practical design procedure.

KEY WORDS: pavement evaluation, pavement design, overlay, rigid overlays, flexible overlays, asphaltic concrete overlays, portland cement concrete overlays, deflection analysis, condition survey, fatigue cracking, reflection cracking

SUMMARY

A great portion of the pavements on the Interstate Highway System are approaching the end of their design lives, and it is certain that reconstruction and rehabilitation of existing pavements will become increasingly important in the future.

Until recently a good rational method to design overlays on rigid pavements did not exist. A recently developed FHWA overlay design procedure for rigid pavement fulfills this need in general. This report describes the development of a rigid pavement overlay design method for Texas SDHPT by adapting, modifying, improving and simplifying the FHWA method. This report also includes a detailed User's Manual for Texas SDHPT.

The Texas SDHPT procedure is based on sound theoretical principles and takes the structural capacity of the existing pavement into account. Fatigue cracking and reflection cracking subsystems are involved in this method.

The fatigue cracking subsystem computes the required overlay thickness, both for portland cement concrete or asphaltic concrete overlays on rigid pavements. The condition and remaining life of the existing pavement, as well as voids underneath the existing pavement are considered in this analysis.

The reflection cracking analysis is conducted for asphaltic concrete overlays on cracked or jointed rigid pavements. Thermally induced horizontal tensile strains, as well as load associated vertical shear strains in the overlay, are considered in this analysis. The final overlay thickness is selected to satisfy both the fatigue cracking and reflection cracking criteria.

This research provides Texas SDHPT with a procedure to design practically all types of overlays on rigid pavements in a rational way.

IMPLEMENTATION STATEMENT

A rigid pavement overlay design procedure has been provided for the Texas State Department of Highways and Public Transportation, including a detailed step-by-step User's Manual. This overlay design procedure will be useful as

(1) a research tool,

(2) a practical design method for designing both rigid and flexible overlays on rigid pavements, and

(3) an overlay design model to be incorporated in the rigid pavement management system (RPS).

TABLE OF CONTENTS

PART I

THE TEXAS SDHPT RIGID PAVEMENT OVERLAY DESIGN PROCEDURE

CHAPTER I-1. INTRODUCTION

BACKGROUND

Because many of the pavements in the Interstate System are approaching the end of their design lives, it can be expected that rehabilitation, and specifically overlays, of existing pavements to improve their structural load carrying capacities will increase in importance in years to come.

In recognition of this fact the Federal Highway Administration has recently completed a research effort with the following goals (Ref 1):

(1) to develop overlay thickness design procedures for the rehabilitation of all common pavement types and

(2) to develop design procedures for eliminating or reducing the reflection cracking of pavement overlays.

At the present time, the Texas State Department of Highways and Public Transportation is conducting a research project (3-8-75-177) with the goal of developing and implementing design, construction, and rehabilitation methods for rigid pavements. The increased importance of pavement rehabilitation makes an estimate of the extent of the needed future rehabilitation and the time required therefore, as well as the development of repair techniques, mandatory. This research project makes maximum use of previous research, experience, and existing theories, and incorporates rational techniques.

This report outlines work done to modify and adapt the FHWA design procedure (Ref 1) for flexible and rigid overlays on rigid pavements for the Texas State Department of Highways and Public Transportation. In the process, the procedure has been evaluated; some modifications, simplifications, and improvements have been made; and a detailed user's guide has been prepared for Texas State Department of Highways and Public Transportation use.

CURRENT OVERLAY DESIGN PROCEDURES

Generally current overlay design procedures are not considered adequate. They are either empirical in nature or have not been implemented for general use.

In 1973 McComb and Labra (Ref 2) reviewed several overlay design methods. They point out that at that time rigid pavement overlay design procedures did not adequately consider the structural value of the existing pavement, did not take remaining life into consideration, and were not based on fatigue criteria.

The Texas State Department of Highways and Public Transportation (SDHPT) recognizes this fact in its Design Manual (Ref 3), as follows:

> The design of an overlay is unique to each particular job. . . . A good method for designing overlays with confidence does not exist. More experience with actual performance is needed.

Several methods for the design of overlays on rigid pavements are in use in Texas; two of them are discussed here briefly.

RPS Models For Overlay Design

In 1971 a rigid pavement management system program (RPS) was developed for the Texas SDHPT (Ref 4) using the best models available at that time and with the intention to update the program as technology improved. The model for overlay design for rigid overlays over rigid pavements in RPS is basically that developed empirically by the Corps of Engineers for airfield pavements (Ref 4), as follows:

$$D = \sqrt[1.4]{C_D h_e^{1.4} + h_o^{1.4}}$$

where

D = equivalent concrete thickness,

h_e = existing concrete thickness,

h_o = overlay concrete thickness,

C_D = a coefficient determined by the condition of the existing pavement. C_D generally varies between 0.35 and 1.0 and is determined by engineering judgement.

The model for designing asphalt concrete overlays of rigid pavements in the RPS system has been developed using linear elastic layer theory. The thickness of the composite pavement, consisting of the existing concrete thickness and the asphalt concrete overlay thickness, is replaced by an equivalent concrete thickness, which is evaluated in analysis by an extended AASHO model for the design of rigid pavements (Ref 4).

Design Procedure Used For Asphalt Concrete Overlay on CRCP on Walker County Project

McCullough (Ref 5) reported the overlay design procedure used on a portion of Interstate 45, approximately 11 miles in length in Walker County, Texas. The overlay was asphaltic concrete on an existing CRC pavement. Input data for this overlay design procedure included: (1) surface deflections, (2) material characteristics, (3) traffice data (axle load groupings with associated repetitions), (4) environmental data, (5) construction variables, and (6) observation of distress.

The pavement was divided into design sections using deflection measurements, observed distress manifestations, and engineering judgement. Statistical methods have been used to ascertain a significant difference between adjacent sections. Laboratory determined material properties of pavement layers, layer thicknesses, deflection information, and stochastic principles were used in determining the subgrade resilient modulus for each design section.

The remaining life of the existing pavement was taken into account by subtracting the estimated cumulative damage from unity. Stresses, strains, and deflections were computed using linear elastic layer theory. The future life of the overlay was predicted using fatigue concepts, taking the remaining life of the existing pavement into consideration. The average stiffnesses for asphalt concrete for each month were used as input with the fatigue program. Data from estimations of remaining life and predictions of future life were used to estimate the required overlay thickness.

This procedure was a rational method using the most up-to-date pavement design technology but, at that stage, was not implemented in such a way that it could be used by the average pavement design engineer.

NEED FOR A NEW OVERLAY DESIGN PROCEDURE

From the preceding discussion of overlay design methods for rigid pavements in general, and more particularly in the state of Texas, it can be seen that, in view of the anticipated increase in expenditure on pavement rehabilitation, it is of the utmost importance to have reliable design criteria for overlays of rigid, as well as flexible, pavements, based on sound theoretical principles.

This need is being fulfilled, in general, by a recently developed overlay design procedure for the Federal Highway Administration (Refs 1 and 6). The overlay design procedure here suggested for use by the Texas State Department of Highways and Public Transportation is basically similar to the Federal Highway Administration method with certain modifications and improvements to suit the needs of the Texas State Department of Highways and Public Transportation.

It is believed that this design procedure, based on sound theoretical principles, field observations, and AASHO Road Tests experience (Ref 1) is a good design procedure for overlays over rigid pavements in the state of Texas.

As with all new design methods, verification is needed. This process, however, takes time and might call for some further refinements and modifications of the method in the future.

OBJECTIVES

The overlay design procedure developed for the Federal Highway Administration involves many variables and is capable of analyzing practically all combinations of overlays (flexible and rigid) over rigid pavements.

The purpose of this study was to implement the design procedure for the Texas State Department of Highways and Public Transportation by

(1) evaluating it to determine whether or not any modifications were needed;

(2) evaluating a sensitivity analysis reported by Nayak et al. (Ref 7) on the RPOD1 computer program, which predicts pavement thicknesses based on fatigue criteria and implementing the findings in the Texas method;

(3) conducting a limited sensitivity analysis on the reflection cracking program RFLCR1 to determine which are the more important variables and using this information in developing the Texas method;

(4) improving and modifying the design procedure;

(5) adapting the procedure to meet the needs of the Texas State Department of Highways and Public Transportation; and

(6) developing a user's manual for the use of the Texas State Department of Highways and Public Transportation.

SCOPE

This report discusses the development and use of a rigid pavement overlay design procedure for the Texas SDHPT. The recently developed FHWA method for design of overlays on rigid pavements was considered to be the basis forthe development of the Texas procedure. A brief summary of the FHWA method and the modifications made to that method in adapting it for Texas use are given in Part I of this report. Part I also outlines the Texas SDHPT overlay design procedure and includes an illustrative example problem.

Part II contains an evaluation of the FHWA method and indicates the necessary modifications, simplifications and improvements to adapt the procedure for Texas SDHPT use.

A detailed User's Manual, for the use of Texas SDHPT, is given in Part III. This Manual includes procedures for evaluation of existing pavements, materials testing procedures, and operating procedures for the computer programs involved in this design method.

CHAPTER I-2. FHWA PROCEDURE

The recently developed FHWA procedure for design of overlays on rigid pavements (Ref 1) is the basis for the development of the Texas SDHPT procedure. In this chapter, the design concepts used in the FHWA procedure are briefly summarized, and modifications to the procedure required to adapt it for Texas use are outlined. More detailed information on the FHWA procedure can be obtained in the work reported by Treybig, et al. (Refs 1 and 6).

DESIGN CONCEPTS

The primary design criteria in the FHWA method are the prevention of fatigue cracking and the prevention or minimizing of reflection cracking.

The procedure is automated (Ref 6) and four different computer programs are used, as can be seen in Table I-2.1. The RPOD1 program is used to determine the required overlay thickness to prevent fatigue cracking and RFLCR1 is then used to check for reflection cracking. It is possible to eliminate the use of any of these programs if the function of that particular program is not required for a specific design problem or if the designer chooses to do that particular operation in another way. For instance, the reflection cracking program, RFLCR1, may be omitted if it is believed that reflection cracking will not be a problem.

A flow chart of this pavement rehabilitation procedure can be seen in Fig I-2.1. There are three basic steps in this procedure:

(1) evaluation of the existing pavement,
(2) determination of design inputs, and
(3) overlay thickness analysis.

Evaluation Of The Existing Pavement

Evaluation of the existing pavement is done by means of a deflection survey and a condition survey. The deflection survey is used to divide

TABLE I-2.1. COMPUTER PROGRAMS USED IN THE FHWA DESIGN PROCEDURE

PROGRAM	FUNCTION
PLOT2:	Deflection Profile Plots profiles from measured deflections
TVAL2:	Statistical Analysis of Design Sections (1) Determines statistically whether selected design sections are significantly different (2) Determines means and standard deviations of deflection data (3) Determines design deflections
RPOD1:	Fatigue Cracking Analysis (1) Characterizes subgrade material using design deflection and laboratory data (2) Does remaining life analysis using Miner's linear damage hypothesis (3) Determines overlay thickness for specified design life, using fatigue principles
RFLCR1:	Reflection Cracking Analysis (1) Computes horizontal, thermally induced, tensile strains in AC overlay (2) Computes vertical, load associated, shear strains in AC overlay due to differential deflection at discontinuities in existing pavement

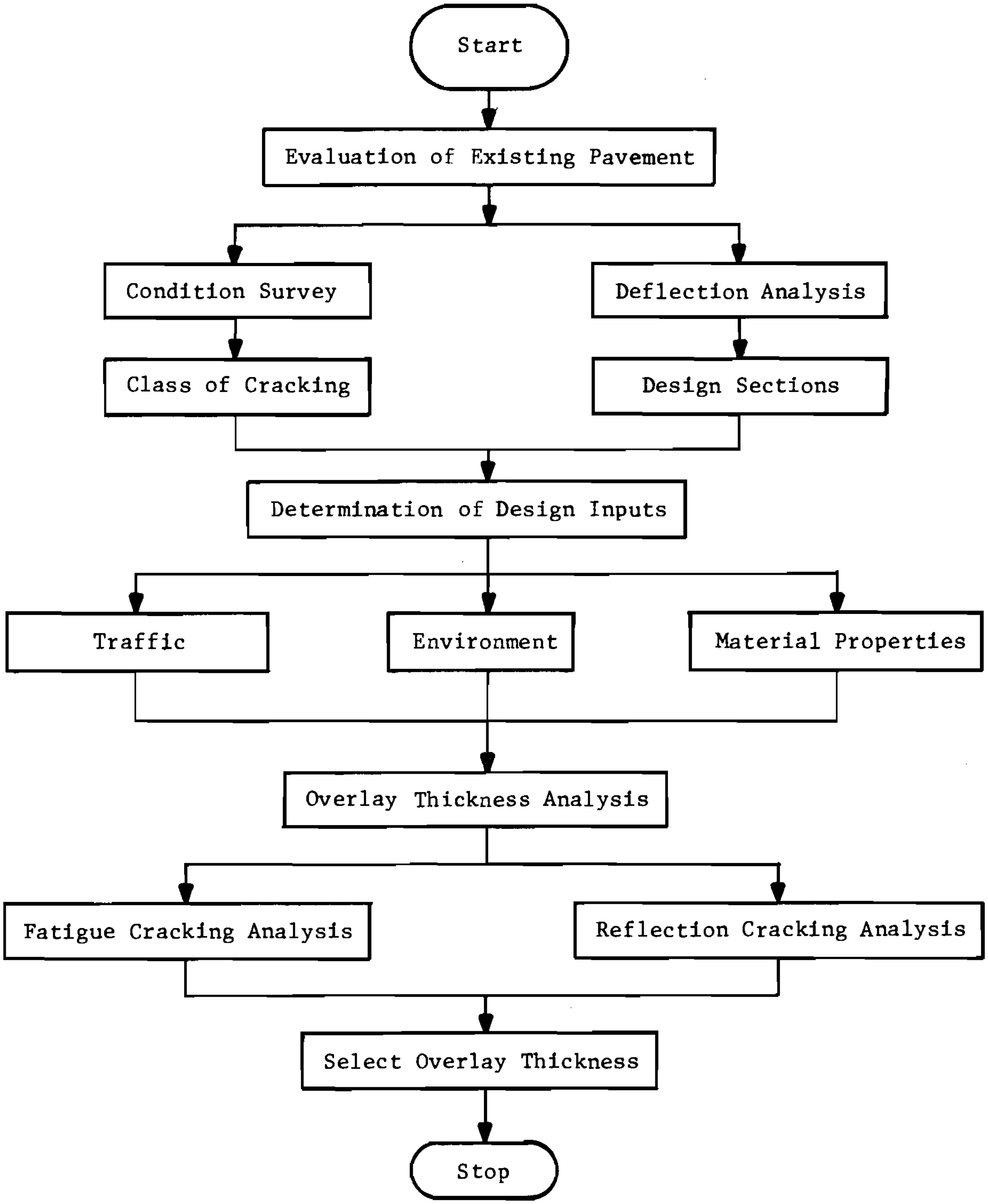


Fig I-2.1. Flow chart of pavement rehabilitation procedure.

the roadway under consideration into design sections that will behave differently from each other under load and to select design deflections for each section. Statistical methods are applied in selection of design sections as well as the determination of a design deflection. Condition survey information is used to classify the pavements into three categories: pavements with a potential of having remaining life, pavements so severely cracked that they would not be considered to have remaining life, and pavements that will be mechanically broken up before overlay.

If the reflection cracking analysis is applicable to a particular design, additional condition survey information is needed, such as differential vertical movement at cracks or joints and the amount of joint movement with change in temperature.

Design Inputs

Determination of design inputs includes both the past and projected future traffic [in terms of 18-kip (80-kN) equivalent single axle loads], environmental considerations, material properties, and dimensions of layers. Laboratory testing is required to determine elastic properties of the various pavement layers. Deflection information, as well as laboratory determination of resilient modulus at different deviator stress levels, is used in characterizing the subgrade material. For the reflection cracking analysis, additional input data, such as the creep modulus of asphaltic concrete, material thermal coefficients, and temperature information, are required.

OVERLAY THICKNESS ANALYSIS

The overlay thickness analysis considers the criteria of fatigue cracking and reflection cracking, as indicated in Fig I-2.1. The reflection cracking analysis is only required for those conditions where reflection cracking is expected to be a problem. In general, the RFLCR1 program is intended for use with asphaltic concrete overlays.

Fatigue Cracking Analysis

In the fatigue cracking analysis, linear elastic layered theory is used to characterize the subgrade material and to compute stresses, strains, and deflections. The remaining life of the existing pavement is taken into account using Miner's linear damage hypothesis.

Governing stresses to be used in the fatigue life computations are assumed to be horizontal tensile stresses due to the applied wheel loads. The position of the governing stress is assumed to be at the bottom of the overlay for pavements without remaining life and at the bottom of the existing pavement for pavements with remaining life.

Stresses computed by the linear elastic layer program are taken to be interior stresses, and stress factors have been derived by means of the discrete element theory program, SLAB49, as well as by Westergaard and Pickett theory (Ref 1). Stresses predicted by ELSYM5, are increased by these factors to give the maximum stress at the critical point for a given combination of pavement and overlay type. This critical stress location is at the corner of the slab for jointed pavements and at the edge for continuous.

Void factors have also been determined using slab theory and are used in this program to account for increased stresses due to voids under pavement slabs (Ref 1).

The computer program, RPOD1, is used for the fatigue cracking analysis and can handle both asphaltic and portland cement concrete overlays on various types of portland cement concrete existing pavements. The output of this program is the required overlay thickness for a specified design life.

Reflection Cracking Analysis

The reflection cracking analysis is primarily intended for asphaltic concrete overlays on rigid pavements (Ref 6) although other overlay types can be analyzed by reviewing the procedure. The RFLCR1 computer program provides a rational procedure for evaluating an overlay's susceptability to reflection cracking. It computes the following at joints or cracks in the existing pavement:

(1) the horizontal tensile strain in the overlay due to thermal movements and

(2) the vertical, load associated, shear strain in the overlay.

The procedure suggests that these computed strain values be compared to allowable maximum values. The program provides for the possible use of bondbreakers, intermediate layers, or reinforcement in the overlay, should these maximum criteria be violated.

MODIFICATIONS TO ADAPT THE FHWA PROCEDURE FOR TEXAS SDHPT USE

The FHWA method has been evaluated in the process of adapting it for use by the Texas SHDPT, and several modifications have been made where deemed necessary. Basically, the procedure is considered to be an excellent one and only a few changes were required to the fatigue cracking analysis program. Part II of this report gives a detailed description of these modifications.

The RPOD2 computer program is a modified version of RPOD1 and includes the following modifications:

(1) The model has been modified to include the design of asphaltic concrete overlays on pavements without remaining life. This type of design was not implemented in RPOD1 because of a problem in modeling this situation with layered theory. In RPOD2 a semi-infinite halfspace, resulting in the same deflection under the design load as the existing pavement, has been used.

(2) RPOD2 allows for the input of concrete flexural strength values for both the existing pavement and the overlay. In RPOD1, a single flexural strength value would have to be specified for both existing and new concrete.

(3) Under certain conditions, it could be more economical and realistic to consider an existing pavement with a low percentage of remaining life not to have remaining life. RPOD2 considers both possibilities for selection of the more economical thickness.

(4) Limiting elastic modulus values have been set for subbases of pavements with class 3 and 4 cracking and mechanically broken up pavements.

(5) In an effort to reduce the number of inputs required by the program, the Dynaflect load was made the default deflection load since it is widely used in Texas for deflection measurements.

(6) Overlay thicknesses on pavements without remaining life were found to be less dependent on the stress sensitivity of subgrade modulus. Therefore, an alternative way of specifying laboratory-determined resilient modulus versus deviator stress data has been provided in RPOD2.

A limited sensitivity analysis indicated that the RFLCR1 program gives reasonable results, therefore no modifications were required to this program.

The input guides have been modified for Texas use, and a step-by-step User's Manual for this procedure has been prepared. Part III of this report contains the User's Manual, which also includes recommended procedures for materials characterization.

CHAPTER I-3. TEXAS SDHPT PROCEDURE FOR OVERLAYS ON RIGID PAVEMENTS

The Texas SDHPT procedure for overlays on rigid pavements, as suggested in this report, is similar to the FHWA procedure, with certain modifications as mentioned in Chapter I-2. Treybig, et al. (Refs 1 and 6) give an in-depth discussion of the development of the FHWA procedure, and modifications to the procedure are discussed in Part II of this report. Here, only a concise discussion of the Texas procedure will be given.

The procedure is outlined in the flow chart in Fig I-2.1. It involves an evaluation of the existing pavement, determination of design inputs, and overlay thickness analysis. For the overlay thickness analysis, the primary design criteria are those of fatigue cracking and reflection cracking. Four computer programs are being used in this procedure as indicated in Table I-3.1. The RPOD2 program listed on this table is the revised version of RPOD1 used in the FHWA method. The other programs are similar to those listed in Table I-2.1.

This is an overlay thickness design procedure for the rehabilitation of all common types of rigid pavements. Figure I-3.1 is a flow diagram indicating the various overlay-existing pavement combinations that can be handled by the RPOD2 program, which does the fatigue cracking analysis in this procedure. From this figure, it can be seen that the condition and remaining life of, and voids underneath the existing pavement are taken into consideration in this analysis.

Another very important computer program is the program RFLCR1 which performs the reflection cracking analysis. Strains in the overlay caused by horizontal temperature movements at a joint or crack in the existing pavement as well as strains caused by vertical load associated movements at the crack or joint are computed by this program. Figure I-3.2 indicates that 20 different analyses can be performed depending on the type of existing pavement, condition of existing pavement, whether or not a stress

TABLE I-3.1. COMPUTER PROGRAMS USED IN THE TEXAS SDHPT OVERLAY DESIGN PROCEDURE

PROGRAM	FUNCTION
PLOT2:	Deflection Profile Plots profiles from measured deflections
TVAL2:	Statistical Analysis of Design Sections (1) Determines statistically whether selected design sections are significantly different (2) Determines means and standard deviations of deflection data (3) Determines design deflections
RPOD2:	Fatigue Cracking Analysis (1) Characterizes subgrade material using design deflection and laboratory data (2) Does remaining life analysis using Miner's linear damage hypothesis (3) Determines overlay thickness for specified design life, using fatigue principles
RFLCR1:	Reflection Cracking Analysis (1) Computes horizontal, thermally induced, tensile strains in AC overlay (2) Computes vertical, load associated, shear strains in AC overlay due to differential deflection at discontinuities in existing pavement

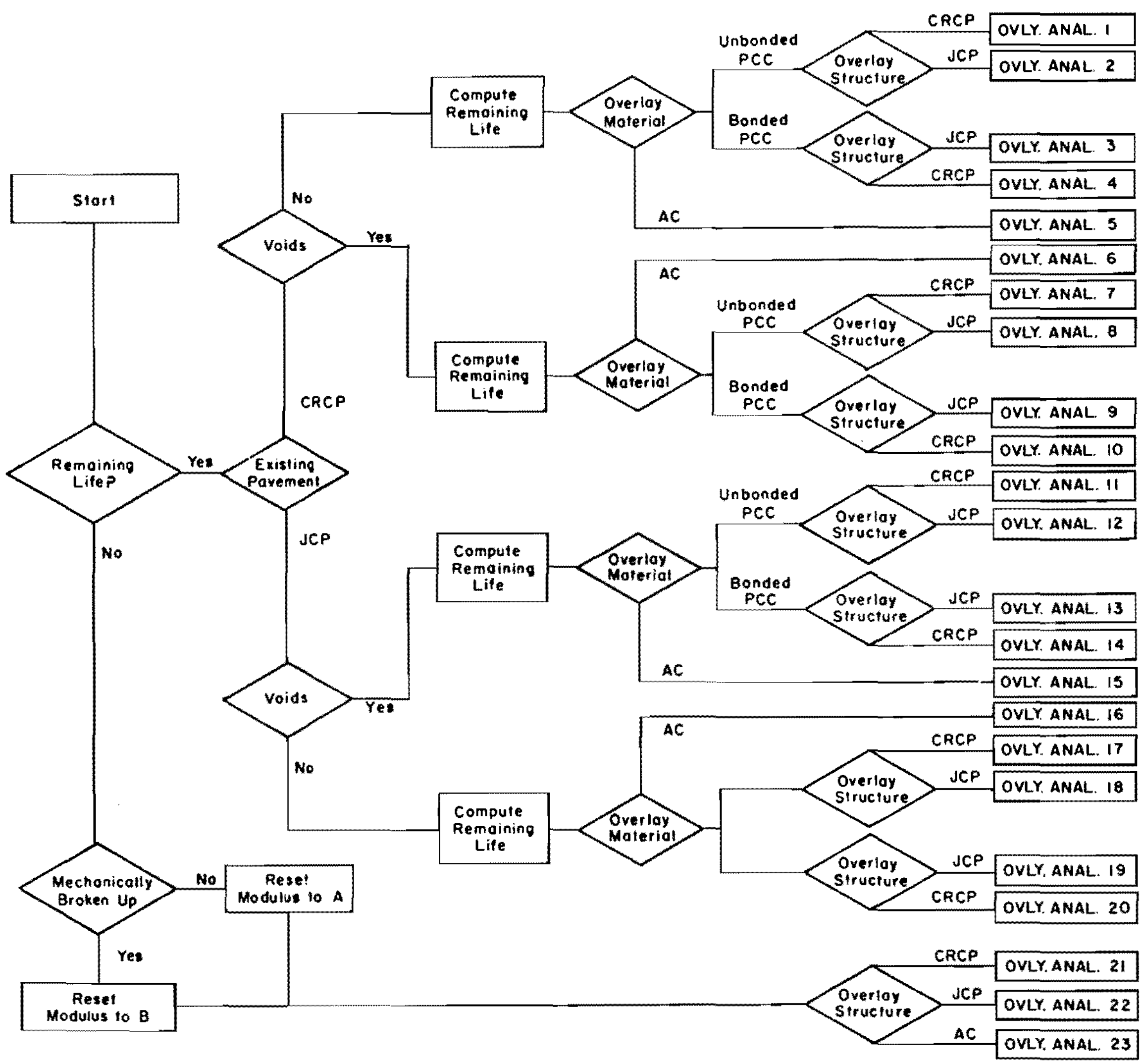


Fig I-3.1. Analysis system for determining overlay thickness of existing rigid pavements for fatigue cracking criteria.

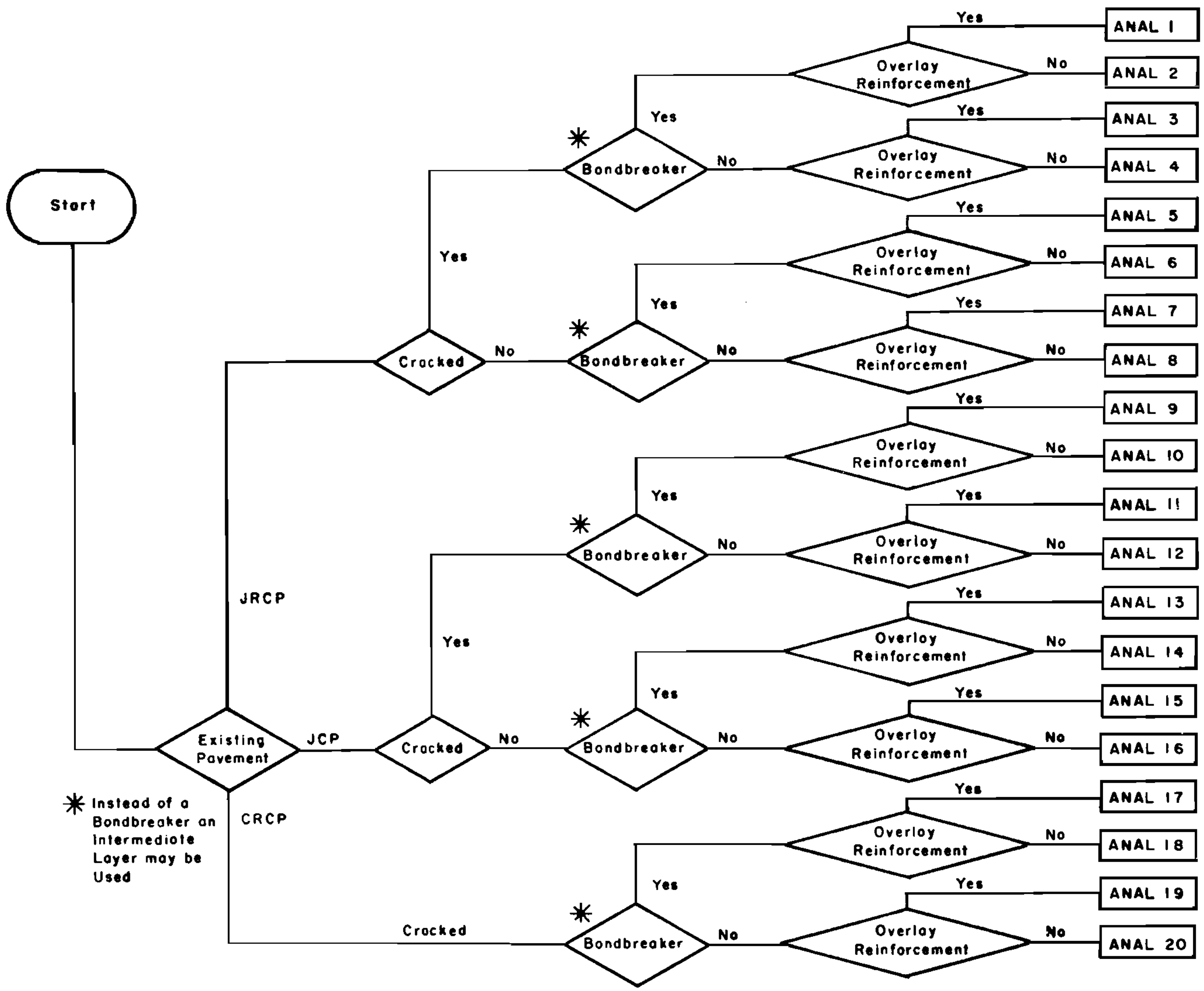


Fig I-3.2. Analysis system for prevention or minimizing reflection cracking.

relieving course is to be used, and whether or not reinforcement is to be used in the overlay. Instead of a bondbreaker, an intermediate layer can be used. This method is primarily intended for asphaltic concrete overlays on rigid pavements (Ref 6) although other overlay types can be analyzed by reviewing the procedure (Ref 1).

EVALUATION OF THE EXISTING PAVEMENT

An evaluation of the existing pavement is made by a condition survey and deflection measurements along the entire length of the project.

Condition Survey

The main purpose of the condition survey, in which the amount and type of cracking present on the existing pavement are determined, is for classification purposes. Using the AASHO Road Test classifications for cracking (Ref 8), existing pavement is classified in the following categories:

(1) uncracked or class 1 and 2 cracking,
(2) class 3 and 4 cracking, and
(3) mechanically broken up.

Each of these categories is treated differently in the fatigue cracking analysis. For the reflection cracking analysis, it is also necessary to classify the pavement as cracked or not cracked, and information regarding percentage of load transfer and temperature movement at cracks or joints is required.

Deflection Analysis

Deflection measurements are taken at regular intervals along the entire length of a project in order to distinguish among sections that behave differently from each other under load. Any reliable deflection measuring device can be used as long as the deflection load magnitude, contact pressure, and load configuration are known.

The designer uses the PLOT2 program to plot the deflection data and then divides the project into separate sections visually. An example deflection profile can be seen in Fig I-3.3; the design sections are

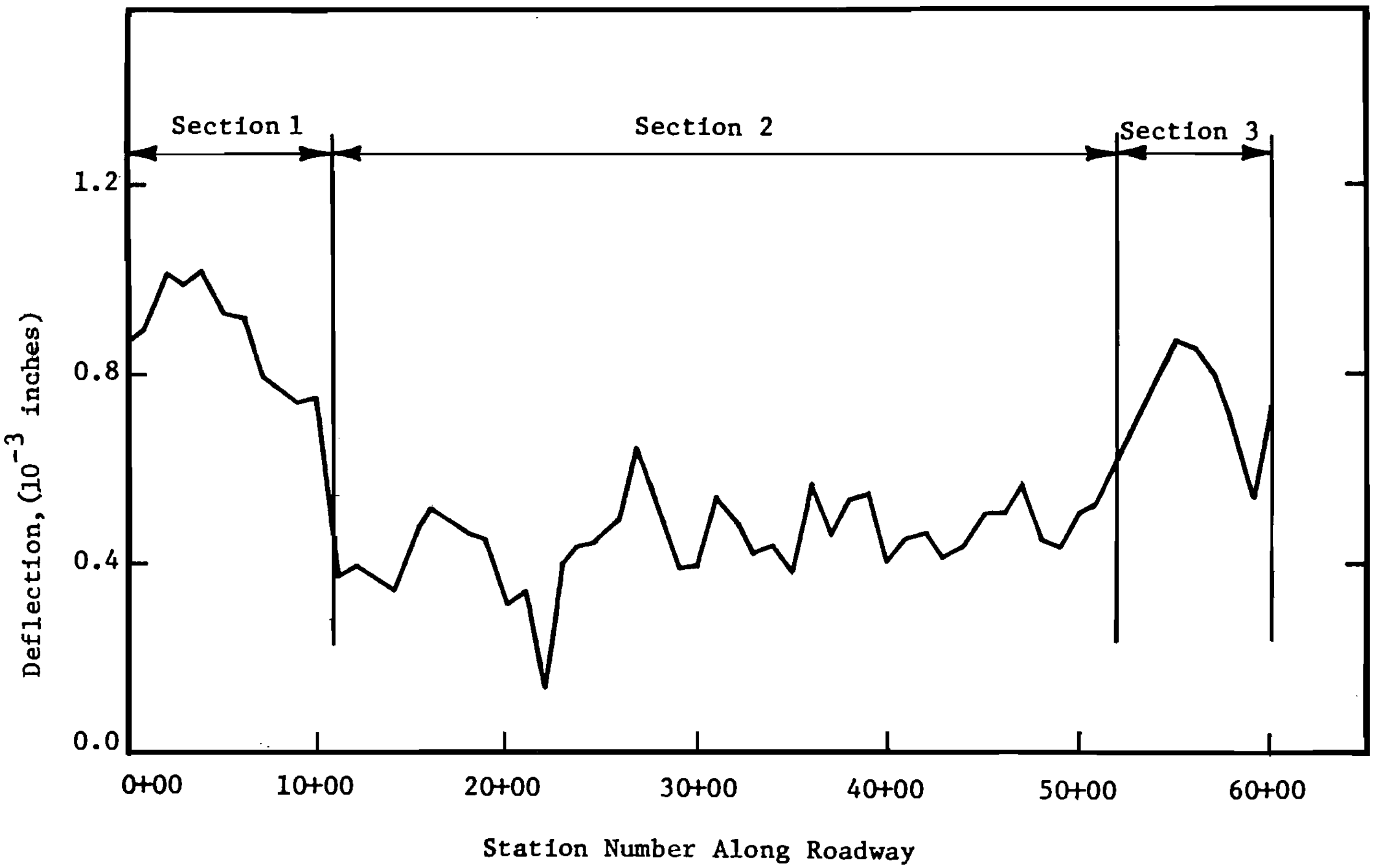


Fig I-3.3. Example deflection profile.

marked on the profile. Next, the TVAL2 program is used to determine by means of the student "t" test whether or not a significant difference exists between two adjacent sections. If so, the sections are treated as two different design sections. If no significant difference is found, the two sections are combined into one design section and are then checked against the next section. This procedure establishes the design sections, each of which becomes a separate design problem.

DETERMINATION OF DESIGN INPUTS

Design inputs for this procedure are basically traffic information, material properties, and dimensions of the different structural layers. Environmental effects are taken into account when materials are characterized or deflection measurements are taken. The reflection cracking analysis is dependent on temperature information.

Traffic

The design load fixed in the RPOD1 program is the 18-kip (80-kN) single axle load. Mixed traffic is to be converted to 18-kip (80-kN) equivalent single axle loads, and use of the AASHTO equivalency factors (Ref 9) is suggested. The design traffic input is the total number of 18-kip (80-kN) equivalent single axle loads expected in the design lane during the design period of the overlay.

In a similar way it is necessary to estimate the total traffic that used the existing pavement prior to overlay in order to determine remaining life.

Material Properties

Basically, the material properties required for this procedure are linear elastic properties (modulus and Poisson's ratio) of all the pavement layers. These properties are determined by means of laboratory tests on specimens taken out of the pavement, and, in addition, characterization of the subgrade material is done by means of deflection measurements. For granular subbase materials, reconstructed samples at field density and moisture contents may be used.

The resilient modulus of the subgrade is determined by means of resilient modulus testing in the laboratory and deflection measurements. Figure I-3.4 is a flow chart outlining the process of determining subgrade resilient modulus.

A design deflection is calculated for a selected confidence level developed for the deflection data taken with the measuring device used:

$$w_{\alpha} = \bar{w} + z\, S_{dw} \tag{I-3.1}$$

where

w_{α} = design deflection

$\bar{w}$ = mean deflection, inches;

S_{dw} = standard deviation of deflection;

z = distance from mean to selected significance level on a normal distribution curve.

The computer program TVAL2 computes design deflection for each design section.

If the deflection load is equal to the design load, the subgrade resilient modulus can be determined directly using design deflection and layered theory.

In the case where the design deflection is determined with a different load than the design load, the procedure indicated in Figs I-3.5 and I-3.6 is to be used to determine the subgrade resilient modulus. This procedure for determining subgrade resilient modulus is performed by the computer.

A relationship between resilient modulus and deviator stress for the subgrade material is determined through resilient modulus testing (Fig I-3.5a). By means of a layered program, ELSYM5, relationships are determined for surface deflection, deviator stress and subgrade modulus, as outlined in Fig I-3.5b. This is done for the deflection load on the

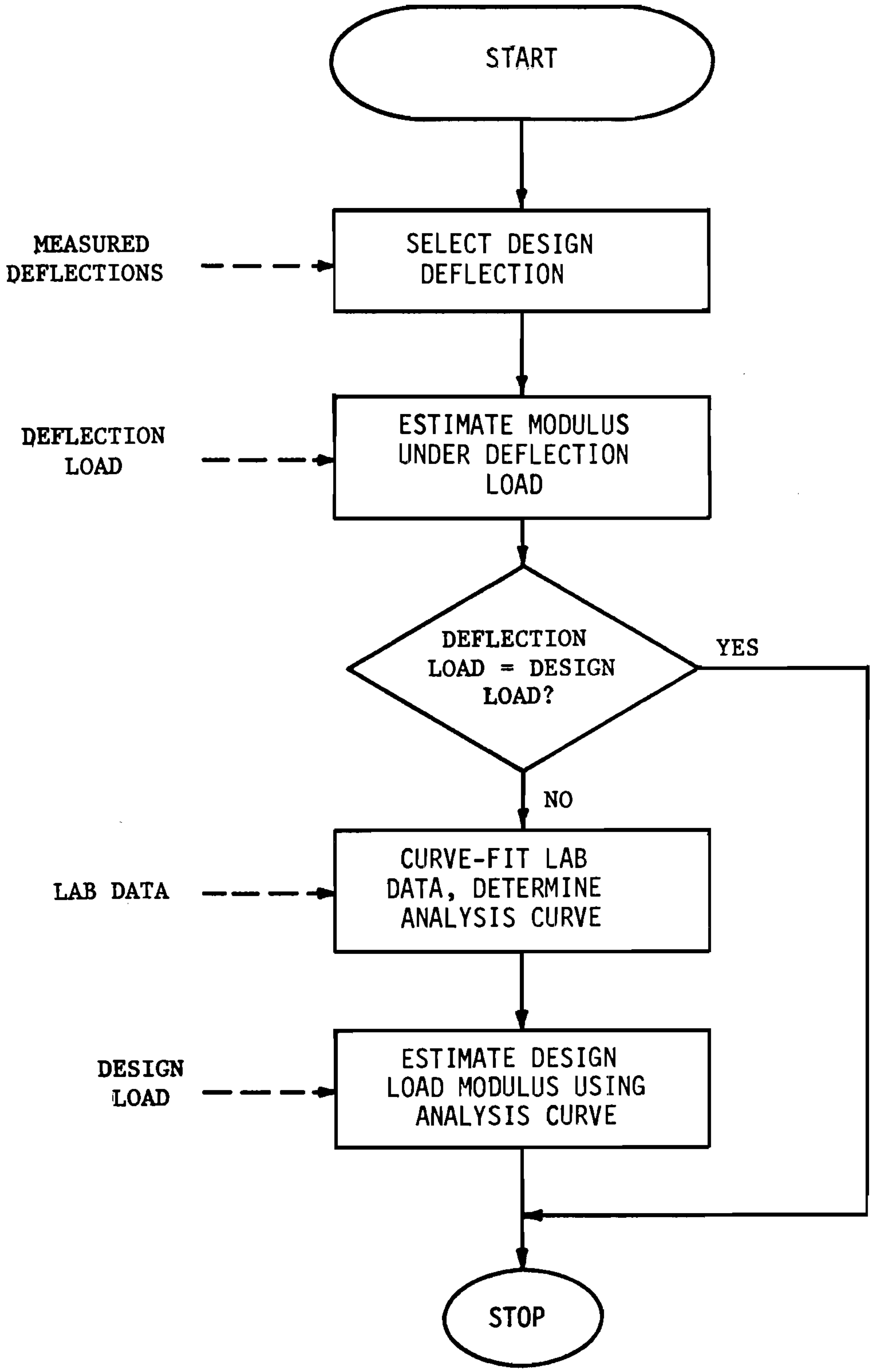
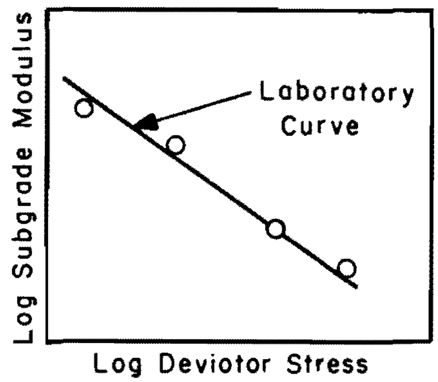
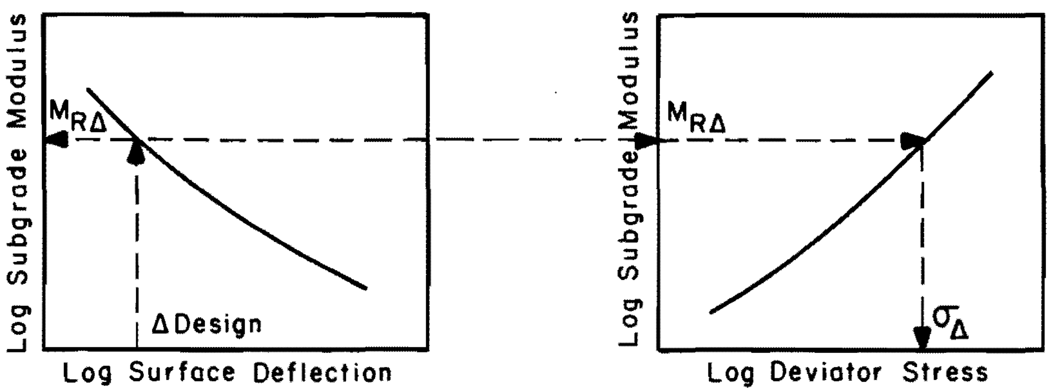


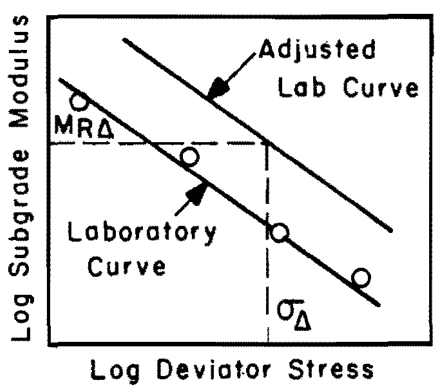
Fig I-3.4. Flow chart for determining subgrade resilient modulus.



a. Relationship between subgrade modulus and deviator stress



b. Calculated relationships between subgrade modulus and surface deflection and deviator stress under deflection load.



c. Relationship of subgrade modulus and deviator stress in determining subgrade modulus.

Fig I-3.5. Determination of subgrade modulus - deviator stress relationship using deflection data.

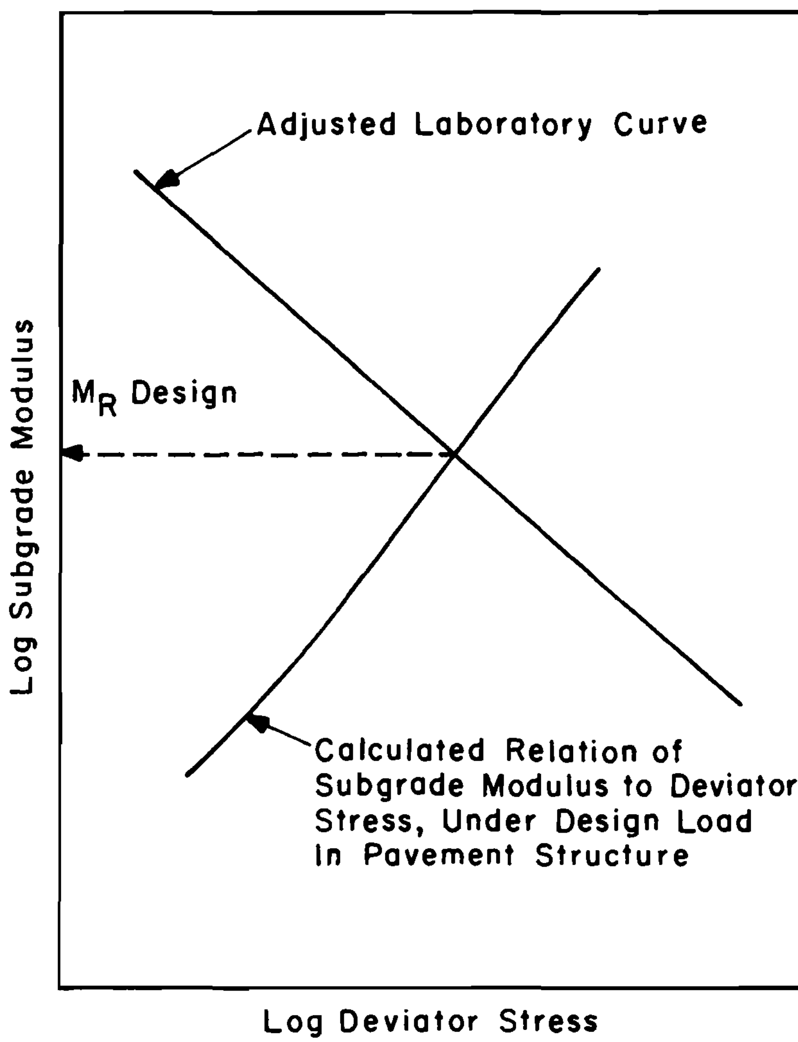


Fig I-3.6. Determination of design subgrade modulus.

existing pavement structure. By entering the design deflection, the corresponding subgrade modulus and deviator stress that would result in the subgrade under the pavement due to the deflection load are determined. The laboratory curve is then adjusted to include these values, as shown in Fig I-3.5c. By using this adjusted lab curve and a calculated relationship between subgrade modulus and deviator stress under the design load on the existing pavement, the design subgrade resilient modulus can be determined, as shown in Fig I-3.6.

This process of characterization of the subgrade is done internally in the RPOD2 program. For this purpose, laboratory data, design deflection, and deflection load, along with elastic properties and dimensions of all pavement layers, are necessary inputs. As an alternative to the laboratory data, the slope of the log resilient modulus versus log deviation stress line for subgrade material can be used as input.

Other inputs, such as horizontal and vertical crack or joint movements and crack or joint widths, are necessary for the reflection cracking analysis and are discussed in detail in Part III of this report.

OVERLAY THICKNESS ANALYSIS

As shown in Fig I-2.1, the overlay thickness analysis is based on two criteria: fatigue cracking and reflection cracking. Fatigue cracking can be prevented by using the correct overlay thickness, whereas reflection cracking can be prevented or minimized by, among other measures, increase overlay thickness or bondbreakers.

Fatigue Cracking Analysis

As previously mentioned, the fatigue cracking analysis is done using the RPOD2 computer program. Inputs into this program are traffic data, material properties, layer dimensions, pavement and overlay types, condition of existing pavement, and deflection measurements. The output is the overlay thickness required to prevent fatigue cracking under the conditions specified.

Computation of Stresses. Computations of stresses, strains, and deflections are done using a linear elastic layered program as a subroutine in RPOD2. McCullough (Ref 5) had previously used layered theory for analyzing overlays on rigid pavements in 1969 and stated:

> A comparison of layered theory and the generally accepted Westergaard theory used in design of Portland cement concrete pavements gave favorable correlation over a wide range of parameters expected in practice.

McCullough also pointed out that although the Westergaard equations, at that time (1969), had been associated with concrete pavements by pavement engineers, for approximately 40 years, its use for overlay design was eliminated in favor of layered theory. It can thus be seen that, though not the most conventional method, layered theory has been used with success to design overlays on rigid pavements.

Stress Factors. The stresses, strains, and deflections computed by layered theory are assumed to be interior stresses, strains, and deflections. Through an extensive study (Ref 1), stress factors have been determined to convert interior stresses to edge or corner stresses. This involved a study of field measurements of deflections and the solution of many problems for interior, edge, and corner conditions, using discrete element theory as well as Westergaard and Pickett theory. Stress factors used in RPOD2 can be seen in Table I-3.2.

CRC pavements are designed for edge loading conditions, and jointed pavements for corner loading conditions. For jointed pavements, this method requires that interior as well as corner deflections be taken on the existing pavement. This can then be used to determine a stress factor, using the relationship in Fig I-3.7. This information is fixed inside RPOD2 and stress factors are automatically determined that way. If this information is not available, however, a default value of 1.5 for the stress factor of JCP overlays on JCP existing pavements is used, which means that a ratio of corner to interior deflection of approximately 2.3 is then assumed.

Void Factors. Voids underneath a pavement cause an increase in stresses due to applied loads. This aspect has been studied in the development of the FHWA procedure, and the following values for void factors are suggested (Ref 1):

TABLE I-3.2. STRESS FACTORS SELECTED FOR VARIOUS OVERLAY-EXISTING PAVEMENT COMBINATIONS TO CONVERT INTERIOR STRESSES TO STRESSES FOR USE IN DESIGN (Ref 1)

Overlay - Existing Pavement Combination		*Stress Factor
Existing Pavement	Overlay	
CRCP	CRCP	1.2
JCP	CRCP	1.2
CRCP	JCP	1.3
JCP	JCP	1.4 - 1.8

*Based on field deflections

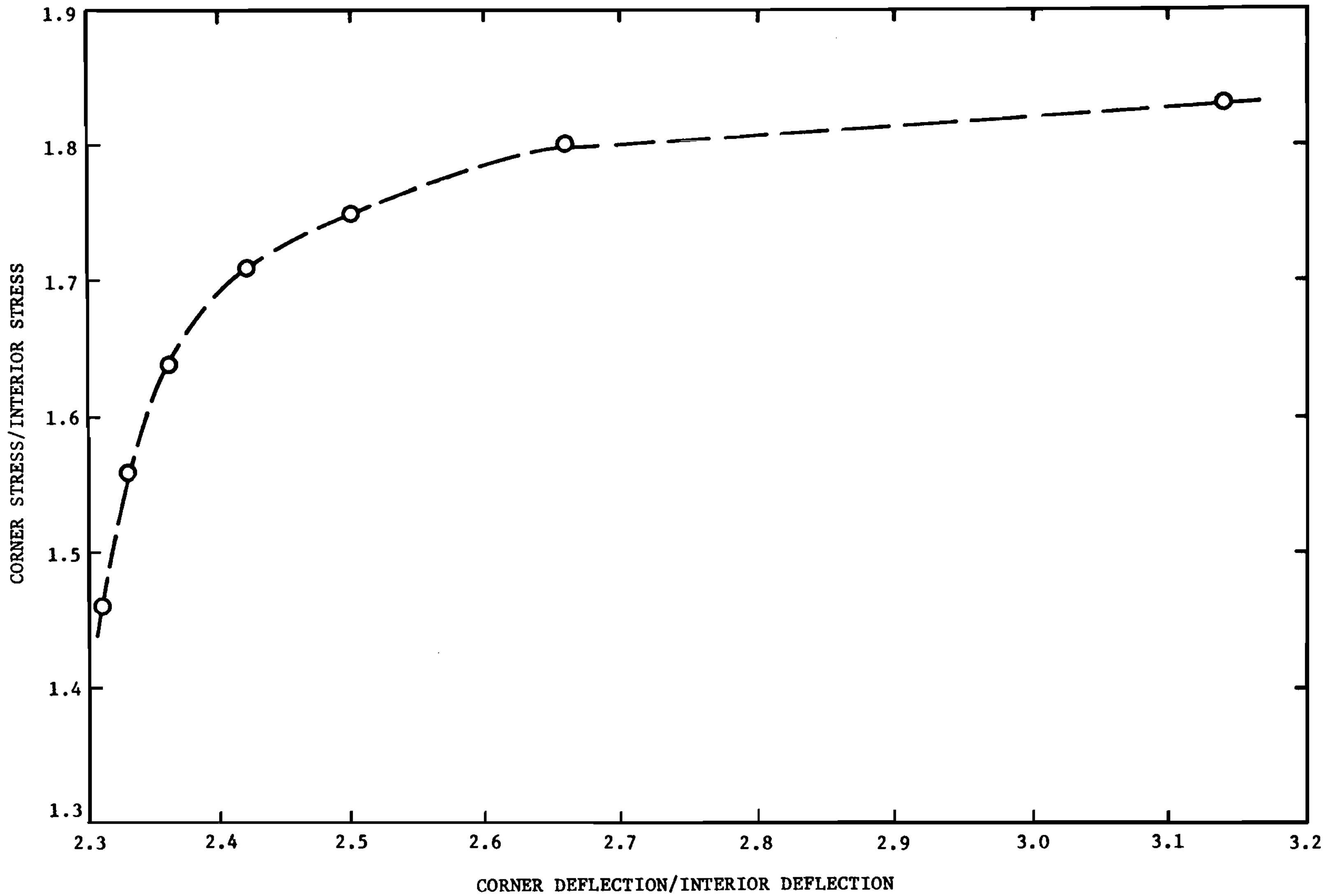


Fig I-3.7. Stress ratio curve for relating interior to corner stresses for a given Deflection Ratio. (Ref 1)

Condition	Void Factor
Edge	1.1
Corner	1.5

It is, however, pointed out that these values are only guidelines and that there is room for further development.

Fatigue Analysis. There is a relationship between allowable number of stress (or strain) applications and the magnitude of these stresses (or strains) given in an equation for portland cement concrete:

$$N = A(f/\sigma)^{B} \qquad \text{(I-3.2)}$$

where

N = number of axle loads until failure;

f = flexural strength of concrete, psi;

σ = computed tensile stress due to design load, psi; and

A,B = constants, depending on mixture characteristics.

By selection of the correct overlay thickness, it is possible to control the stress to give the desirable fatigue life (permissible stress applications) (Ref 10).

The fatigue equations were fixed inside the RPOD2 program as follows:

for portland cement concrete,

$$N = 23440\left(\frac{f}{\sigma}\right)^{3.21} \qquad \text{(I-3.3)}$$

for asphaltic concrete,

$$N = 9.7255 \times 10^{-15}\left(\frac{1}{\varepsilon}\right)^{5.16267} \qquad \text{(I-3.4)}$$

where

ε = computed strain due to design load.

Remaining Life. The concept of remaining life was used and defined by McCullough (Ref 5) as follows:

$$R_L(x,t,l,e,m,) = 1 - \sum_{i=1}^{n} \frac{N_{u-i}}{N_u}(x,t,l,e,m) \tag{I-3.5}$$

where

R_L = remaining life;

N_{u-i} = the number of load applications of level i experienced from the beginning to time t;

N_u = number of load applications of level i required to cause failure in simple loading;

(x,t,l,e,m) = functional notation to denote the subject relations are a matrix function of space, time, loading, environment, and material properties.

This concept is used in both the RPOD1 and the revised RPOD2 programs.

Position of Governing Stress. The position of the governing stress, used in the fatigue equation, is dependent on whether or not the existing pavement has any remaining life. If the existing pavement has remaining life, the stress at the bottom of the existing pavement is taken as the governing stress, but the allowable stress repetitions predicted with the fatigue equation using this stress are multiplied with the remaining life.

For pavements with no remaining life, a stress relieving layer is suggested for use between the existing pavement and the overlay. The governing stress is taken to be at the bottom of the overlay. The position of the governing stress for these different conditions can be seen in Fig I-3.8.

If the existing pavement has less than one percent remaining life, it is considered not to have remaining life. In the range of 1 to 25 percent remaining life, RPOD2 determines overlay thicknesses by considering the existing pavement both to have remaining life and not to have remaining life. This insures the most economical selection of overlay thickness.

Designs of asphalt concrete overlays on pavements without remaining life are handled in the RPOD2 program using a semi-infinite halfspace, which results in the same deflection under design load as the existing pavement. The difficulty of modeling a cracked pavement with layered theory has been overcome this way.

Reset of Existing Pavement Elastic Modulus In Case of No Remaining Life. For pavements with no remaining life or pavements with class 3 and 4 cracking, an effective modulus of the existing layer of 500,000 psi (3,447 MPa is used. For mechanically broken up pavements, the effective modulus is 70,000 psi (423 MPa). Moduli of subbases have been limited to values below the above mentioned effective moduli.

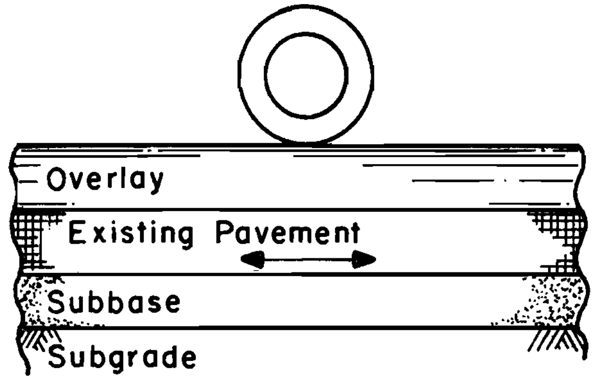
Selection Of Overlay Thickness. The RPOD2 program computes fatigue lives for 3, 6, 9, and 12 inch overlays on the existing pavement and then interpolates from this information to obtain the required overlay thickness for the specified overlay design traffic.

Reflection Cracking Analysis

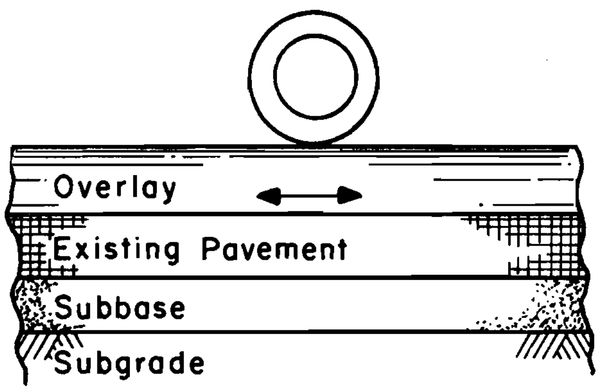
The second criterion used for selection of overlay thickness in the Texas SDHPT design procedure is that of reflection cracking.

Overlays over cracked or jointed concrete pavements or flexible pavements with cement stabilized bases pose the problem of reflection cracking. This is due to stress concentrations caused by horizontal (thermal) and vertical (load associated) movements in the joints or cracks (Ref 11).

FHWA Report Number FHWA-RD-77-66 (Ref 1) provides excellent background information on reflection cracking of which only Fig I-3.9 will be repeated here. Figure I-3.9 is a flow diagram of a process which can be used to determine the most suitable treatment for reducing reflection cracking.



a. Position of governing stress - pavement with remaining life.



b. Position of governing stress - pavement without remaining life.

Fig I-3.8. Positions of governing stress used in fatigue analysis for different pavements.

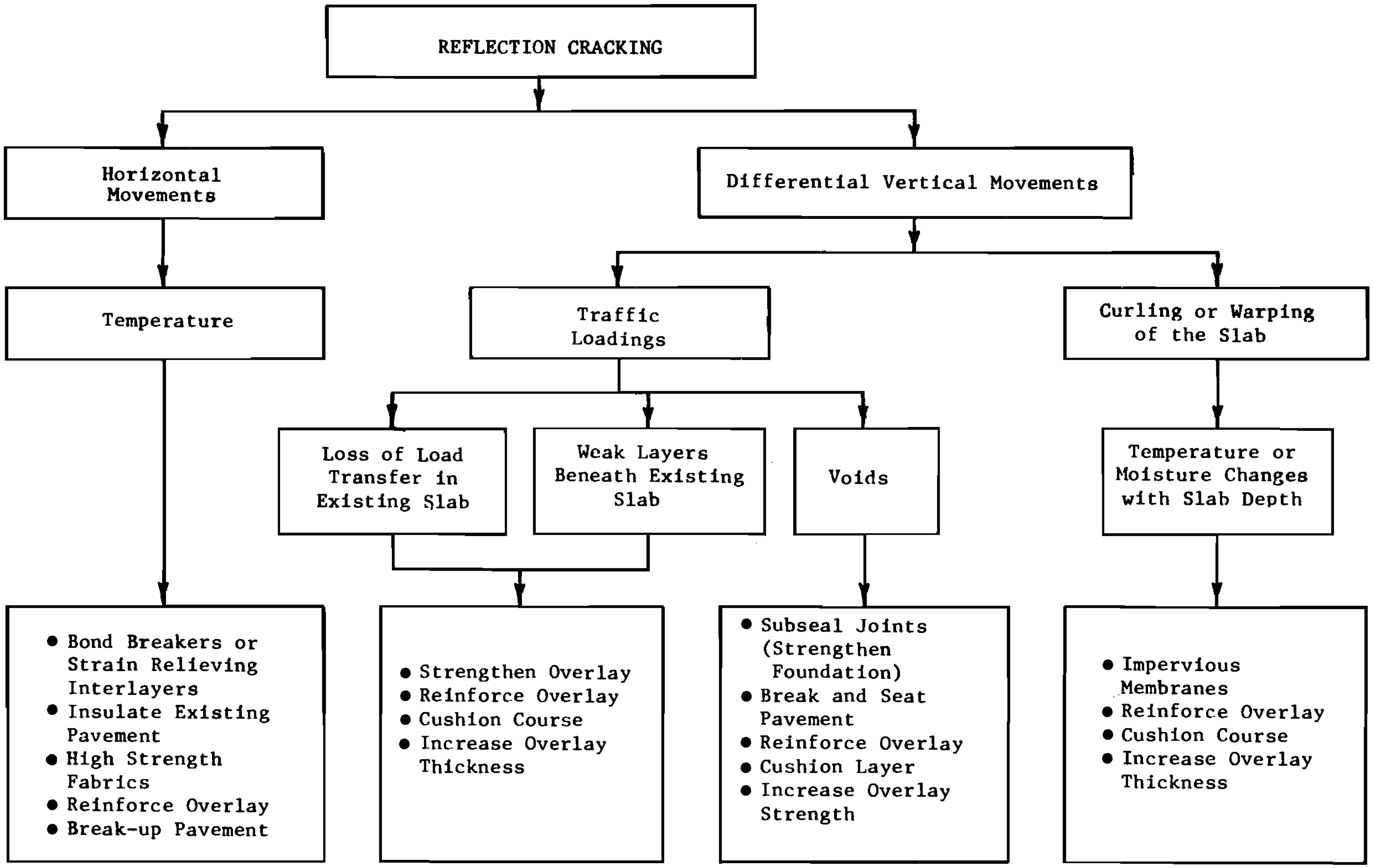


Fig I-3.9. Flow diagram to determine a treatment which can be considered to reduce reflection cracking (Ref 1).

It indicates that horizontal movements are caused by temperature changes and that treatment against this mode of failure would include measures to relieve stress concentrations, insulate the existing pavement, increase tensile strength of the overlay, or decrease the amount of movement in the joint or crack by breaking it up mechanically and in that way decreasing the slab length.

Furthermore, it can be seen that differential vertical movements are caused by traffic loadings and curling or warping of the slab. Load associated movements are due to inadequate load transfer at the joints or cracks, insufficient subgrade support, or a lack of subgrade support (voids). Those measures that would improve this problem are increased overlay strength and strain relieving layers. If voids are present, a combination of strengthening of the foundation, strengthening of the overlay, and relieving of the strain could be considered in design.

Curling or warping of the slab could be caused by differential temperature or moisture changes with slab depth. Waterproofing, insulation, strain relieving layers, and increased overlay strength are possible solutions to this problem. Treybig et al. (Ref 1) give information on this subject.

In the past, prevention of reflection cracking in overlays over PCC was, to a large degree, based on experience (Ref 1). The development of a model to predict strains in the overlay due to relative movements in the underlying joint or crack, as used in the FHWA design procedure, is a much needed step in the overlay design field.

This model is primarily concerned with asphaltic concrete overlays but can be used for portland cement concrete overlays, provided the procedure is reviewed and the assumptions involved are recognized.

This design procedure considers basically two failure modes in the case of reflection cracking (Ref 1):

(1) an opening mode due to horizontal movements in the existing pavement due to temperature changes;

(2) a shearing mode resulting from inadequate load transfer across a joint or crack.

Some of the basic assumptions made in developing this model were:

(1) There is linear elasticity and all the associated assumptions.

(2) The governing equation is that of static equilibrium of forces acting on the pavement.

(3) Temperature variations are uniformly distributed in the existing slab.

(4) Concrete movement is continuous with slab length.

(5) Movement of a layer is constant through layer thickness.

(6) Material properties are uniform in all directions throughout the layer.

The reflection cracking subsystem has been computerized. The computer program performing this analysis is called RFLCR1 (Ref 1). Inputs to this computer program can be seen on Table I-3.3, and the outputs are

(1) shear strains in the overlay caused by differential vertical movement in the joint or crack (due to traffic loadings) and

(2) tensile strains in the overlay caused by horizontal movement in the joint or crack (due to a drop in temperature).

Conceptually, the reflection cracking analysis consists of evaluating overlay thickness using the following (Ref 1):

$$C_R = f(E_o, E, D, \Delta T, \alpha, F_i, w_d, X_{BB}) \tag{I-3.6}$$

where

C_R = reflection cracking,

E_o = creep modulus of asphalt concrete or portland cement concrete,

E = dynamic modulus of asphalt concrete or portland cement concrete,

D = thickness of existing pavement of overlay,

TABLE I-3.3. INPUT VARIABLES FOR RFLCR1

Category	Input Variables
Existing Pavement Properties	Existing pavement type (JRCP, JCP, CRCP, etc).
	Existing pavement conditions (cracked or uncracked)
	Elastic modulus, psi
	Thermal Coefficient, in/in/^{o}F
	Thickness, inches
	Density, pcf.
	Joint or crack spacing, feet
	Movement at sliding, inches
Existing Pavement Reinforcement Properties	Elastic modulus of steel, psi
	Steel thermal coefficient, in/in/^{o}F
	Area of steel per foot width, in^{2}
	Perimeter of steel, inches
	Steel to concrete bonding stress, psi
Horizontal Characterization of Pavement	Mean high temperature, ^{o}F
	Joint width at high temperature, inches
	Mean low temperature, ^{o}F
	Joint width at low temperature, inches
	Minimum temperature observed since construction of pavement, ^{o}F
Vertical Characterization	Mean joint width, inches
	Load transfer, percent/100
Overlay Properties	Overlay Type
	Creep modulus, psi
	Thermal coefficient mi/mi/^{o}F
	Thickness, inches
	Density, pcf
	Poisson's ratio
	Dynamic modulus, psi
	Overlay to existing surface bonding stress, psi
Overlay Reinforcement Properties	Overlay reinforcement type
	Elastic modulus, psi
	Thermal coefficient in/in/^{o}F
	Area of reinforcement per foot width, in^{2}
	Allowable tensile strain, in/in
Bond Breaker	Width or length in direction of traffic, feet
Intermediate Layer Properties	Creep modulus, psi
	Thermal coefficient, in/in/^{o}F
	Thickness, inches
	Density, pcf
	Allowable strain, in/in
	Dynamic modulus, psi
Temperature changes, Design load	Design temperature change for existing pavement, ^{o}F
	Design temperature change for intermediate layer, ^{o}F
	Design temperature change for overlay, ^{o}F
	Design load weight, in pounds
	Width of design load in inches

ΔT = temperature change of pavement materials,

α = coefficient of volume change for pavement materials,

F_i = force movement relationship between pavement layers resulting from friction, adhesion, bearing, etc.,

w_d = differential deflection at crack or joint,

X_{BB} = width of bondbreakers.

In addition, the program also calculates and gives information concerning maximum tensile stresses in the existing pavement prior to overlay, the slope of the friction curve used in the analysis, and values of the restraint coefficient prior to and after overlay. This restraint coefficient represents any force which will restrict free concrete movement. Field measurements are used to calibrate the model to the actual pavement.

It can be noted from Table I-3.3 that there are numerous input requirements for calculating these strains. These are discussed in Part III. A limited sensitivity analysis of the RFLCR1 program is discussed in Chapter II-3.

This program allows for the use of bondbreakers, interlayers, and reinforcement in the overlay, as can be seen in Fig I-3.2.

Figure I-3.10 is a flow diagram indicating the reflection cracking subsystem. Using the input variables listed in Table I-3.3 and the thickness predicted by the fatigue cracking sybsystem (PROD2), the horizontal tensile and vertical shear strains in the overlay are computed by means of the RFLCR1 program. These strains are then compared to allowable strains in order to establish whether reflection cracking is likely to occur. If reflection cracking is probable, the design might either be changed or a decision made to maintain the resulting cracks with increased cost. Redesign might involve the increase of overlay thickness, the introduction of a bondbreaker, the use of a strain relieving intermediate layer, or the use

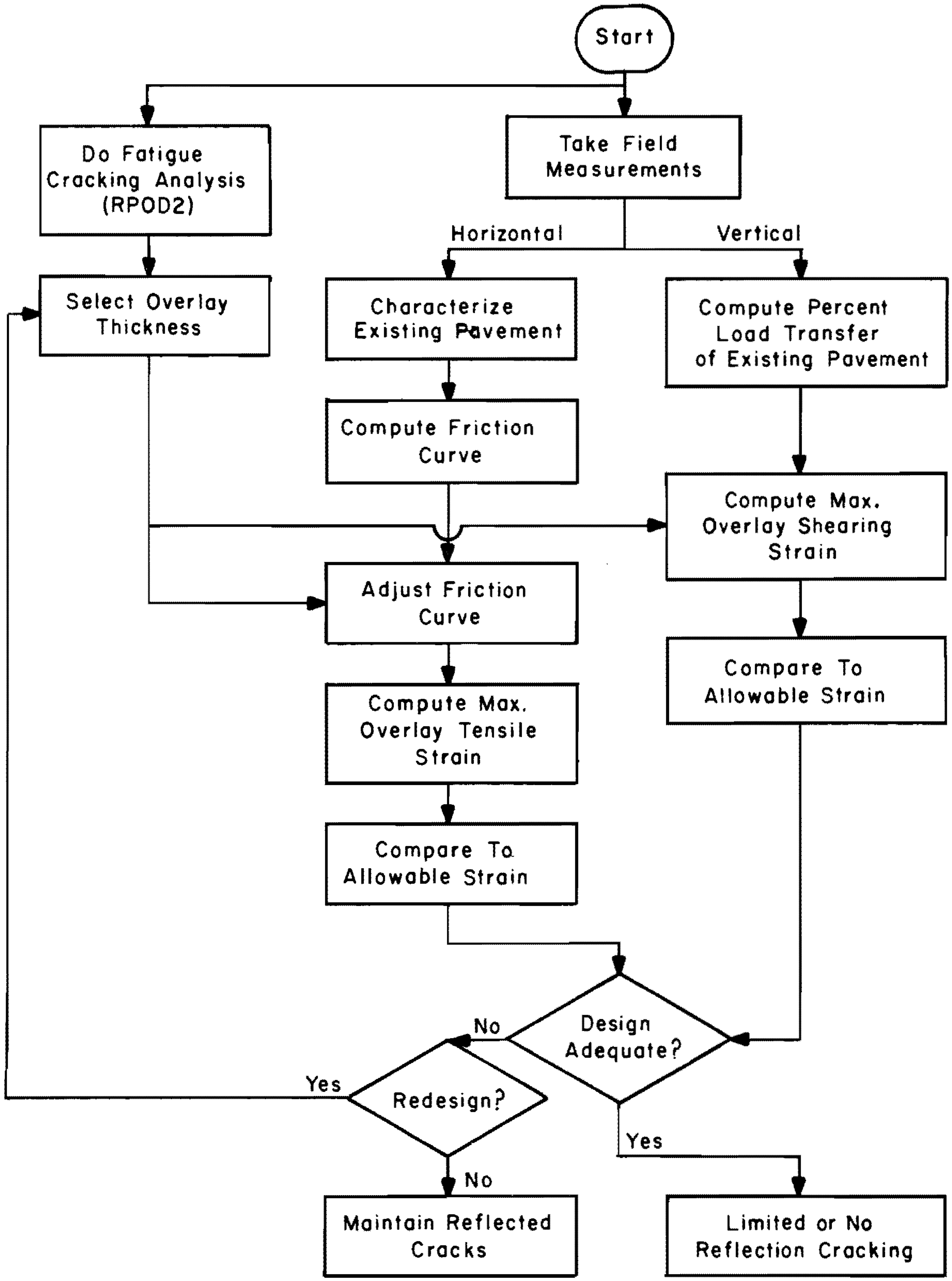


Fig I-3.10. Flow diagram showing the overall reflection cracking analysis procedure.

of reinforcement in the overlay. The computer program is capable of handling all these different options.

Finally, the overlay thickness selected must satisfy both the fatigue and the reflection cracking criteria, as shown in Fig I-2.1.

CHAPTER I-4. ILLUSTRATIVE OVERLAY DESIGN PROBLEM

This chapter presents a design example which illustrates the use of the Texas SDHPT overlay design method for rigid pavements. This example involves both the fatigue cracking and reflection cracking subsystems. The four programs PLOT2, TVAL2, RPOD2 and RFLCR1 were used in the example and coded data input are presented for illustration. Appendix 1 contains example computer output. The User's Manual in Part III of this report provides detailed information on the use of this design precedure.

BACKGROUND INFORMATION

A substantial increase in traffic is expected on a 5,900 feet long section of divided highway in Colorado County, Texas. The existing pavement is CRCP and to handle the anticipated traffic, the decision was made to overlay the pavement with either a bonded CRCP overlay or an asphaltic concrete overlay. Designs will be made for each type of overlay for comparison purposes. This highway is located on rolling grassy terrain with isolated patches of trees. Complete closure to traffic of this section, or a portion thereof, can be facilitated only by introducing a four-mile long detour, which would cause considerable delay and inconvenience to road users. Overlays for the two roadways of a divided highway should be designed separately. In this design example only one such design is included.

CONDITION SURVEY

A condition survey was conducted on this pavement and results are shown in Table I-4.1. A complete description of the condition survey procedure is shown in Part III. This condition survey indicated that the section between stations 15 and 27 exhibits class 3 cracks and that a considerable amount of spalling is present. The drainage of that portion is poor. The rest of the pavement is generally in good condition, it has class 1 and 2 cracks, and drainage conditions are good. Construction

TABLE I-4.1. ILLUSTRATIVE OVERLAY DESIGN PROBLEM - CONDITION SURVEY

Stations		Cracking					Faulting	Spalling	Pumping	Drainage		Grade			Other Comments
From	To	Uncr.	Cl. 1	Cl. 2	Cl. 3	Cl. 4				Good	Poor	Cut	Fill	Natural	
0	1		✓						✓	✓			✓		
1	2		✓							✓			✓		
2	3			✓				✓		✓			✓		
3	4		✓							✓			✓		
4	5		✓							✓			✓		
5	6			✓						✓			✓		
6	7			✓						✓			✓		
7	8			✓						✓			✓		
8	9			✓						✓			✓		
9	10			✓						✓			✓		
10	11		✓							✓			✓		
11	12		✓							✓			✓		
12	13		✓							✓			✓		
13	14		✓							✓			✓		
14	15			✓						✓				✓	
15	16				✓			✓			✓			✓	
16	17				✓			✓			✓			✓	
17	18				✓			✓			✓			✓	
18	19				✓			✓			✓			✓	
19	20			✓							✓			✓	
20	21			✓							✓			✓	
21	22				✓			✓			✓			✓	
22	23			✓							✓			✓	
23	24				✓			✓			✓			✓	
24	25				✓			✓			✓			✓	
25	26			✓						✓				✓	
26	27			✓				✓		✓				✓	
27	28		✓							✓			✓		
28	29		✓							✓			✓		
29	30		✓							✓		✓			
30	31			✓						✓		✓	✓		
31	32		✓							✓		✓			
32	33		✓							✓		✓			
33	34		✓						✓	✓		✓			
34	35		✓							✓				✓	
35	36		✓							✓				✓	
36	37		✓							✓				✓	
37	38		✓							✓				✓	
38	39		✓							✓			✓		
39	40		✓							✓			✓		
40	41		✓							✓					
41	42		✓							✓			✓		
42	43		✓							✓			✓		
43	44		✓							✓			✓		
44	45		✓							✓				✓	
45	46		✓							✓				✓	
46	47			✓						✓		✓			
47	48			✓						✓		✓			
48	49			✓						✓		✓			
49	50			✓						✓			✓		
50	51			✓						✓			✓		
51	52			✓						✓			✓		
52	53		✓							✓		✓			
53	54		✓							✓		✓			
54	55		✓							✓		✓			
55	56			✓						✓		✓			
56	57			✓						✓		✓			
57	58			✓						✓		✓			
58	59			✓						✓		✓			

records indicated that the CRC pavement is 8 inches (203 mm) thick throughout the length of the section. The first 2,550 feet of pavement has a 6-inch (152-mm) cement stabilized subbase and for the remainder of the section, a 4-inch (102-mm) asphalt stabilized base was used. Figure I-4.1 shows a longitudinal pavement section as obtained from construction plans.

Field data was also collected for use in the reflection cracking subsystem for the asphaltic concrete overlay and are presented later, with the reflection cracking analysis.

DEFLECTION SURVEY

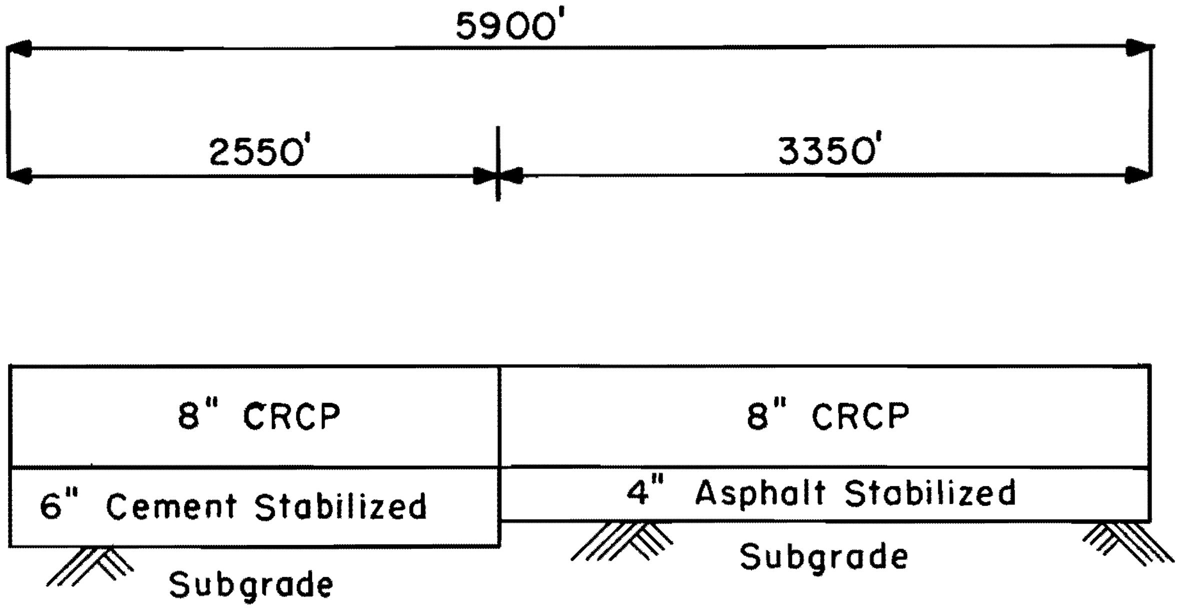
A deflection survey, based on the procedure described in Part III, was made on this pavement section, using the Dynaflect as the deflection measuring device. Measurements were obtained on each roadway at 100-foot (61-m) intervals, 3 feet (914-mm) from the outer edge of the pavement. These measurements were taken between cracks to represent an interior condition.

Deflection measurements were also made at approximately 300-foot (91.5-m) intervals to determine the differential deflection at cracks. The Dynaflect load wheels were placed immediately at one side of the crack and the sensors positioned in a way to allow deflection measurements at both the loaded and unloaded sides of the crack. These deflection data are necessary inputs to the reflection cracking subsystem and will be discussed later, under the reflection cracking analysis.

SELECTION OF DESIGN SECTIONS

Both the condition survey and the deflection survey were used to divide the pavement into design sections. The procedure for selection of design sections is described in Part III. A review of the condition survey information (Table I-4.1), as well as the construction records (Fig I-4.1), led to the following tentative design sections:

Section 1	Stations 0 to 15
Section 2	Stations 15 to 25
Section 3	Stations 25 to 59



1 ft.=.3048 m
1 in.= 25.4 mm

Fig I-4.1. Longitudinal section of pavement to be overlaid.

The next step was to analyze the deflection information. The computer program PLOT2 was used to plot a profile of interior deflections at 100-foot (30.5-m) intervals. Table I-4.2 is a coding sheet indicating the required input for the PLOT2 program and Fig I-4.2 is the computer plotted profile. The longitudinal distance in feet is printed on the x-axis and the deflection values in mils are plotted on the y-axis. The plotted points were connected by hand to complete the profile. The pavement was divided into three design sections, as indicated on Fig I-4.2, by visual inspection of the deflection profile. These three sections coincided well with those tentatively selected based on the condition survey and construction records.

The three sections were then statistically tested to see if they were significantly different, using the TVAL2 computer program. The coded data input for the TVAL2 program can be seen on Table I-4.3. It should be noted that Card Type 4 for the PLOT2 input (Table I-4.2) is reused as Card Type 4 for the TVAL2 input (Table I-4.3), so that deflection data need only be punched once. The output of the TVAL2 program is presented in Table I-4.4. This output includes a listing of all deflections evaluated for each section as well as their means and standard deviations. Each section is then compared to the other sections to see if they are significantly different. The results, on Table I-4.4, show that each comparison failed to pass the "Student's t" test, which means they are significantly different and should be treated as separate design sections. Table I-4.5 indicates the final design sections selected as well as the design deflection for each section.

According to the condition survey data, design sections 1 and 3 were classified as category 1 pavements (class 1 and 2 cracking) with a potential of having remaining life. Design section 2 exhibits class 3 cracking and was classified under category 2 (class 3 and 4 cracking). It is important to note that this distinction in classification has a significant effect on the results as a pavement with excessive cracking is not considered to carry tensile stresses.

TABLE I-4.2. INPUT DATA FOR PROGRAM PLOT2

```
         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
          ILLUSTRATIVE OVERLAY DESIGN PROBLEM
 DYNAFLECT READING - SENSOR 1 STATIONING  (FEET)   DEFLECTION  (MILS)
(2A4,A2,F10.0,F5.0)
*    100  .60
FINISH
```

*A total of 59 cards of this type were included, one for each deflection measurement.

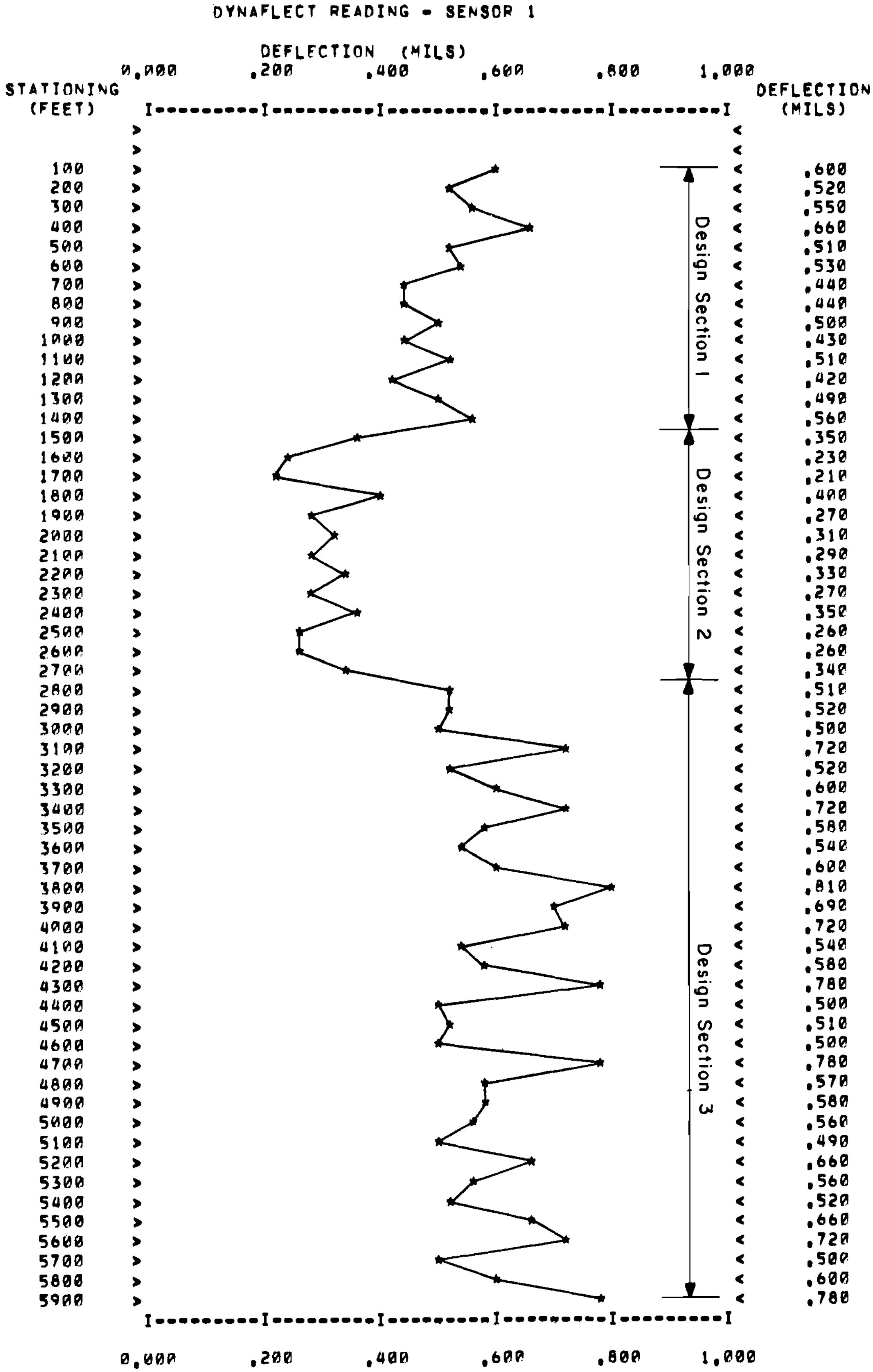


Fig I-4.2. Deflection profile plot for pavement to be overlaid.

TABLE I-4.3. INPUT DATA FOR PROGRAM TVAL2

```
         1    1    2    2    3    3    4    4    5    5    6    6    7    7    8
    5    0    5    0    5    0    5    0    5    0    5    0    5    0    5    0
   59    3ILLUSTRATIVE OVERLAY DESIGN PROBLEM
   14   13   32
   95 95. (10X,F10.0,F5.0)
*    100  .60
```

*Card reused from PLOT2 input. Fifty-nine cards were included, one for each deflection measurement. The position of measurement is not read by TVAL2.

TABLE I-4.4. OUTPUT FROM PROGRAM TVAL2

TVAL2 - DEFLECTION SECTION COMPARISON PROGRAM, VERSION 2.0

ILLUSTRATIVE OVERLAY DESIGN PROBLEM.

DEFLECTIONS FOR EACH SECTION

SECTION 1	.600	.520	.550	.660
	.510	.530	.440	.440
	.500	.430	.510	.420
	.490	.560		
SECTION 2	.350	.230	.210	.400
	.270	.310	.290	.330
	.270	.350	.260	.260
	.340			
SECTION 3	.510	.520	.500	.720
	.520	.600	.720	.580
	.540	.600	.810	.690
	.720	.540	.580	.780
	.500	.510	.500	.780
	.570	.580	.560	.490
	.660	.560	.520	.660
	.720	.500	.600	.780

	MEAN	STANDARD DEVIATION
SECTION 1	.511	.068
SECTION 2	.298	.054
SECTION 3	.607	.100

SECTION VS.	SECTION	DF	CALCULATED T	95 P/C CONF. LEVEL TABLE T	PASS/FAIL
1	2	25	8.974	2.060	FAIL
1	3	44	3.237	2.017	FAIL
2	3	43	10.445	2.018	FAIL

DESIGN DEFLECTION CONFIDENCE LEVEL 95.0

SECTION	INTERIOR DESIGN DEFLECTION
1	.623
2	.387
3	.772

TABLE I-4.5. FINAL DESIGN SECTIONS AND DESIGN DEFLECTIONS

Design Section Number	Station Limits	Design Deflection, mils
1	0 + 00 to 14 + 50	.623
2	14 + 50 to 25 + 50	.387
3	25 + 50 to 59 + 00	.772

1 mil = .025 mm

MATERIAL PROPERTIES

Properties of materials were determined according to the testing procedures outlined in part III. The following boring plan was adopted for this project, after selection of design sections:

Design sections 1 and 2, one boring each, and
Design section 3, three borings.

Core samples were obtained from the existing pavement and from subbase materials. Undistrubed tube samples of the subgrade material were used in laboratory tests. The laboratory-determined material properties for this design example are listed in Table I-4.6. Poisson's ratio values were not determined and it was decided to use the fixed values for the different pavement materials, as provided in the RPOD2 program. Layer thicknesses were determined from borings and checked against the construction data. Laboratory-determined values of material properties for this illustrative example are shown on Table I-4.7.

TRAFFIC COMPUTATIONS

The traffic information necessary for the overlay design was determined as specified in Part III. A directional distribution factor of 0.5 and a lane distribution factor of 1 was used for this divided highway. Mixed traffic was all converted to 18-kip (80-kN) equivalent single axle loads (ESAL) using the AASHTO equivalency factors. It is estimated that the pavement has already carried 4 million 18-kip (80-kN) ESAL since construction. Because of limited funds for this project, it was decided to investigate two alternative designs:

Design life A	7 million 18-kip (80-kN) ESAL
Design life B	10 million 18-kip (80-kN) ESAL

The required thicknesses for these alternative designs, together with the construction funds available, will be used in selecting the final overlay thickness and design life.

The same traffic information was used for all three design sections.

TABLE I-4.6. MATERIALS TESTS REQUIRED FOR ILLUSTRATIVE OVERLAY DESIGN PROBLEM

Location	Material	Property Tested
Existing Pavement	Portland Cement Concrete	Elastic Modulus
		Flexural Strength
	Cement Stabilized Subbase	Elastic Modulus
	Asphalt Stabilized Subbase	Dynamic Modulus
	Subgrade	Resilient Modulus
Overlay	Portland Cement Concrete	Elastic Modulus
		Flexural Strength
	Asphalt Concrete	Dynamic Modulus

TABLE I-4.7. MATERIAL PROPERTIES FOR DIFFERENT DESIGN SECTIONS

Material Type	Section 1		Section 2		Section 3	
	Elastic Modulus, psi	Flexural Strength, psi	Elastic Modulus, psi	Flexural Strength, psi	Elastic Modulus, psi	Flexural Strength, psi
Proposed AC Overlay	400000	-	400000	-	400000	-
Proposed CRCP Overlay	4500000	640	4500000	640	4500000	640
Existing CRCP Overlay	4200000	570	3800000	670	3200000	680
Cement Stabilized Subbase	500000	-	500000	-	-	-
Asphalt Stabilized Subbase	-	-	-	-	25000	-
	Resilient Modulus, psi	Deviator Stress, psi	Resilient Modulus, psi	Deviator Stress, psi	Resilient Modulus, psi	Deviator Stress, psi
Subgrade	22867	1	44642	1	34300	1
	22400	2	29673	2	30489	2
	16530	5	15686	5	28583	5
	14442	8	5859	8	22866	8

1 psi = 6.894 KPa

TABLE I-4.8. RPOD2 INPUT DATA FOR SECTION 1

```
PROBLEM  1        1.
ILLUSTRATIVE OVERLAY DESIGN PROBLEM,SECTION 1,CRCP OVERLAY.
PAVEMENT 3  4000000. 570.     TYPE 1,2
LAYER    1  4200000.  8.      CRCP
LAYER    2   500000.  6.      STAB
LAYER    3      5000.         SUBG
LAB DATA 4
    22867.     1.       22400.     2.       16530.     5.       14442.     8.
DEFLECT   .000623
OVERLAY     4500000.      640.CRCPBOND
TRAFFIC  2
  7000000. 10000000.
PROBLEM  2        1.
ILLUSTRATIVE OVERLAY DESIGN PROBLEM,SECTION 1,AC OVERLAY.
OVERLAY      400000.          AC
END
```

FATIGUE CRACKING ANALYSIS

A detailed description of the procedure for conducting the fatigue cracking analysis is given in the User's Manual (Part III). This analysis is computerized and the RPOD2 program is used for this purpose. Table I-4.8 shows the coded input data sheet to run RPOD2 for design Section 1 of this example. It will be noted on this sheet that in order to switch from the bonded CRCP overlay in the first problem to the asphaltic concrete overlay in the second problem (both for design Section 1), it was only necessary to make changes on the "overlay" card. The rest of the information was taken from the previous problem. For design Sections 2 and 3, data were input in a similar way. Since Section 2 was classified as a Category 2 pavement, the existing pavement elastic modulus used in that case was 500,000 psi (3,447 MPa), as recommended in the User's Manual. The elastic modulus of the existing CRCP for Section 2 was determined, because it is required for the reflection cracking analysis.

The RPOD2 output is included in Appendix I. Problems 1 and 2 are, respectively, the bonded CRCP overlay and the asphaltic concrete overlay designs for design Section 1. Problems 3 and 4 are the CRCP and asphaltic concrete overlay designs for design Section 2, and problems 5 and 6 are the overlay designs for Section 3. The output indicates all the input variables, such as existing pavement characteristics, deflection data, laboratory test data for subgrade material, overlay characteristics, and design traffic. Also included in the output are the system results, which consist of overlay life predictions, calculated fatigue lives for four different overlay thicknesses, a plot of overlay thickness versus fatigue life, and the required overlay thicknesses for the design life specified. It will be noted on this output that design Section 1 has less than 25 percent remaining life, and, therefore, the program automatically calculated required overlay thicknesses for both the case where the existing pavement has remaining life and the no-remaining life case. The thinner of the two thicknesses was then selected as a design thickness. Design Section 2 was classified as a Category 2 pavement and, therefore, not considered to have remaining life. Although design Section 3 was originally classified as Category 1 pavement, the remaining life calculations pointed out that it did not have remaining life, and the program automatically treated this

TABLE I-4.9. SUMMARY OF OVERLAY THICKNESSES
(IN INCHES)

Overlay Type	CRCP		AC	
Design Life ($x 10^6$)	7	10	7	10
Design Section				
1	5.2	6.4	8.5	9.0
2	6.0	6.8	8.0	8.5
3	7.1	7.9	9.0	9.5

1 inch = 25.4 mm

section as a Category 2 pavement. The RPOD2-predicted overlay thicknesses are summarized in Table I-4.9. Some inconsistencies in overlay thickness for the design sections will be observed. For example, the overlay thickness required for Section 2 was greater than that required for Section 1 for the CRCP overlay while thinner for the AC overlay. These inconsistencies are a result of the differences in the analyses used for each design and are not considered significant.

Table I-4.9 will be used in the process of selecting the final design thicknesses. Since, however, the asphaltic concrete overlay may be subjected to reflection cracking, the reflection cracking analysis is conducted next.

REFLECTION CRACKING ANALYSIS

The User's Manual (Part III) provides a step-by-step guide for performing the reflection cracking analysis. The computer program RFLCR1 is used to compute strains in the overlay caused by thermal and load associated relative movements at cracks in the CRCP. These computed strains are then compared to predetermined maximum allowable strains to predict whether reflection cracking will occur in the overlay. The reflection cracking analysis is only conducted for the asphaltic concrete overlay, since the RPOD2 program provides for the use of a bondbreaker for rigid overlays on pavements with class 3 and 4 cracking, mechanically broken up pavements, and pavements with no remaining life.

Existing Pavement Properties

Simultaneously with the condition survey, the crack spacing was determined according to the method specified in the User's Manual. Values of means and standard deviations of crack spacing, as well as selected design crack spacings, for the different design sections are listed in Table I-4.10.

Elastic properties and thickness values for the existing pavement layers were the same for the reflection cracking analysis previously used with the RPOD2 program. Densities of pavement materials were determined from cores during the sampling process. The thermal coefficients of steel

TABLE I-4.10. SUMMARY OF CRACK SPACING DATA

Design Section	Crack Spacing, feet		
	Mean	Standard Deviation	Design Crack Spacing
1	4.63	2.67	8.0
2	4.68	2.80	8.2
3	9.11	6.77	17.8

1 foot = .3048 m

and concrete were not determined, but values suggested in the User's Manual (Part III) were used. The movement at which sliding occurs was estimated as 0.2 inch (5-mm) for both subbase types. Construction plans indicated that 39 5/8-inch (16-mm)-diameter reinforcing bars were used in the 24-foot (7.3-m) cross section of existing CRCP. Table I-4.11 is a summary of the existing pavement properties used for this design example.

Characterization Measurements

Horizontal and vertical characterization measurements taken during the condition survey are presented on Tables I-4.12 and I-4.13, respectively. The horizontal movement data were evaluated, as indicated on Fig I-4.3, according to the method suggested in Part III. A 90 percent confidence interval was selected for use in relation with horizontal movements. For determining the percentage of load transfer at cracks, also for a 90 percent confidence interval, the vertical differential deflection data were used. The procedure for determining the design value for percentage of load transfer is described in Part III. Selected values for design for horizontal and vertical characterizations of the existing pavement are given in Table I-4.14. The minimum temperature observed, for Colorado County, was determined from Fig III-4.4 in the User's Manual.

Overlay Properties

The same values for dynamic modulus and Poisson's ratio as used in the fatigue cracking subsystem were used in the reflection cracking analysis. The overlay thicknesses predicted by RPOD2 were used to check whether reflection cracking could be expected in the asphaltic concrete overlays. No laboratory data existed for the creep modulus, and it was decided to use the procedure for determining this material property by means of nomographs, as described in Part III. The overlay to existing pavement bonding stress was selected from Table III-4.4 in the User's Manual. A summary of overlay properties is given in Table I-4.15.

To relieve the horizontal tensile strain in the overlay bondbreaker widths of 2 feet (610-mm), 2 feet (610-mm), and 4 feet (1.22-m) were used for design Sections 1, 2, and 3, respectively. The design temperature change was determined using Figs III-4.10 and III-4.11 in the User's Manual

TABLE I-4.11. SUMMARY OF EXISTING PAVEMENT PROPERTIES

Variable	Value		
	Section 1	Section 2	Section 3
Concrete elastic modulus, psi	4200000	3800000	4500000
Concrete thermal coefficient, in./in./°F	.000006	.000006	.000006
Thickness, in.	8.0	8.0	8.0
Density, pcf	140.0	140.0	140.0
Design crack spacing, ft	8.0	8.2	17.8
Movement at sliding, in.	.02	.02	.02
Steel elastic modulus, psi	29000000	29000000	29000000
Steel thermal coefficient, in./in./°F	.000006	.000006	.000006
Area of steel/ft width, in.^{2}	.48	.48	.48
Perimeter of steel/ft width, in.	3.19	3.19	3.19
Steel to concrete bonding stress, psi	295	295	295

1 psi = 6.894 KPa
1 in./in./°F = 1.8 mm/mm/°C
1 in. = 25.4 mm
1 pcf = 16.01 Kgm^{-3}

TABLE I-4.12. ILLUSTRATIVE OVERLAY DESIGN PROBLEM - HORIZONTAL MOVEMENTS

Measurement Number or Location	Joint or Avg. Crack Spacing, feet L	Air Temperature, °F T_L	*Joint or Crack Width, inches $Y(T_L)$	Air Temperature, °F T_H	*Joint or Crack Width, inches $Y(T_H)$	Temperature Change, °F ΔT_c	Joint or Crack Movement, inches $Y(T_L) - Y(T_H)$
100	85	69	.024	81	.020	12	.004
400	120	70	.030	80	.023	10	.007
700	75	70	.022	80	.018	10	.004
1000	79	71	.022	80	.019	9	.003
1300	96	70	.025	79	.021	11	.004
1600	108	70	.032	80	.027	10	.005
1900	80	70	.030	80	.026	10	.004
2200	99	70	.031	80	.027	10	.004
2500	93	70	.030	80	.027	10	.003
2800	199	70	.026	80	.015	10	.011
3100	95	70	.013	80	.008	10	.005
3400	179	70	.024	80	.014	10	.010
3700	116	70	.016	80	.010	10	.006
4100	68	70	.011	80	.007	10	.004
4400	144	70	.017	80	.010	10	.007
4700	181	70	.023	80	.012	10	.011
5000	55	70	.007	80	.004	10	.003
5300	216	70	.026	80	.016	10	.010

*Measurement device: microscope   1 in. = 25.4 mm,   (°F-32) x 5/9 = °C

TABLE I-4.13. ILLUSTRATIVE OVERLAY DESIGN PROBLEM - DIFFERENTIAL VERTICAL DEFLECTIONS

Measurement Number or Location	Joint Width, inches	Temperature, °F	*Deflection, mils		Differential Deflection, inches	Percent Load Transfer, %
			Loaded Joint W_L	Unloaded Joint W_U	W_d	L_T
100	.021	84	.69	.65	.04	94.2
400	.025	84	.63	.58	.05	92.1
700	.020	84	.53	.51	.02	96.2
1000	.020	85	.37	.35	.02	94.5
1300	.024	85	.55	.52	.03	94.5
1600	.029	85	.30	.27	.03	90.0
1900	.028	85	.37	.32	.05	86.4
2200	.030	85	.27	.23	.04	85.2
2500	.029	85	.26	.22	.04	84.6
2800	.021	86	.56	.50	.06	89.3
3100	.012	86	.78	.73	.05	93.5
3400	.019	86	.93	.82	.11	88.2
3700	.014	86	.67	.61	.06	91.0
4100	.010	86	.78	.73	.05	93.6
4400	.015	87	.81	.73	.08	90.1
4700	.018	87	.57	.53	.04	93.0
5000	.006	87	.75	.72	.03	94.7
5300	.020	87	.54	.47	.07	87.0

*Dynaflect measurements

(°F-32) x 5/9 = °C, 1 in. = 25.4 mm

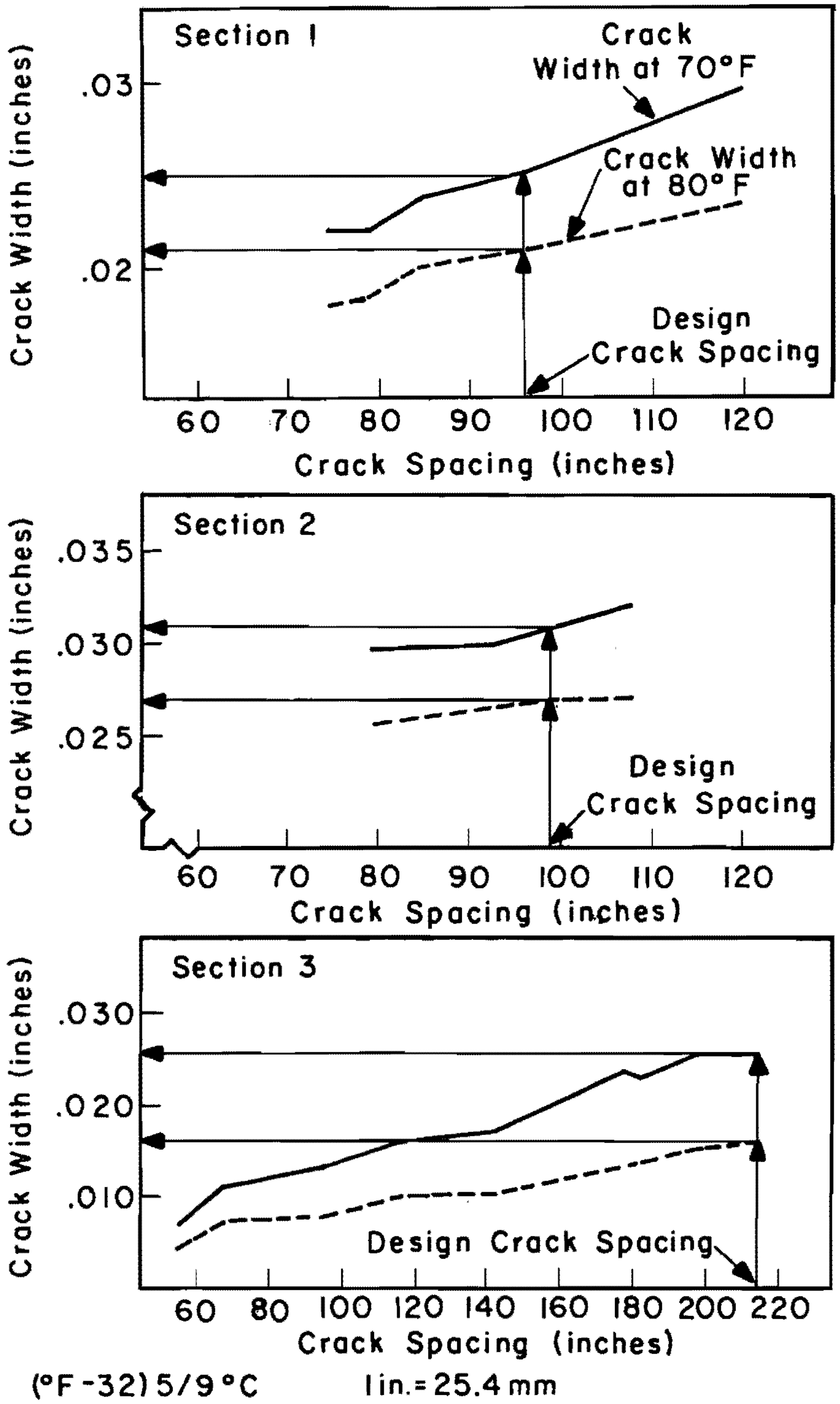


Fig I-4.3. Relation between crack spacing and concrete movement at a crack for temperature change and specific location.

TABLE I-4.14. HORIZONTAL AND VERTICAL CHARACTERIZATION DATA

Variable	Value		
	Section 1	Section 2	Section 3
Horizontal Characterization			
Mean high temperature, ^{o}F	80.0	80.0	80.0
Joint width at high temperature, in.	.021	.027	.016
Mean low temperature, ^{o}F	70.0	70.0	70.0
Joint width at low temperature, in.	.025	.031	.026
Vertical Characterization			
Design load transfer, %/100	.92	.83	.87
Minimum temperature observed, ^{o}F	13.0	13.0	13.0

($^{\circ}$F-32) x 5/9 = $^{\circ}$C

1 in. = 25.4 mm

TABLE I-4.15. SUMMARY OF OVERLAY PROPERTIES

Variable	Value		
	Section 1	Section 2	Section 3
Creep modulus, psi	250,000	250,000	250,000
Dynamic modulus, psi	400,000	400,000	400,000
Thickness, in.	8.5, 9.0	8.0, 8.5	9.0, 9.5
Density, pcf	136	136	136
Thermal coefficient, in./in./°F	.000012	.000012	.000012
Bonding stress, psi	500	500	500

1 psi = 6.894 KPa
1 in. = 25.4 mm
1 pcf = 16.01 Kgm^{-3}
1 in./in./°F = 1.8 mm/mm/°C

and assuming a slab temperature after placement of the overlay of 110°F (43.3°C). Design temperature values determined for this example were 85°F (47°C) for the existing pavement and 97°F (54°C) for the overlay.

A design load weight of 18-kip (80-kN) and a load width of 28 inches (711-mm) were assumed for this design example.

Reflection Cracking Evaluation

Table I-4.16 shows coded RFLCR1 input data for design Section 1. By entering "PART" on card 13, it was possible to use the short form (card 14) and change the overlay thickness from 8.5 to 9.0 inches. After the last problem the calculations were terminated by specifying "STOP" (see card 15). Input for Sections 2 and 3 were coded in a similar way. RFLCR1 output are included in Appendix 1. Reflection cracking input variables listed on the output include existing pavement properties, horizontal and vertical existing pavement characterization data, overlay properties, bondbreaker width, design temperature changes, and design load specifications. The RFLCR1 results consist of restraint coefficients (beta values), slope of friction curve, existing pavement stresses, and overlay strains.

A summary of the reflection cracking subsystem results is given in Table I-4.17. Also included on the same table are the allowable values for tensile and shear strains. These allowable values were determined according to the procedures outlined in Part III. The maximum allowable shear strain was determined using Fig III-4.12 in the User's Manual. From the information on Table I-4.17, it can be concluded that reflection cracking is not likely to occur in the asphaltic concrete overlays for Sections 1 and 2. Although the tensile strain exceeds the allowable slightly in Section 3, it was decided not to increase the overlay thickness in view of the limited funds available. The risk associated with that design will be rather small and the occasional reflection crack that might occur will have to be maintained in the future.

In summary, it can be concluded that all the designs in Table I-4.9 are feasible designs. Because handling traffic would be extremely difficult for a CRCP overlay for which the full width of the road would have to be closed to traffic for a considerable time, it was decided to construct an asphaltic concrete overlay. Since an increase of only 5 percent in the

TABLE I-4.16. RFLCR1 INPUT DATA FOR SECTION 1

```
ILLUSTRATIVE OVERLAY DESIGN PROBLEM.
ASPHALTIC CONCRETE OVERLAY ON CRCP PAVEMENT, SECTION 1.
CRCP        4200000..000006      8.     140.         8.0      .02
STEEL      29000000..000006      .48      3.19      295.
HORZ CHAR   80.      .021       70.      .025       13.
VERT CHAR  .023        .92
  AC         250000..000012      8.5       136.    .3         400000.    500.
OVETYP
BOND BREAK    2.0
INTER LAYR
DESIGN       85.                97.        18000.    28.
SLOPE SWCHO.
PART
   250000.   400000.     9.0     2.0      85.0                97.0           0.0
STOP
```

TABLE I-4.17. SUMMARY OF REFLECTION CRACKING ANALYSIS RESULTS

Design Section	Design Life, x 10^6 18-kip ESAL	Overlay Thickness, inches	Horizontal Tensile Strain, x 10^{-3} in./in.		Vertical Shear Strain, x 10^{-6} in./in.	
			Predicted	Allowable	Predicted	Allowable
1	7	8.5	1.81	2.00	.39	4.72
	10	9.0	1.79	2.00	.37	4.43
2	7	8.0	1.83	2.00	.89	4.72
	10	8.5	1.81	2.00	.84	4.43
3	7	9.0	2.11	2.00	.60	4.72
	10	9.5	2.09	2.00	.57	4.43

1 in. = 25.4 mm
1 in./in. = 1 mm/mm
18-kip = 80-KN

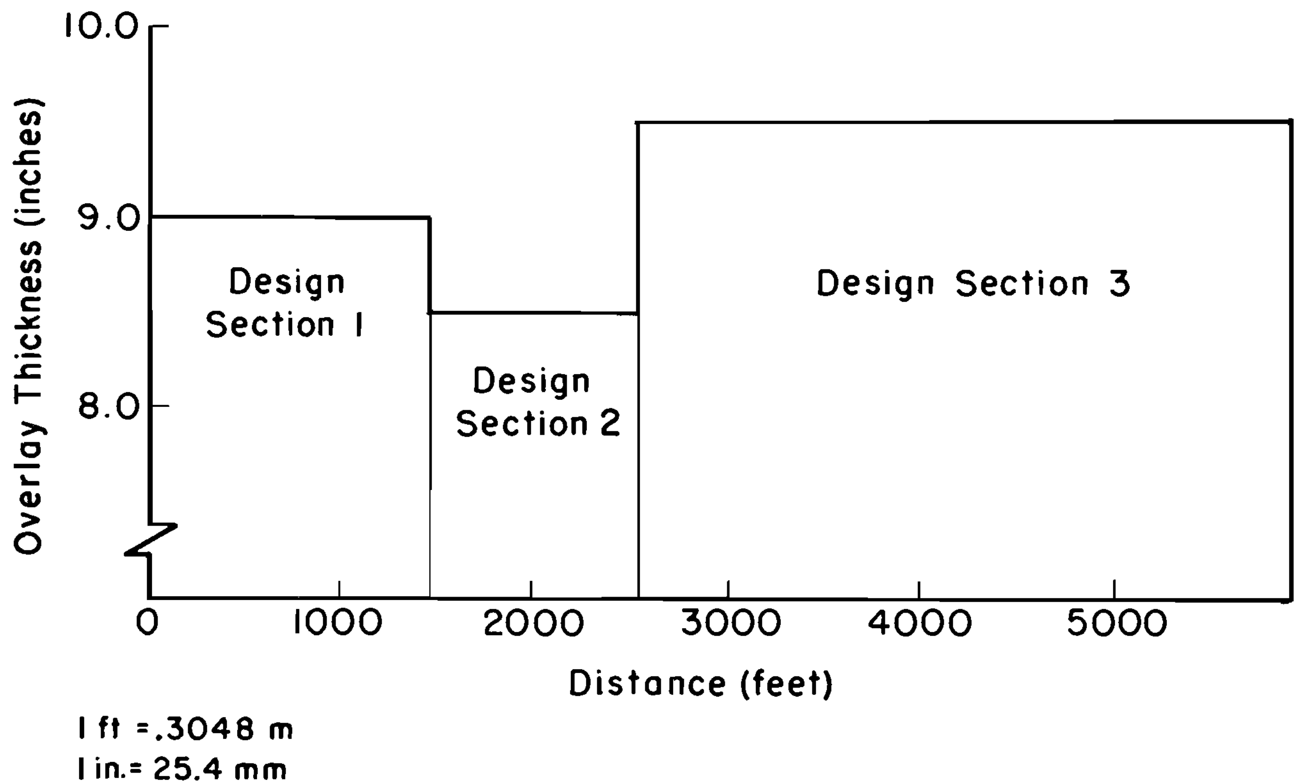


Fig I-4.4. Illustrative overlay design problem: final asphaltic concrete overlay thickness design.

overlay thickness would increase the design life from 7 million to 10 million 18-kip (80-kN) ESAL, it was decided to use the 10 million 18-kip (80 kN) ESAL design life.

The final overlay design for this example is as indicated on Fig I-4.4.

CHAPTER I-5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The Texas SDHPT rigid pavement overlay design procedure presented here is a sound method that basically consists of fatigue cracking and reflection cracking subsystems. The fatigue cracking subsystem considers the remaining life of the existing pavement, using fatigue principles, and determines the required overlay thickness for a specific design life. Miner's linear damage hypothesis is used in this process. Practically all portland cement concrete and asphaltic concrete overlays can be designed on various types and conditions of existing pavements. The reflection cracking subsystem provides a rational means for analyzing an overlay for the possible occurence of reflection cracking. Four computer programs are being used in the Texas SDHPT procedure. They are PLOT2, TVAL2, RPOD2 and RFLCR1.

This procedure was developed by adapting, through evaluation, modification, improvement, and simplification, the recently developed FHWA rigid pavement overlay design method for the Texas SDHPT. The revisions made to the FHWA procedure are briefly discussed in this part of the report and are discussed in detail in Part II. They include modifications to

(1) the RPOD1 program to generate RPOD2,

(2) the input guides for the four computer programs, and

(3) the materials characterization procedures.

The use of the Texas procedure is illustrated by means of an example design problem in Part I.

In conclusion, it can be said that previously the Texas SDHPT did not have a generally accepted method for design of structural overlays on rigid pavements. This procedure provides a means to design practically all kinds of overlays on rigid pavements in a rational way. The User's Manual presented in Part III is intended for Texas SDHPT use and will enable the average design engineer to use this design method.

Recommendations

Field verification of any new design procedure is a necessary but time-consuming process. It is, therefore, recommended that this procedure be implemented for trial use on real design problems as soon as possible. This can be done by designing a number of overlay sections for construction in conjunction with SDHPT personnel. The actual performance of these overlay sections should then be monitored.

Since this method is computerized and very adaptable to the rigid pavement management system (RPS), it should be incorporated in RPS, because the overlay design models outlined herein appear to be better than those presently used in RPS.

This overlay design method can also be a useful research tool and can be used for such studies as determining the most economical time to overlay pavements, the investigation of new overlay materials, and the evaluation of methods to prevent reflection cracking.

Eventually it is hoped that this overlay procedure will provide pavement designers with a sound practical method of designing overlays on rigid pavements rationally.

Any future relevant research findings, such as more information on the friction curve between concrete pavements and subbases, methods for obtaining values for stress sensitivity of soils by indirect means, and improved materials characterization procedures, should be used to update this design procedure.

PART II

DEVELOPMENT OF THE TEXAS SDHPT RIGID PAVEMENT OVERLAY DESIGN PROCEDURE

CHAPTER II-1. INTRODUCTION

As pointed out in Part I of this report, the Texas SDHPT rigid pavement overlay design procedure is based on the newly developed FHWA method. In the process of adapting the method for use by the Texas SDHPT, the FHWA method has been thoroughly evaluated and modifications have been made where needed.

The basic procedure, as indicated on Fig I-2.1 is considered to be excellent and no changes were required. Evaluation studies have been concentrated on the fatigue cracking and the reflection cracking subsystems and some modifications have been made to these, as discussed herein.

This part of the report deals with the evaluation and development studies. Evaluation of and modification to the fatigue cracking subsystem are discussed in Chapter II-2. Chapter II-3 contains an evaluation of the RFLCR1 program by means of a limited sensitivity analysis and also gives some recommendations on materials characterization. Chapter II-4 is a brief summary of the findings and recommendations for this part of the report.

The modifications discussed here are included in the Texas SDHPT procedure for overlays on rigid pavements outlined in Part I. Part III contains a detailed User's Manual for this overlay design procedure.

CHAPTER II-2. FATIGUE CRACKING ANALYSIS

In this chapter, the fatigue cracking analysis used in the FHWA method is thoroughly evaluated and areas in need of modification are determined. The revised computer program RPOD2 has been developed for use in the Texas procedure.

EVALUATION OF FATIGUE CRACKING ANALYSIS (RPOD1)

Some of the outstanding characteristics of the fatigue cracking analysis used in the FHWA method are:

(1) Deflections are used together with the results of laboratory testing to characterize the subgrade material.

(2) Deflections, stresses and strains are computed using linear elastic layered theory, and more specifically the ELSYM5 program.

(3) The remaining life of the existing pavement is taken into consideration in designing the overlay.

(4) The condition of the existing pavement is taken into account in the overlay design.

This method makes use of the most up-to-date theories and techniques in pavement design and can handle all kinds of combinations of existing pavement, overlay types, materials, voids, etc., as was pointed out in Part I of this report.

The principle of using layered theory for design of overlays on rigid pavements is not a very traditional one, but the work done by McCullough (Ref 5) indicated that "a computer oriented solution to layered theory is the most appropriate solution for overlay design . . ." McCullough also indicated that a comparison of layered theory and the generally accepted Westergaard theory used in design of portland cement concrete pavements gave a favorable correlation over the range of parameters to be expected in practice. The layered solutions were compared to Westergaard interior solutions.

Some advantages of using linear elastic layered theory instead of plate theory (Ref 5) are:

(1) A complete state of stress can be predicted so that subsurface layers can be rationally evaluated without applying empirical procedures.

(2) The subgrade material properties necessary for layered solutions are relatively simple to determine in the laboratory by means of the resilient modulus test. On the other hand, the modulus of subgrade reaction, or k-value, associated with plate theory cannot be measured in the laboratory and elaborate field tests are required.

It is also worthwhile to note that it is not possible to take the stress sensitivity of the subgrade support into account when using k-values and that the stress levels in the pavement are different under a 30-inch (762-mm) diameter plate loaded to, say, 10 psi (68.9 MPa) than under normal traffic design loads.

The method used in this procedure, to use both resilient modulus test results and deflection data in determining the subgrade resilient modulus under the design load, is an excellent approach to the problem.

The remaining life concept used here, which is based on Miner's linear damage hypotheses, has also been used by McCullough (Ref 5), as mentioned in Part I.

In order to evaluate this method, to adapt it to Texas needs, to be able to modify it and to improve it, various facets of the RPOD1 computer program have been studied and will be discussed here. Necessary and desirable changes to the program itself as well as the use of the program are also included in this section.

Sensitivity Analysis Conducted on RPOD1

An extensive sensitivity analysis has been conducted on the RPOD1 computer program as reported by Nayak et al. (Ref 7). Information from this sensitivity study has been used in the development of this Texas method.

It should be kept in mind that the results of a sensitivity analysis are affected greatly by the ranges selected for varying the input variables. As pointed out by Nayak et al. (Ref 7), the best way of selecting these ranges is to use the standard deviation, which is a measure of the variation of each variable about its arithmetic mean, as a basis for the sensitivity study. This is the method used by Nayak et al. (Ref 7), and the results can be used provided the following limitations are kept in mind.

(1) The design traffic used for the sensitivity study was 30 million equivalent 18-kip (80-kN) single axle loads in the design lane, which is extremely high and results in unrealistically great overall thicknesses. The effect of the various input variables relative to each other, should, however, not be affected by this.

(2) Benkelman beam deflections were used in that study, with the result that the design load was equal to the deflection load. As pointed out in Part I, it is only when these two loads differ from each other that the subgrade stress sensitivity will have an effect on the design. Thus the effect of the stress sensitivity of the subgrade could not be determined in that experiment.

(3) The way the range for the "laboratory data" was selected would result in approximately parallel laboratory curves which would represent materials with essentially the same stress sensitivity of their resilient moduli. Even if the deflection load were different from the design load, the effect of the stress sensitivity of the subgrade would not be detected this way.

(4) In selecting values for the base course modulus, Nayak et al. (Ref 7) found a high standard deviation for this variable and decided to reduce it using engineering judgement in the selection process. This turned out to be a very important variable for many of the existing pavement-overlay combinations investigated, as can be seen in Table II-2.1.

(5) In the case of Poisson's ratio, for all layers, engineering judgement has been used in selecting a standard deviation. Poisson's ratio also turned out to be important in many cases (See Table II-2.1).

Table II-2.1 is a summary of the rankings of the input variables to the RPOD1 program and was compiled from information obtained from the report by Nayak et al. (Ref 7). It will be noted that the results of both the fractional factorial sensitivity analysis and the single factorial experiment are included in this summary table. Although it is realized that relative rankings of design variables in single factorial experiments are not absolute rankings, that interactions are not taken into consideration, and that the effect of each variable is only estimated at one level of the other variables (Ref 7) in the one-factor-at-a-time experiment, it was considered

TABLE II-2.1. SUMMARY OF RANKING INPUT VARIABLES TO THE RPOD1 COMPUTER PROGRAM RELATIVE TO THEIR IMPORTANCE ON THE PREDICTED OVERLAY THICKNESS

Experiment Type	Fractional Factorial				Single Factorial				
Overlay Type	CRCP	JCP	ACP	CRCP	JCP	CRCP	JCP	JCP	JCP
Existing Pavement Type	CRCP	JCP	CRCP	CRCP	CRCP	JCP	JCP	CRCP	JCP
Bonding Condition	Bonded	Unbonded	Bonded	Unbonded	Unbonded	Unbonded	Unbonded	Unbonded	Unbonded
Voids	None	None	None	None	None	None	None	Voids	Voids
Cracks	Class 1 & 2	Class 1 & 2	Class 1 & 2	Mechanically Broken	Class 1 & 2	Class 1 & 2	Class 3 & 4	Class 1 & 2	Class 1 & 2
Input Variable									
Modulus of Subbase	1	2	2	4	1	6	--	2	5
Design Deflection	2	1	1	2	2	5	1	1	2
Thickness of Surface	3	--	3	--	5	--	--	5	--
Modulus of Surface	4	4	--	--	6	1	2	6	1
Thickness of Subbase	5	--	4	--	3	--	--	3	--
Poisson's Ratio of Surface	6	--	--	--	4	--	--	4	--
Modulus of Subbase x Design Deflection	--	3	--	6	--	--	--	--	--
Poisson's Ratio of Overlay	--	5	--	3	--	4	4	--	3
Modulus of Overlay	--	6	--	1	--	2	3	--	4
Modulus of Surface x Thickness of Subbase	--	--	5	--	--	--	--	--	--
Modulus of Subbase x Thickness of Base	--	--	6	--	--	--	--	--	--
Modulus of Bondbreaker	--	--	--	5	--	3	5	--	6
Poisson's Ratio of Bondbreaker	--	--	--	--	6	--	--	--	--
Thickness of Bondbreaker	--	--	--	--	--	--	6	--	--

useful and appropriate to include this information in Table II-2.1. The table includes the 6 most important variables for the different experiments considered in that study.

In studying Table II-2.1 it will be noted that:

(1) In general, the modulus of the subbase turned out to be a very important variable. This might be somewhat surprising, but it should be kept in mind that one of the advantages of using layered theory is that factors outside the concrete slab can be taken into account more accurately. Although the selection of the standard deviation for this variable was based on engineering judgement, and selection of a smaller standard deviation could result in a lower ranking of this variable, it is felt that the standard deviation selected is reasonable and that the importance of the subbase modulus should not be overlooked. It is also worthwhile to note that the stress sensitivity of the subbase material, especially when unstabilized, can have an effect on the predicted overlay thickness. The present design method cannot take this into account, but future research should be directed toward considering the subbase stress sensitivity in the design procedure.

(2) The design deflection is another very important variable. It should be kept in mind that the design deflection is used to characterize the subgrade material, and, in this case, where stress dependency of the subgrade material did not come into play, it means that the subgrade support is important.

(3) Other variables that are important are the thickness and modulus of the surface layer, the modulus of the overlay, thickness of the subbase, and the modulus and thickness of the bondbreaker.

(4) The Poisson's ratios of the overlay, surface layer, and bondbreaker turned out to be important in some instances. Here again engineering judgement has been used in establishing a value for the standard deviation of Poisson's ratio (Ref 7). It is pointed out by Kennedy et al. (Ref 12), the source of information used by Nayak et al. to determine standard deviations for Poisson's ratios that the large variation in Poisson's ratio for each project is possibly due to the fact that Poisson's ratio is very sensitive to small errors in deformation measurements. The FHWA method (Ref 6) suggests the use of default values for Poisson's ratio as an alternative to laboratory determination. This is feasible, and in fact it may be better to use well

determined fixed values than values determined for a specific project with a very limited amount of testing.

(5) It should be noted that Nayak et al. (Ref 7) used a correlation between concrete flexural strength and modulus of elasticity, and varied these two input variables together. This is realistic, since normally an increase in modulus will be accompanied by an increase in flexural strength (Refs 1 and 13). The effect of the "concrete modulus" in the sensitivity analysis, therefore, represents the combined effect of both variables.

(6) The number of 18-kip (80-kN) equivalent single axle applications prior to overlay has no effect on overlays for pavements with class 3 and 4 cracking or mechanically broken up pavements in the FHWA method but has a direct effect on pavements with remaining life. Considering the accuracy of predicting future, and in this case, past, traffic loads, a standard deviation of 0.5 million 18-kip (80-kN) equivalent single applications used in the sensitivity analysis seems low. It was also noted, by studying the single factorial experiments, that, in the case of the JCP existing pavements, the traffic prior to overlay did not have an effect, which indicates that the pavement probably did not have any remaining life in those cases. The effect of the remaining life on the predicted overlay thickness needs some study and is discussed later on in this chapter.

In summary it can be said that, according to the sensitivity analysis conducted on the RPOD1 computer program by Nayak et al. (Ref 7), as discussed in this section:

(1) The design deflection, elastic moduli, and thicknesses of the different layers seem to be the most important input variables.

(2) The stress sensitivity of the subgrade resilient modulus, as well as the effect of the value of the subgrade resilient modulus, has not been considered in that sensitivity analysis and will be investigated in this chapter.

(3) The effect of remaining life on overlay thickness warrants investigation.

(4) Poisson's ratios for the different materials should be fixed for general use rather than having to determine them for each individual project.

Comparison of RPOD1 Response With Thickness Computed Manually Using ELSYM5 For Calculation of Stresses and Deflections

The purpose of this comparison was to gain confidence in the computer program and compare its results to those obtained from manual calculations. The pavement structure used in this analysis, as can be seen in Fig II-2.1, was a CRCP overlay on a CRCP existing pavement with no voids and capable of carrying tensile stresses.

Determination of the Subgrade Resilient Modulus. Using the ELSYM5 program and analyzing the existing pavement structure with a Dynaflect load, the relationship of subgrade resilient modulus versus surface deflection and deviator stress at the top of the subgrade has been determined, as shown in Fig II-2.2. Using the design Dynaflect deflection of 0.56×10^{-3} inch (.014-mm), a value for resilient modulus and a corresponding deviator stress value could be determined for the subgrade material. These values have been plotted as point "X" on Fig II-2.3. "Adjusted laboratory curves" have been constructed through point X. Slopes of the log resilient modulus versus log deviator stress relationship (S_{SG}) of 0, -0.3, -0.6, -0.9 and -1.3 have been used in this analysis. These values represent ranges in lab data. The ELSYM5 calculated relationship between resilient modulus and deviator stress at the top of the subgrade, resulting from the design load on the existing pavement structure, is also indicated on Fig II-2.3. Design resilient moduli for different values of S_{SG} has been determined from these plots and are listed in Table II-2.2. The same pavements have been analyzed with RPOD1, and the RPOD1 selected values for resilient modulus are also indicated in Table II-2.2.

These values compare well considering that graphical solutions and iterations have been involved.

Remaining Life of the Existing Pavement. The remaining life of the existing pavement has been determined using the maximum horizontal tensile stress at the bottom of the existing pavement (prior to overlay) and applying a stress factor of 1.2 to adjust this stress for an edge stress condition (See Table I-3.2). Using this stress in the fatigue equation, the original life of the existing pavement has been determined. The fatigue equation used in the RPOD1 program is given in Part I (Eq I-3.3).

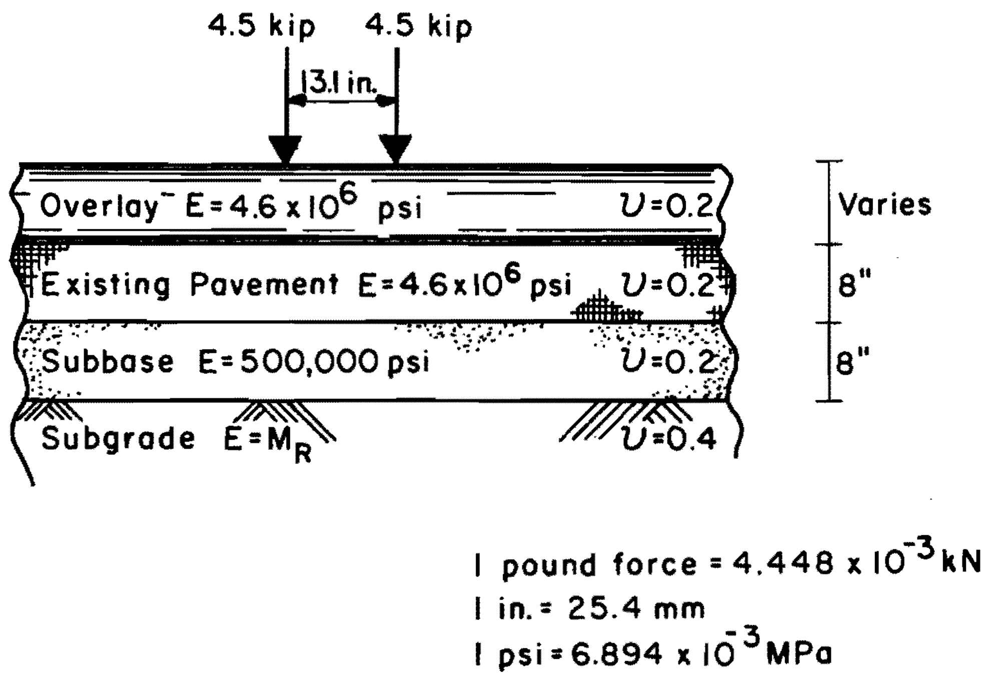


Fig II-2.1. Pavement structure used in comparison study.

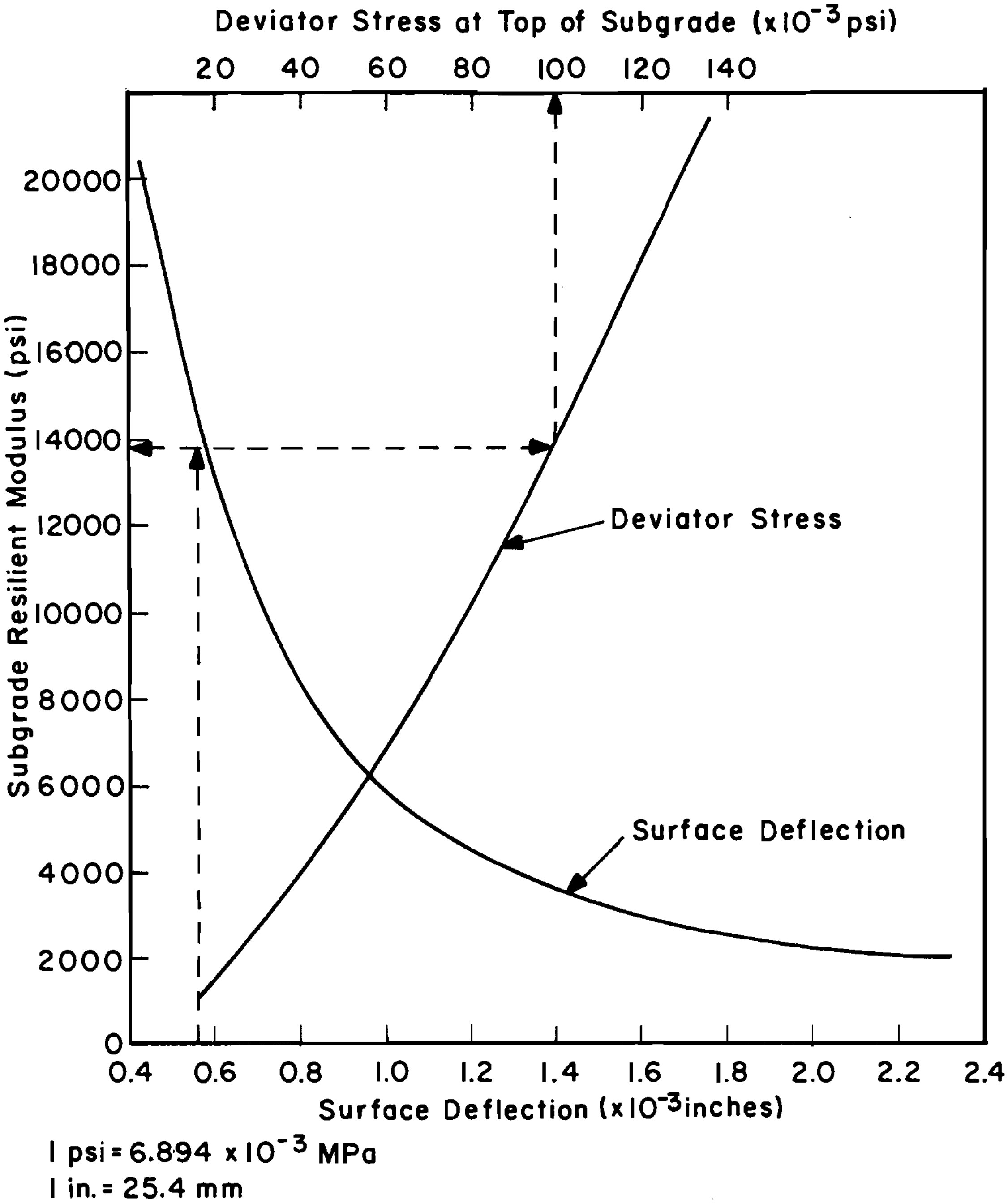


Fig II-2.2. Calculated relations of subgrade resilient modulus versus surface deflection and deviator stress at the top of the subgrade under dynaflect load.

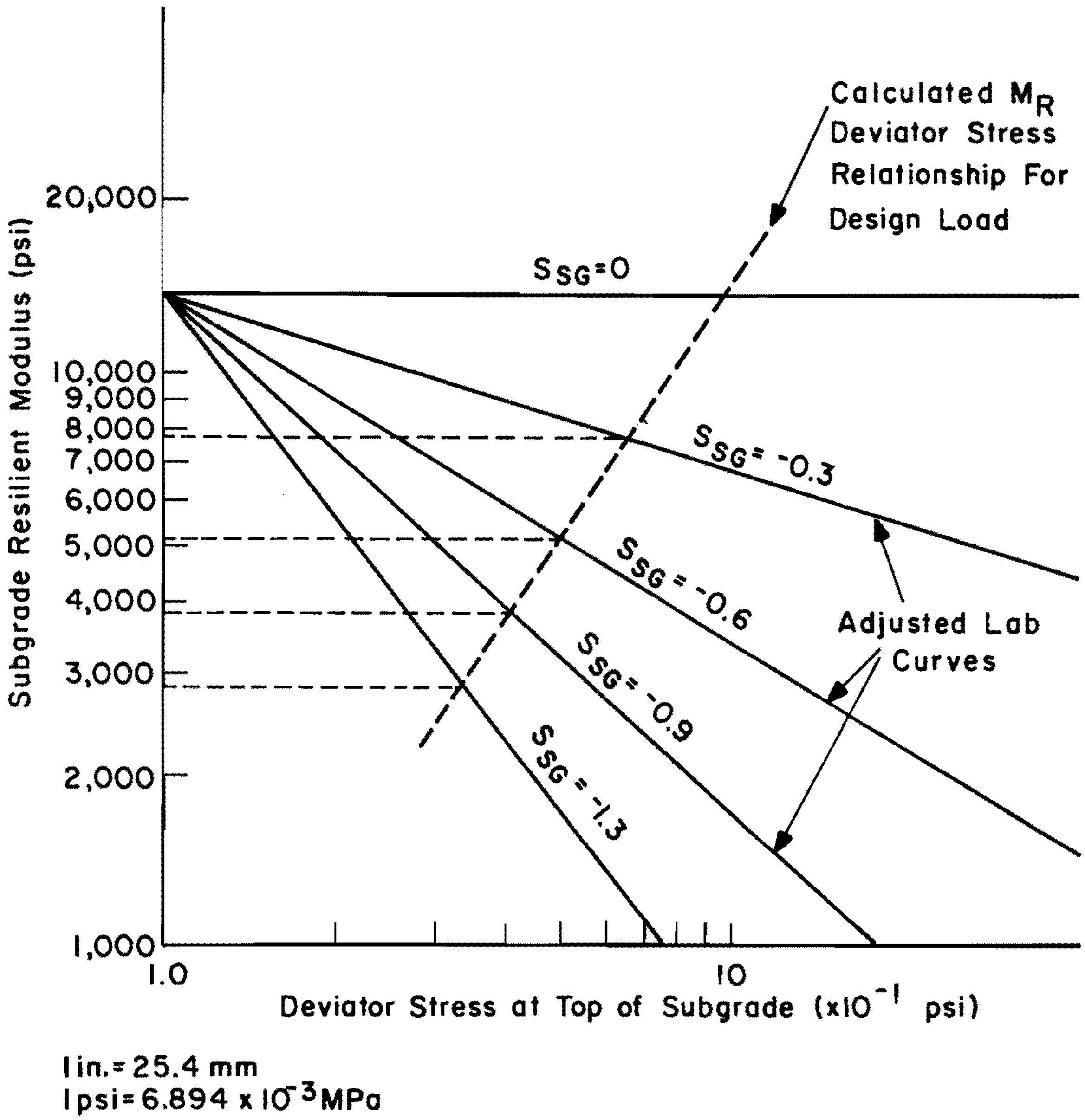


Fig II-2.3. Determination of design subgrade resilient modulus for various resilient modulus versus deviator stress relationships.

TABLE II-2.2. COMPARISON OF VALUES OF SUBGRADE RESILIENT MODULUS FOR DIFFERENT VALUES OF S_{SG} USED BY THE TWO METHODS

S_{SG}	M_R Calculated Using ELSYM5, psi	M_R Used By RPOD1, psi
-1.3	2850	2840
-0.9	3850	3820
-0.6	5170	5173
-0.3	7750	7743
0	13700	13646

1 psi = 6.894×10^{-3} MPa

Traffic prior to overlay has been taken as 4 million 18-kip (80-kN) equivalent single axle loads for calculating the remaining life of the existing pavement. A comparison of these results with the remaining life predictions out of RPOD1 can be seen on Fig II-2.4. It can be seen that the results compare fairly well.

Check on Design Life. Designs were based on a design life of the overlay of 7 million 18-kip (80-kN) equivalent single axle loads.

Using the overlays as predicted by RPOD1, the design life has been checked manually and is indicated in Table II-2.3.

Discussion. Out of this manual check on RPOD1 it can be seen that, taking into account that iteration, interpolation, and, in some instances, even extrapolation are used in RPOD1 and that the manual method involved graphical solutions, the RPOD1 program seems to perform its function well.

Effect Of Load Configuration On Rigid and Flexible Pavements

Design loads applied to the pavement structure in the RPOD1 program are two 4.5-kip (20-kN) loads at a distance of 13.11 inches (333-mm) apart, representing one half of an 18-kip (80-kN) single axle with dual wheels. Because of concern that the dual wheels on the other half might still have an effect on a rigid pavement, with a large deflection basin, the effect of wheel configuration was studied for a rigid and a flexible pavement structure.

Four possible wheel configurations, to represent an 18-kip (80-kN) equivalent single axle load on a pavement, have been considered in this study - see Fig II-2.5. The four possibilities are:

(1) four 4.5-kip (20-kN) loads at positions indicated in the figure,

(2) two 9-kip (40-kN) loads, one on each half of the axle,

(3) one 9-kip (40-kN) load, (half axle only), and

(4) two 4.5-kip (20-kN) loads at a distance 13.1 inches (333-mm) apart.

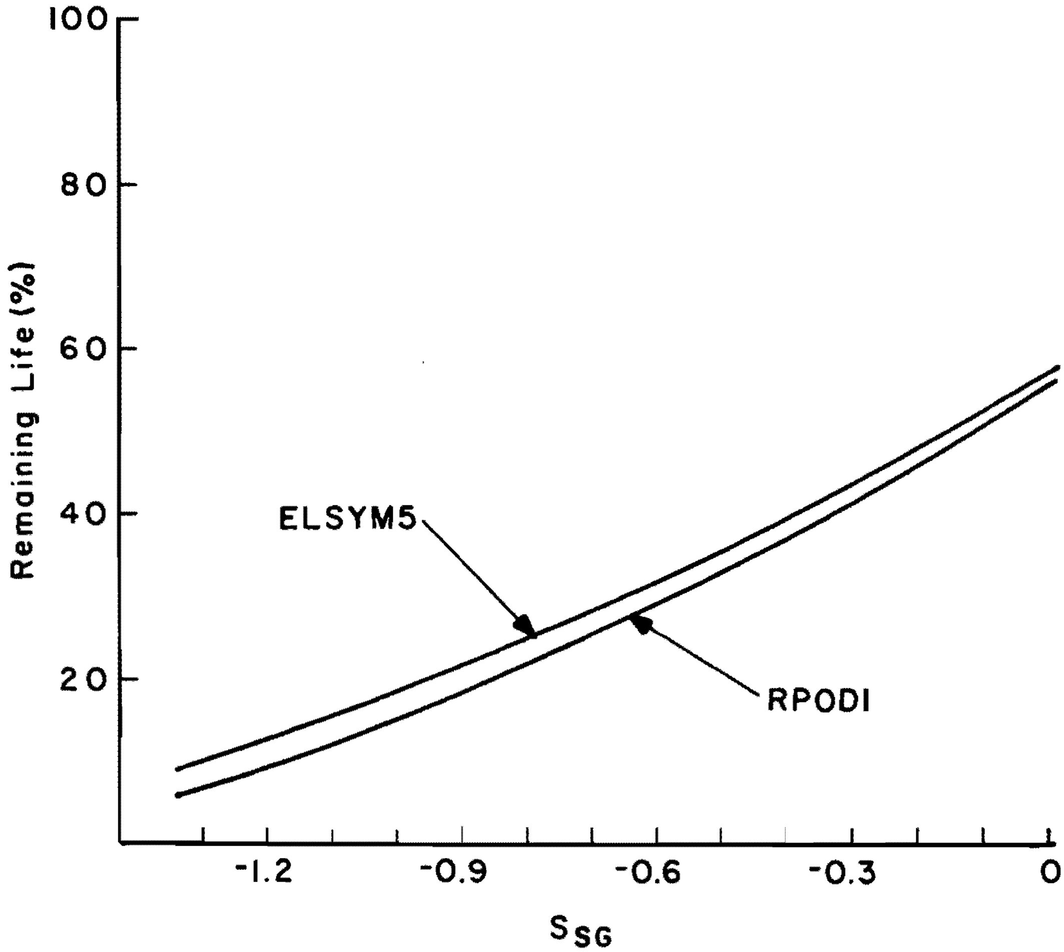


Fig II-2.4. Relationship between predicted remaining life and the stress sensitivity of the subgrade resilient modulus.

TABLE II-2.3. CALCULATED FATIGUE LIVES FOR OVERLAYS PREDICTED BY RPOD1

S_{SG}	Predicted Overlay Thickness, in.	Horizontal Stress At Bottom of Existing Layer, psi	Remaining Life, %	Fatigue Life (x 10^6), 18-k EAL
-0.9	6.5	51.7	21.4	7.29
-0.6	4.5	57.3	31.9	7.84
-0.3	*2.7	62.9	43.7	7.94
0	*1.5	65.6	57.1	9.06

*Warning signalled by RPOD1 that these values were obtained by extrapolation.

1 in. = 25.4 mm, 1 psi = 6.894 x 10^{-3} MPa, 18 kip = 80 kN

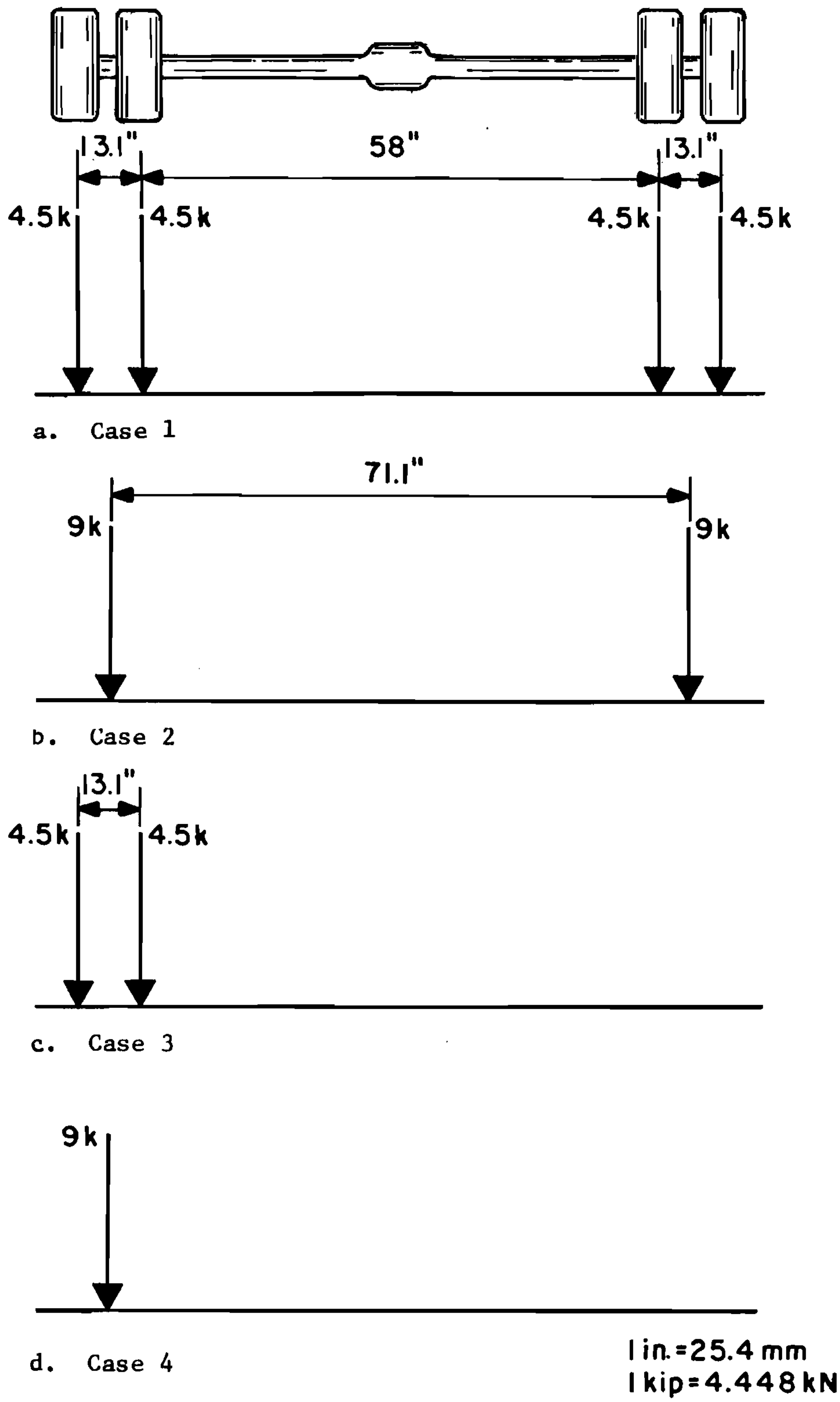


Fig II-2.5. Load configurations used in this study.

For Cases 3 and 4, the assumption is that the loads on the other half of the axle have a negligible effect on the resulting stresses, strains, and deflections.

Two pavement structures were considered in this study as indicated on Fig II-2.6. They are

(1) a rigid pavement with an overlay and

(2) a flexible pavement.

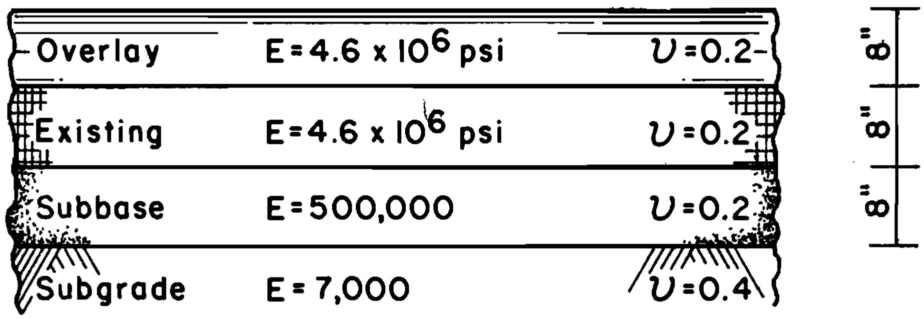
The ELSYM5 computer program was used to determine stresses, strains, and deflections in the structures under the different loading conditions. The results are summarized in Table II-2.4.

This matter has been investigated further by changing the loads inside the RPOD1 program to four 4.5-kip (20-kN) loads (Case 1) and also to two 9-kip (40-kN) loads (Case 2) and running the program for various pavement structures and overlay types. These results are shown in Table II-2.5.

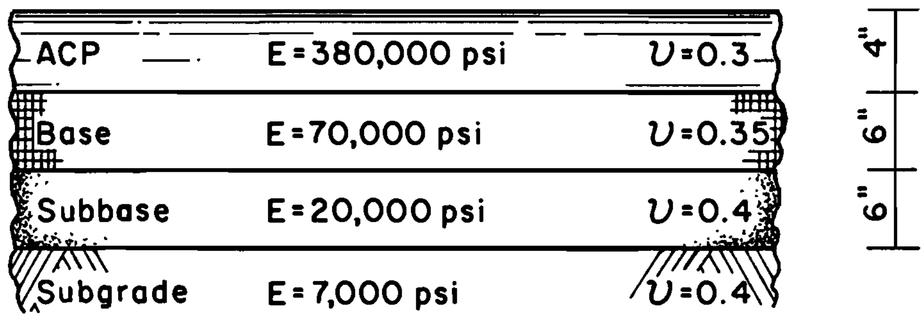
Discussion. Table II-2.4 indicates that, for rigid pavements, the deflection basin can be so large that it would be better not to neglect the influence of the two loads on the far side of the axle. Substitution of Case 1 loading with two 9-kip (80-kN) loads, 71.1 inches (1.806-mm) apart seems to be better. For the flexible pavement, substitution of Case 1 loading with two 4.5-kip (20-kN) loads (Case 3 loading) seems reasonable except for deflection predictions.

Table II-2.5 indicates clearly that the load configuration can have a large effect on the predicted pavement thickness, especially for pavements with remaining life. The reason for this is that the fatigue equation is used twice: to determine the percentage of remaining life and to predict the fatigue life of the pavement-overlay system. The fatigue curve used in RPOD1 has, however, been derived using the two 4.5-kip (20-kN) load configuration (Case 3), which are the same loads used as design loads in the program. It is therefore believed that this combination of design load and fatigue curve would result in adequate accuracy. This load configuration has been maintained for RPOD2.

In the process of characterizing the subgrade by means of deflections, the fatigue curve is not used, and, therefore, it would be essential to



a. Rigid pavement



b. Flexible pavement

1 in.= 25.4 mm
1 psi = 6.894 x 10^{-3} MPa

Fig II-2.6. Pavement structures used in load configuration study.

TABLE II-2.4. SUMMARY OF RESULTS OF LOAD CONFIGURATION STUDY

Loading / Maximum Value of Response / ELSYM5 Response	CASE 1	CASE 2		CASE 3		CASE 4	
	Max. Response	Max. Response	Percent of Case 1	Max. Response	Percent of Case 1	Max. Response	Percent of Case 1
Rigid Pavement							
Surface deflection ($x10^{-3}$), inches	9.49	9.51	0	5.62	-41	5.64	-41
Stresses: Hor. bottom layer 1, psi	-2.06	-1.30		1.26		-0.40	
Hor. bottom layer 2, psi	50.28	53.2	+6	40.80	-19	44.35	-12
Hor. bottom layer 3, psi	9.46	9.58	+1	7.24	-23	7.61	-20
Vert. top layer 4, psi	-0.45	-0.47	+4	-0.33	-27	-0.35	-22
Strains: Hor. bottom layer 1 ($x10^{-6}$)	0.84	1.27		0.94		1.30	
Hor. bottom layer 2 ($x10^{-6}$)	9.37	9.71	+4	7.40	-21	7.83	-16
Hor. bottom Layer 3 ($x10^{-6}$)	16.19	16.18	0	11.90	-26	12.31	-24
Flexible Pavement							
Surface deflection ($x10^{-2}$) inches	2.92	3.14	+8	2.35	-20	2.63	-10
Stresses: Hor. bottom layer 1, psi	76.55	86.22	+13	77.13	+1	86.74	+13
Hor. bottom layer 2, psi	22.81	27.89	+22	22.76	0	27.85	+22
Hor. bottom layer 3, psi	5.82	6.56	+13	5.81	0	6.55	+13
Vert. top layer 4, psi	-4.12	-4.67	+13	-4.07	-1	-4.62	+12
Strains: Hor. bottom layer 1	1.77	1.90	+7	1.78	+1	1.91	+8
Hor. bottom layer 2	2.86	3.17	+11	2.80	-2	3.11	+9
Hor. bottom layer 3	2.85	3.01	+7	2.73	-3	2.90	+3

1 in. = 25.4 mm, 1 psi = 6.894×10^{-3} MPa

TABLE II-2.5. COMPARISON OF RPOD1 PREDICTED OVERLAY THICKNESSES AND FATIGUE LIVES, FOR DIFFERENT LOADING CONFIGURATIONS ON VARIOUS RIGID PAVEMENT STRUCTURES

Overlay* Type	*Existing Pavement		** Subbase Type	Loads, kips	Remaining Life, %	Overlay Thickness (inches) for $N_{18}=7x10^6$	$N_{18}(x10^6)$ for Overlay Thickness			
	Type	Thickness, in.					3 in.	6 in.	9 in.	12 in.
CRCP	CRCP	8"	STAB	2x4.5	60.7	1.3	16.58	41.30	95.18	203.81
				4x4.5	37.4	3.9	5.44	11.98	24.97	49.08
				2x9.0	17.9	7.7	2.11	4.66	9.48	18.23
				2x4.5	None	5.2	2.53	9.82	32.82	84.86
				4x4.5	None	5.2	3.03	9.39	24.92	50.75
JCP	JCP	9"	STAB	2x4.5	84.8	1.4	14.69	35.78	80.78	169.21
				4x4.5	73.8	3.3	6.37	13.64	25.89	47.44
				2x4.5	None	6.7	1.30	5.17	16.97	44.79
				4x4.5	None	7.0	1.58	5.04	13.42	26.54
CRCP	CRCP	8"	GRAN	2x4.5	53.4	3.8	4.95	15.83	42.48	101.29
				4x4.5	30.7	7.4	1.62	4.56	11.09	23.54
				2x4.5	None	7.0	1.37	4.61	15.52	44.00
				4x4.5	None	7.7	1.43	3.89	10.76	25.82
JCP	JCP	11"	NONE	2x4.5	64.9	3.3	6.35	18.07	44.95	100.54
				4x4.5	32.9	8.3	1.51	3.88	8.18	16.12
				2x4.5	None	8.5	0.86	2.64	8.23	22.34
				4x4.5	None	9.8	0.94	2.21	5.56	12.59

*Existing pavement and Overlay: $E = 4.6 \times 10^6$ $\nu = 0.15$

**Subbase: thickness = 8", $E_{STAB} = 500{,}000$ psi, $\nu_{STAB} = 0.2$, $E_{GRAN} = 70{,}000$ psi, $\nu_{GRAN} = 0.4$

specify all the loads of the deflection measuring device. If for example the Benkelman beam is used, Case 1 loading should be specified.

Effect Of the Concept Of Remaining Life

As pointed out earlier, the concept of using the remaining life of the existing pavement in designing overlays was introduced by McCullough in 1969 (Ref 5). The same concept is also being used in the Shell Method, as described by Claessen and Ditmarsch (Ref 14), for flexible pavements and also in the FHWA method for flexible pavements (Ref 15). Work done by Zaniewski (Ref 16) uses the same concept for flexible pavements. This concept is discussed in Part I and is formulated by Eq I-3.5.

The effect of the remaining life of the existing pavement on the predicted overlay thickness has been studied by varying for the pavement indicated on Fig II-2.1 the traffic prior to overlay and keeping everything else constant. This resulted in a varying amount of remaining life in the existing pavement. Using the RPOD1 program, a relationship between remaining life of the existing pavement and the required overlay thickness could be established. This is indicated on Fig II-2.7.

It will be noted that, in taking the remaining life of the existing pavement into consideration, the required thickness of overlay is reduced drastically (See Fig II-2.7). On the other hand, if the fact that some of the life of the existing pavement has been consumed by the traffic prior to overlay is not recognized, the resulting overlay thickness could be far too thin.

If the existing pavement has less than 12 percent remaining life, it is not designed for remaining life by the RPOD1 program. The reason for this is explained on Fig II-2.8. It can be seen that, for a remaining life less than RL_x, using the remaining life concept would result in a thicker overlay thickness than if the overlay had been designed as if the existing pavement had no remaining life. In the RPOD1 program, RL_x was chosen as 12 percent. For the Texas method, a modification has been made to the program to overcome this problem. This modification is discussed later in this chapter.

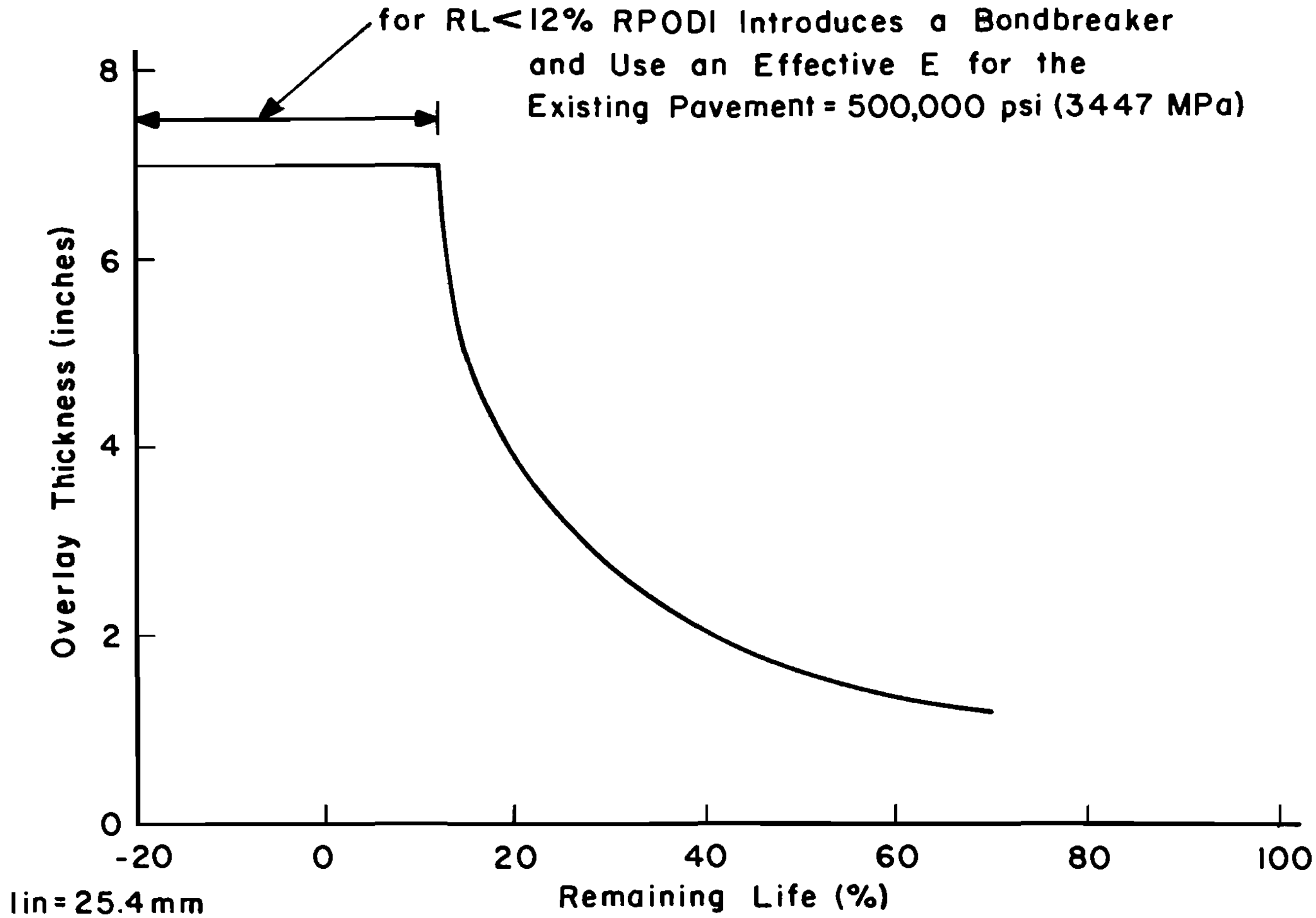


Fig II-2.7. Relationship between overlay thickness and remaining life of the existing pavement.

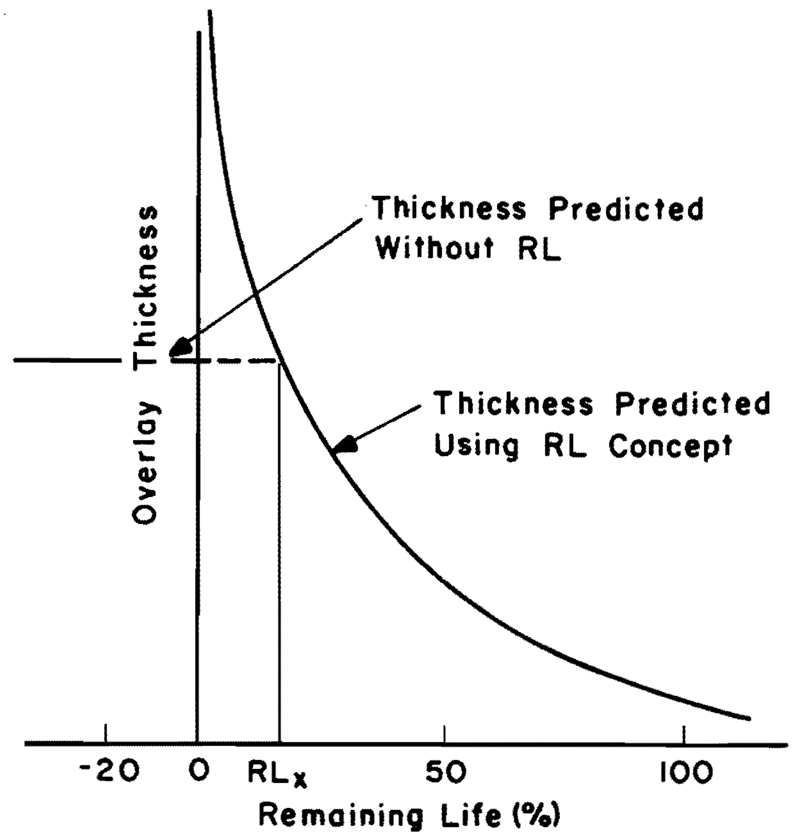


Fig II-2.8. Conceptual relation between overlay thickness and remaining life of existing pavement.

Effect Of Subgrade Resilient Modulus On Overlay Thickness

In this study, both manual calculations and RPOD1 calculations have been used to determine the effect of subgrade resilient modulus on overlay thickness.

For the manual calculations, the equation considering the remaining life of the existing pavement as presented in the Shell Method (Ref 14) has been used as follows:

$$N_{D2} = \frac{N_{D1} \cdot N_{A2}}{(N_{D1} - N_{A1})} \tag{II-2.1}$$

where

N_{D1} = Design life of existing pavement,

N_{A1} = Number of standard axles carried to date,

N_{A2} = Number of standard axles expected in the subsequent design period,

N_{D2} = New design number of standard axles (determined from fatigue equation with horizontal tensile stress at the bottom of the existing pavement after overlay).

The pavement structure considered here is the same as indicated on Fig II-2.1 with the exception of the use of an unbonded overlay. The stress relieving layer was considered to be 2 inches (50.8-mm) thick and to have an elastic modulus of 100,000 psi (689 MPa). Figure II-2.9 shows how the design life of the existing pavement (N_{D1}) increases with increase in subgrade resilient modulus. If the number of load applications prior to overlay (N_{A1}) is assumed to be constant, the remaining life of the existing pavement, which can be expressed as follows (See Eq I-3.5 and Eq II-2.1):

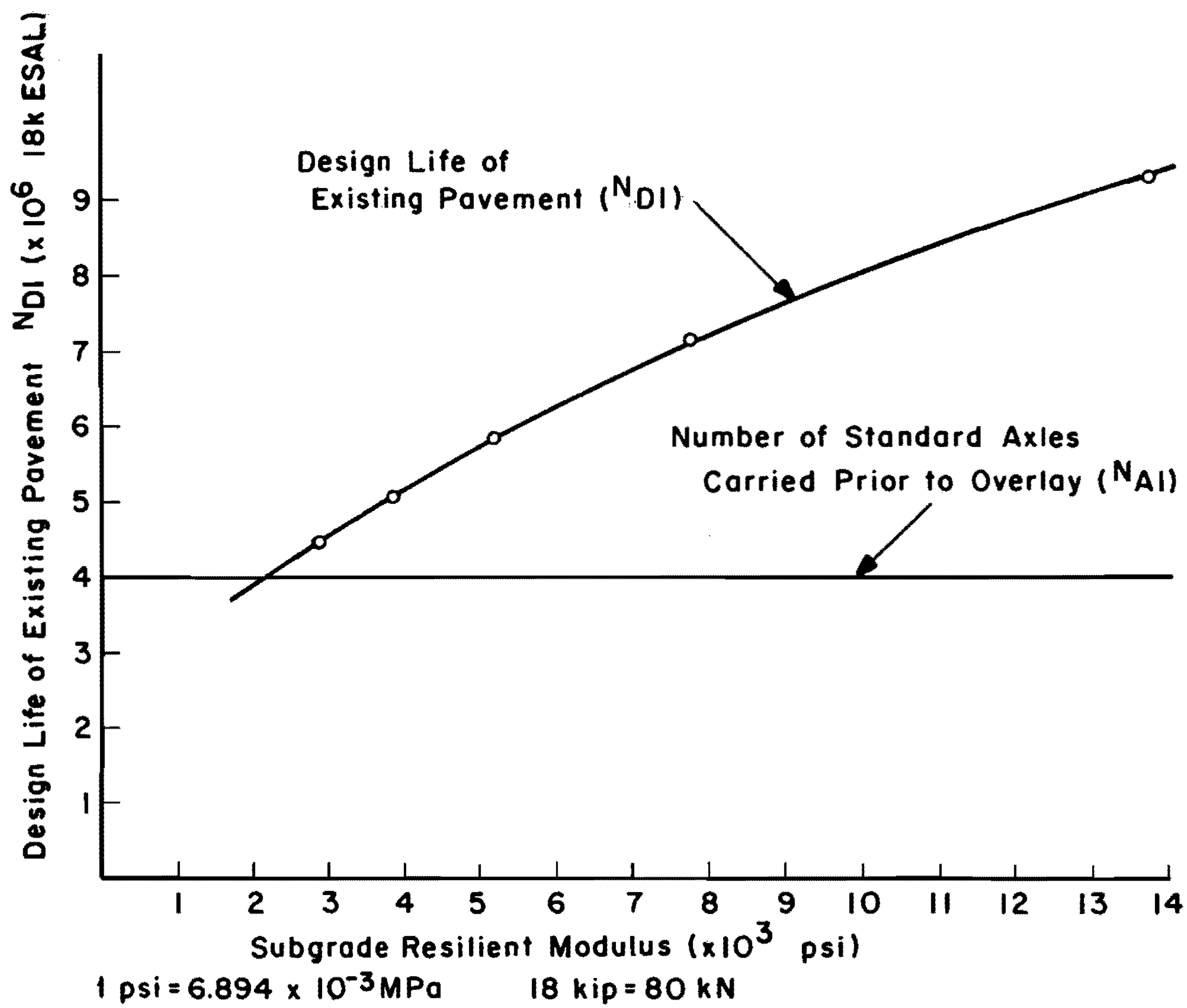


Fig II-2.9. Relation between design life of existing pavement and subgrade resilient modulus.

$$\text{Remaining life} = 1 - \frac{N_{A1}}{N_{D1}} \tag{II-2.2}$$

(All variables are as previously defined),

therefore increases as the subgrade resilient modulus increases. Figure II-2.10 indicates for different overlay thicknesses the relationship between number of standard axles determined from the fatigue equation, with the governing stress at the bottom of the existing pavement, after overlay (N_{D2}), and subgrade resilient modulus. By using both Figs II-2.9 and II-2.10 and Eq II-2.1, the relationship between overlay thickness and subgrade modulus was obtained and plotted on Fig II-2.11. RPOD1-predicted thicknesses for similar conditions are also plotted on the same graph.

Figure II-2.12 contains a set of curves, for the same structure as above, which relate required overlay thickness to subgrade resilient modulus for various fixed percentages of remaining life. The governing stress was considered to be at the bottom of the overlay (σ_1) for the no-remaining-life case (curve marked "RL = 0%"). For all other curves on this plot the governing stress was at the bottom of the existing pavement. RPOD1-predicted overlay thicknesses versus subgrade resilient modulus are plotted on the same figure as a dashed line.

Discussion. It can be seen on Fig II-2.11 that, especially for overlays thicker than 3 inches (76.2-mm), the RPOD1 results are in close agreement with manual calculations. In the RPOD1 program a relationship between overlay thickness and design life is determined, for thicknesses between 3 and 12 inches (76.2 and 306.8-mm) and the required overlay thickness for a specified design life is then obtained by interpolation. Below 3 inches (76.2-mm) the overlay thickness is obtained by extrapolation, which is probably the reason for the difference in results. Structural overlays that thin are, however, not recommended so that this difference has no practical implication.

It will be noted from Fig II-2.11 that varying the subgrade resilient modulus values from 3000 to 14,000 psi (20.7 to 96.5 MPa) has an effect of more than 6 inches (152.4-mm) on the required overlay thickness for an

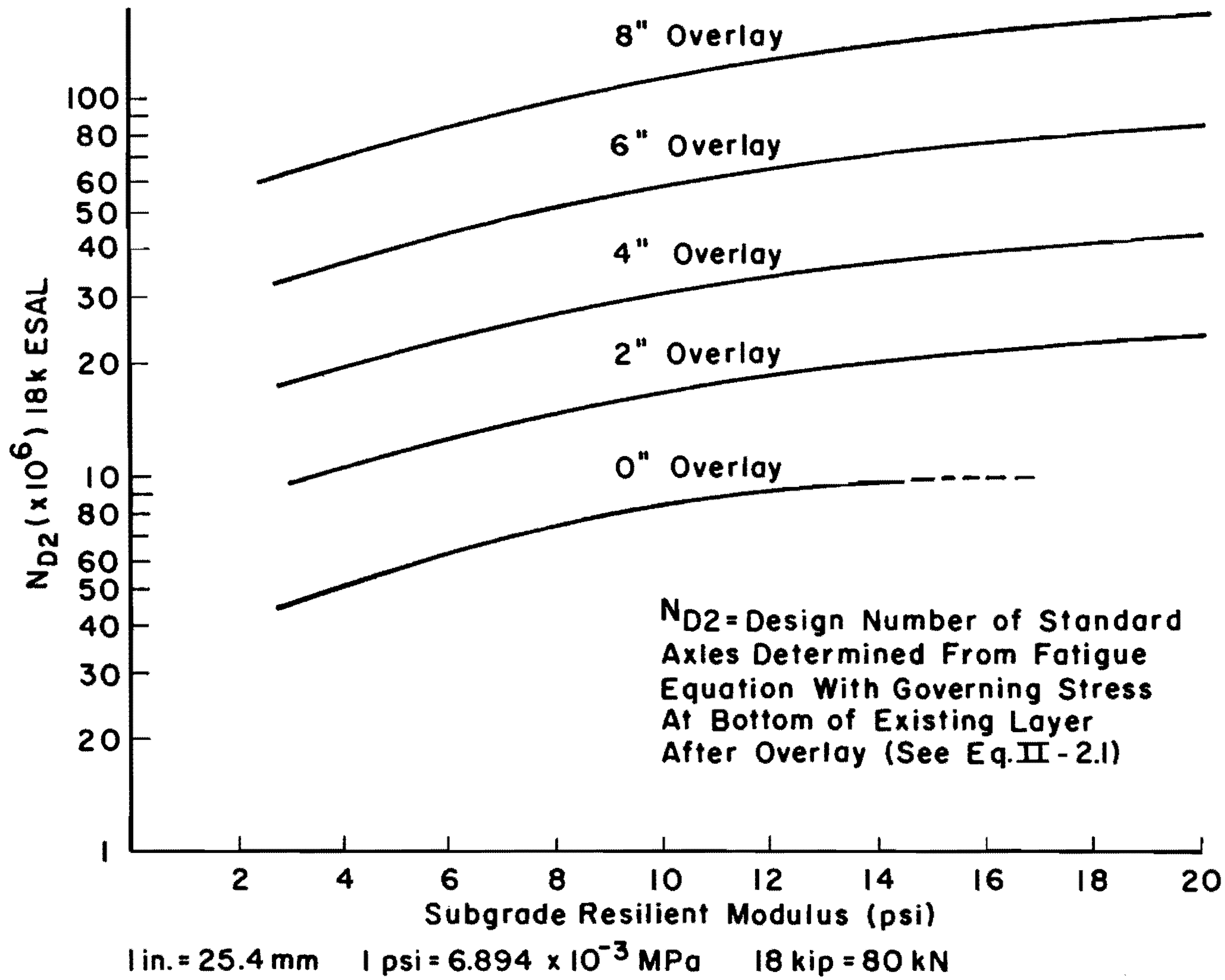


Fig II-2.10. Relationship between N_{D2} and subgrade resilient modulus.

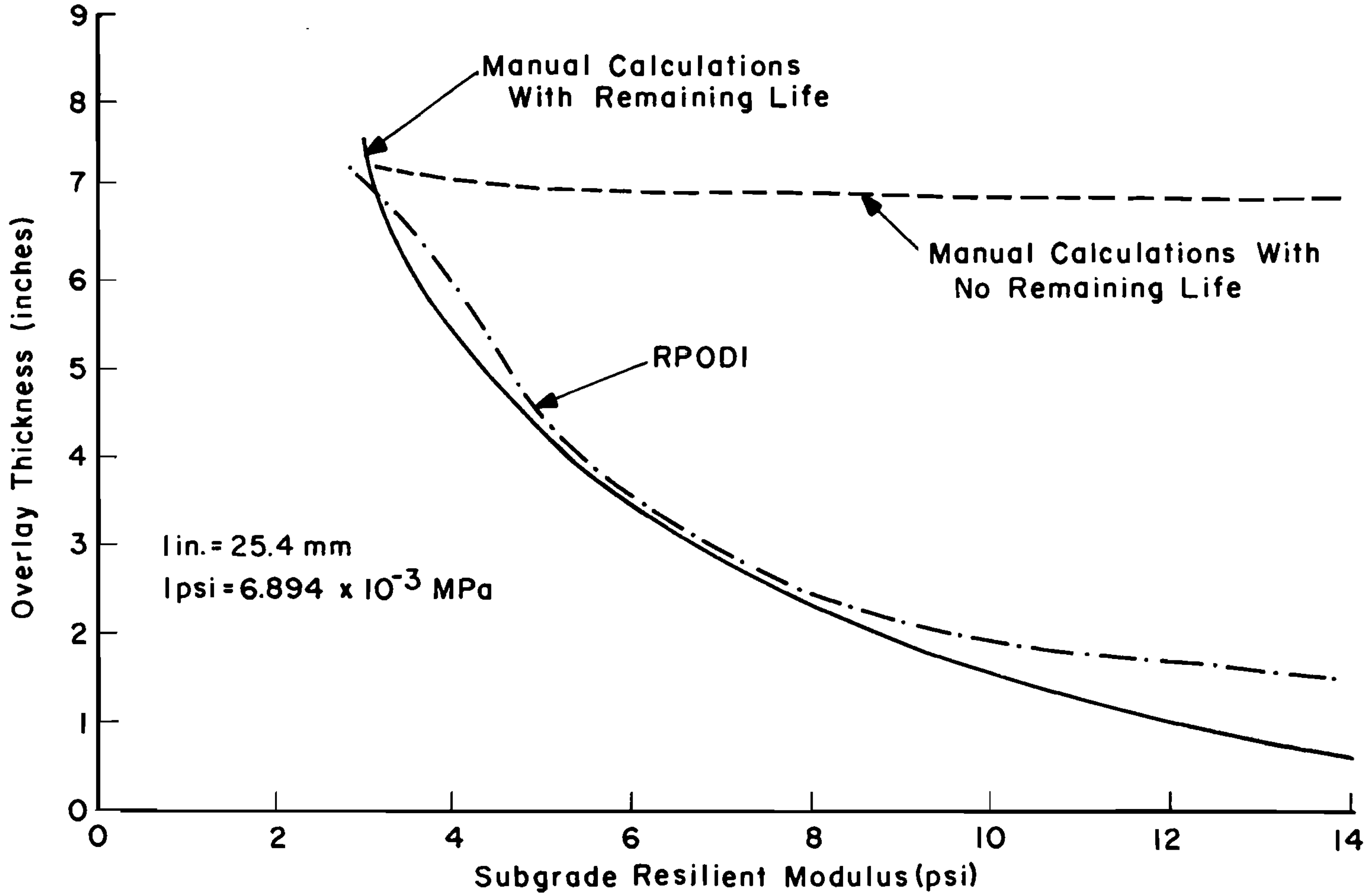


Fig II-2.11. Relationship between overlay thickness and subgrade resilient modulus.

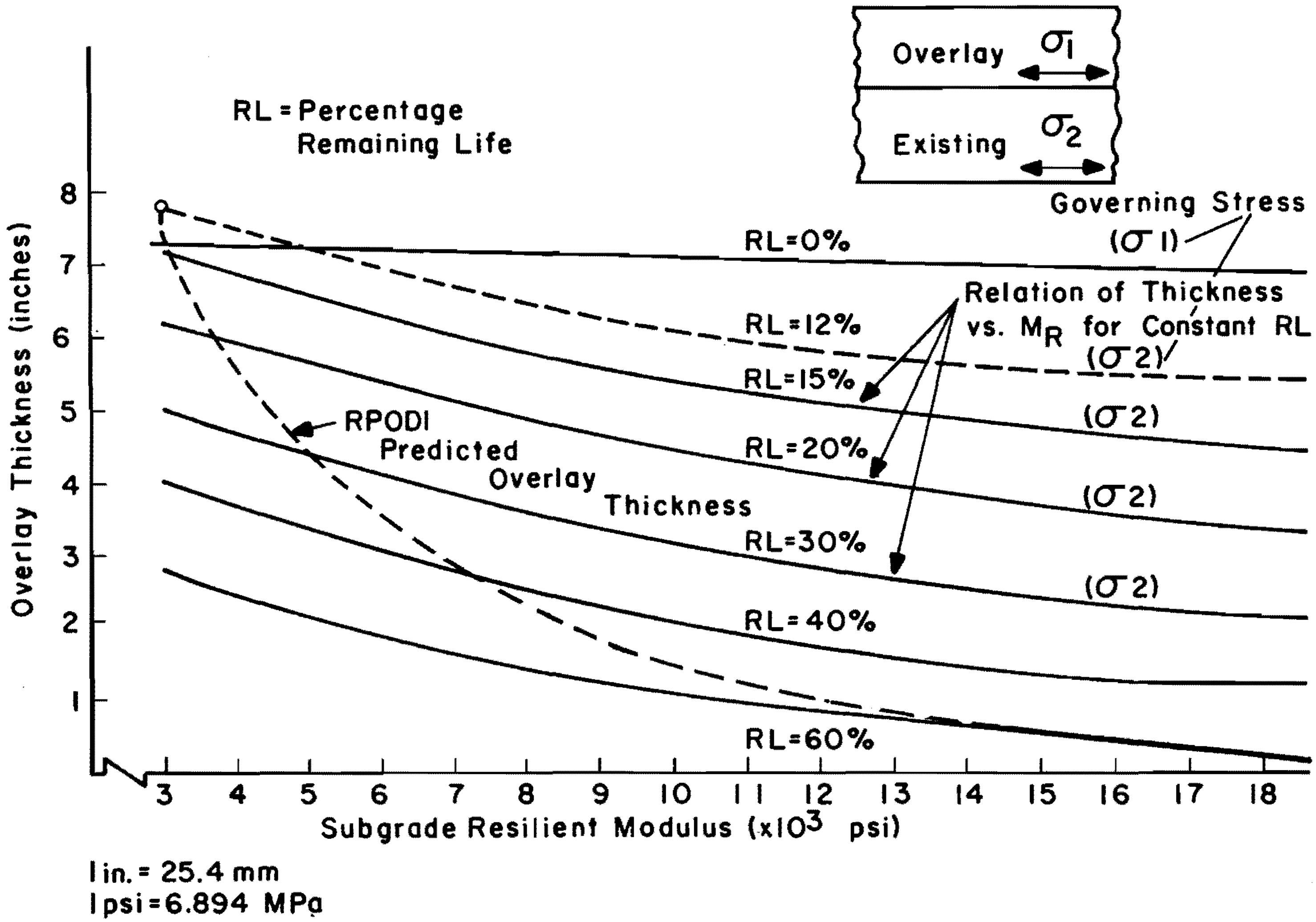


Fig II-2.12. Required overlay thickness versus subgrade M_R for different remaining lifes.

existing pavement with remaining life. For the same existing pavement, if remaining life is not considered in design, this effect is only 0.5 inch (12.7-mm). In the case of existing pavements with remaining life, the fatigue equation is used twice in the overlay design: first to determine the percentage of remaining life and then to calculate the fatigue life of the pavement system after overlay. This makes the predicted overlay thickness more sensitive to changes in stress.

It is interesting to note on Fig II-2.12 that the effect of having an additional layer (the existing pavement without remaining life) between the subgrade and the position of the governing stress, is to make the overlay thickness less sensitive to changes in subgrade support. This can be observed by comparing the general slopes of the relationships derived with the governing stress at the bottom of the overlay (marked with "σ_1") to those where the governing stress was considered to be at the bottom of the existing pavement (marked "σ_2").

The drastic reduction in overlay thickness with increase in subgrade resilient modulus when the existing pavement has remaining life (Fig II-2.12) is due to the combined effect of having the governing stress lower down in the pavement system and the increase in remaining life.

Effect Of the Stress Dependency Of the Subgrade Resilient Modulus on Overlay Thickness

The resilient modulus of subgrade materials is generally stress dependent. As mentioned in Chapter I-3, the design subgrade modulus is determined by means of repetitive loading triaxial testing and deflection measurements in the FHWA method.

When plotted on a log-log scale the modulus versus deviator stress relationship is generally close to a straight line (Refs 1, 5, 14 and 15). Zaniewski (Ref 15) points out that, as confining pressure increases, the resilient modulus of the subgrade material increases but in such a way that individual curves for different confining pressures are parallel. Mathematically he expresses it as follows:

$$M_R = a(\sigma_1 - \sigma_3)^b \qquad \text{(II-2.3)}$$

where

a = intercept on the "subgrade modulus" axis, as shown on Fig I-2.6(a),

b = slope of the log resilient modulus versus log deviator stress line ($= S_{SG}$),

σ_1 = applied vertical stress,

σ_3 = applied horizontal stress,

$(\sigma_1 - \sigma_3)$ = deviator stress.

The intercept on the subgrade modulus, a, is a function of confining pressure and b remains constant (within reasonable limits) with a change in confining pressure.

The slope, b, or S_{SG}, as defined here, is generally negative for clayey materials and positive for granular materials (Ref 6).

Range of S_{SG}. In order to see what the influence of S_{SG} is on overlay thickness, it was necessary to determine a range in which S_{SG} would vary for typical subgrade soils. Laboratory test results, which were readily made available for this project by Austin Research Engineers, Inc., as well as data obtained from reports of the Corps of Engineers (Refs 17 and 18), have been analyzed. Details can be found in Appendix 2.

The only information available for most of the materials considered was resilient modulus test results and a description of the material, generally according to the Unified Soil Classification System. Determination of the ranges of S_{SG} for different soil types was attempted, but with the information available this was not possible. No correlation could be found between S_{SG} and soil type.

For the materials considered, which included clays, silty clays, sandy silts, clayey silts, and a very fine grained sand, a range for

S_{SG} for a 90 percent confidence interval was found to be between -1.17 and +.07 (See Appendix 2). A gravelly sand had an S_{SG} of +0.3.

The RPOD1 program cannot be used for subgrades with a positive S_{SG}, but in general most Texas subgrade soils are likely to have negative S_{SG} values. If a positive S_{SG} is encountered, no stress dependence of the subgrade modulus must be assumed (S_{SG} = 0). This approximation will result in a conservative design.

For purposes of this investigation S_{SG} can be expected to vary for typical subgrade soils in a reasonable range of -1.2 to 0.

Effect of S_{SG} on Overlay Thickness. In this study the following CRCP overlays on CRCP pavements have been considered:

(1) bonded CRCP on CRCP with no voids and no cracks,

(2) unbonded CRCP on CRCP with no voids and no cracks,

(3) bonded CRCP on CRCP, with void, no cracks,

(4) unbonded CRCP on CRCP, with void, no cracks,

(5) CRCP on CRCP, no voids and class 3 and 4 cracking,

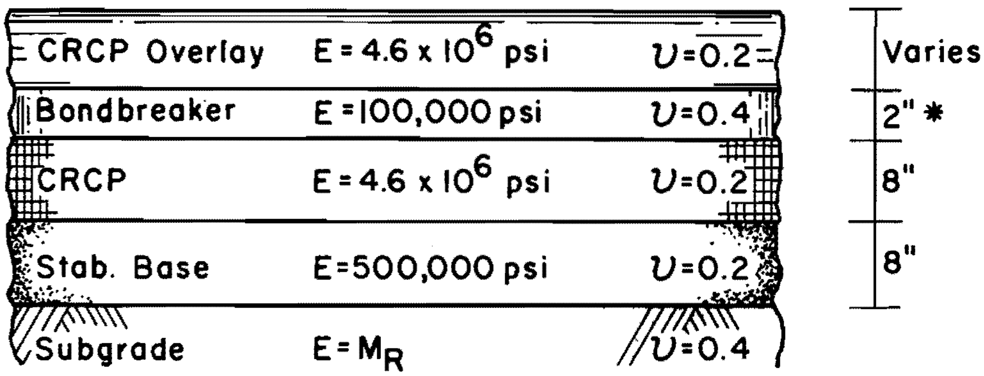
(6) CRCP on CRCP, mechanically broken up.

Two existing pavement structures were considered, one with a stabilized base and the other with a granular base, as shown in Fig II-2.13. Values for input variables to RPOD1 were as determined by Nayak et al. (Ref 7) except for the deflection load, which was selected as a Dynaflect load, and the design deflection, which was selected as 0.565 x 10^{-3} inches (.014-mm) by studying deflection data on various CRCP pavements.

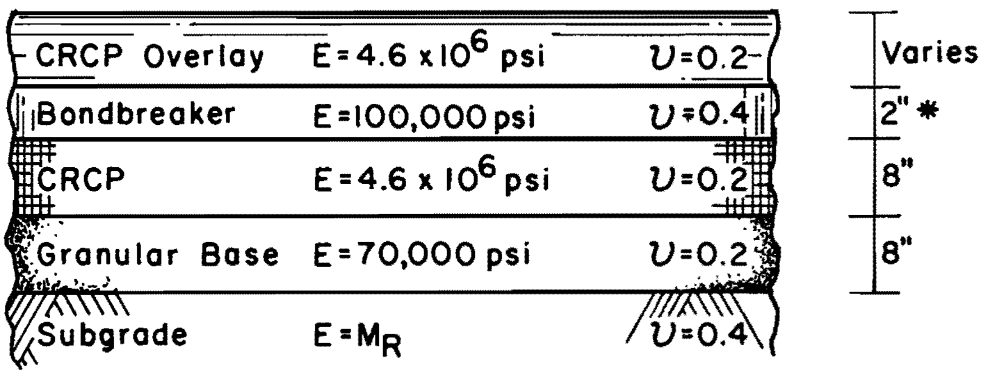
Laboratory data were specified in such a way as to vary S_{SG} from -1.3 to 0. Table II-2.6 indicates how laboratory data input was used to vary S_{SG}. The PROD1 program can only handle negative values for S_{SG} and in order to input S_{SG} = 0 a slightly negative laboratory curve has to be used, as indicated in Table II-2.6.

Traffic applications prior to overlay were selected as 4 million 18-kip (80-kN) equivalent single axle loads and the overlays were designed for 7 million 18-kip (80-kN) equivalent single axle loads.

Figures II-2.14 and II-2.15 show the effect of S_{SG} on overlay thickness for the pavements with the stabilized subbase and the granular subbase, respectively. Making 4 million load applications to the existing pavement



a. Pavement structure with stabilized subbase.



b. Pavement structure with granular subbase.

* Dimension in Cases Where a Bondbreaker Has Been Used

in.= 25.4 mm
1 psi = 6.894×10^{-3} MPa

Fig II-2.13. Pavement structures used in analysis to study the effect of S_{SG} on overlay thickness.

TABLE II-2.6. "LABORATORY DATA" INPUT FOR RPOD1 FOR DIFFERENT VALUES OF S_{SG}

S_{SG}	"Laboratory Data" Input (psi)					
	σ_{dev1}	M_{R1}	σ_{dev2}	M_{R2}	σ_{dev3}	M_{R3}
-1.3	1	20000	5	2468	10	1002
-0.9	1	20000	5	4698	10	2518
-0.6	1	20000	5	7615	10	5024
-0.3	1	20000	5	12340	10	10023
* 0	1	20000.1	5	20000.0	10	19999.9

1 psi = 6.894×10^{-3} MPa

*Only negative S_{SG} values may be specified for RPOD1. To input $S_{SG} = 0$, a slightly negative S_{SG} is specified.

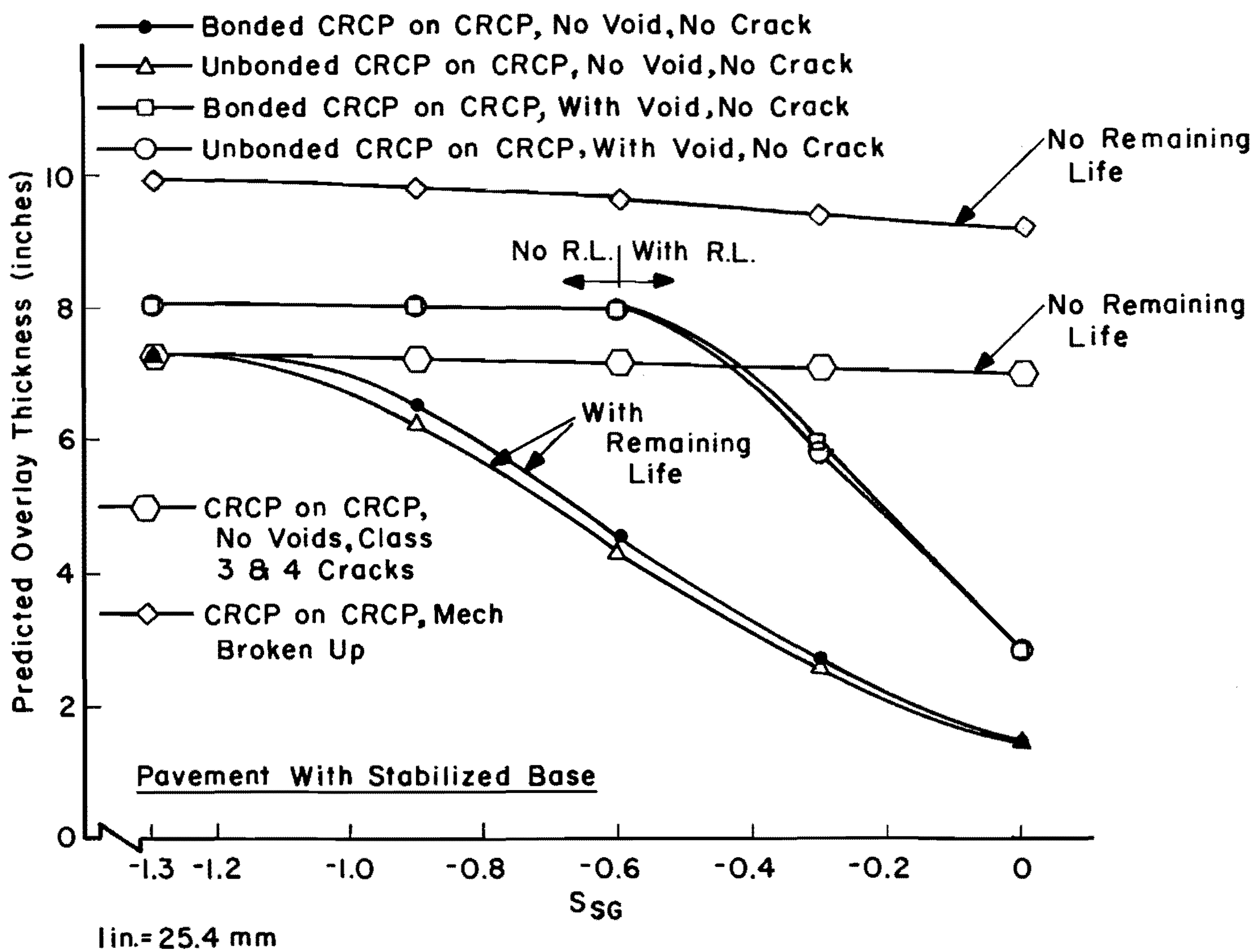


Fig II-2.14. Sensitivity of RPOD1 response to the slope of the subgrade resilient modulus versus deviator stress line (on Log-Log scale).

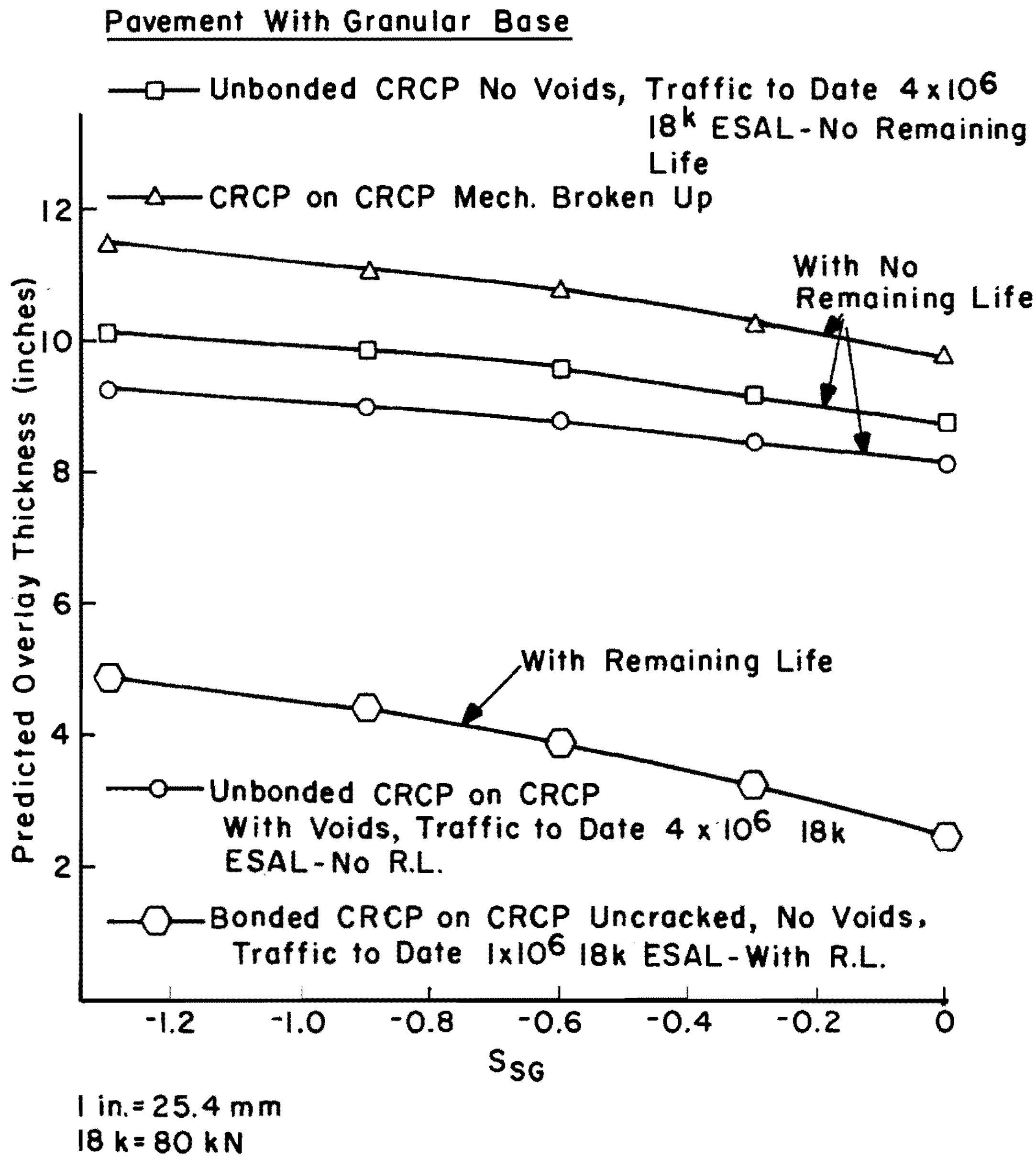


Fig II-2.15. Sensitivity of RPOD1 response to the slope of the log resilient modulus versus log deviator stress line for subgrade.

with a granular subbase resulted in no remaining life; therefore the number of load applications was reduced to 1 million to obtain the curve marked "with remaining life" on Fig II-2.15.

Discussion of Results. The results of this study, especially those on Fig II-2.14, indicate that, in cases where the existing pavement did not have remaining life, the predicted overlay thickness was relatively insensitive to variation in S_{SG}. For existing pavements with remaining life, however, a variation in S_{SG} can result in considerable variation in predicted overlay thickness. This effect is less pronounced in Fig II-2.15, but does exist.

The reason for this phenomenon is that in characterizing the subgrade material, using the measured deflection, different resilient moduli are obtained for materials with different stress sensitivities (S_{SG}). The more stress sensitive the material, the lower the resilient modulus to be used with the design load (for negative values of S_{SG}). It has already been pointed out in the previous section that the subgrade resilient modulus, as well as the percentage of remaining life in the existing pavement, has a great effect on overlay thickness if the existing pavement has remaining life (Figs II-2.8, II-2.12 and II-2.13).

These results suggest that relatively more effort should be put in characterizing the subgrade material in the case of pavements with remaining life, relative to the no-remaining-life case.

The Effect Of Change In Stress Level In The Subgrade, Due To The Overlay, On Predicted Overlay Thickness

In the RPOD1 program, the subgrade modulus is determined under the design load on the existing pavement and this modulus is then used throughout the rest of the overlay design process. The overlay, however, reduces the stress levels in the subgrade which will, for stress sensitive soils with negative values for S_{SG}, result in an increased subgrade resilient modulus. This will cause the design to be conservative. The effect of this increase in resilient modulus on predicted overlay thickness has been studied in this section.

The pavement system studied was an unbonded CRCP overlay on CRCP pavement with no voids present, as indicated on Fig II-2.16. Required overlay thicknesses for both an existing pavement with class 3 and 4 cracks and an uncracked existing pavement were obtained. Design deflection used was 0.565×10^{-3} inch (.014-mm).

<u>Analysis for an Existing Pavement with Remaining Life</u>. The adjusted lab curves indicated on Fig II-2.17 have been developed similarly to those in Fig II-2.3. Also plotted on Fig. II-2.17 is the computed resilient modulus versus deviator stress relationship, at the top of the subgrade, resulting under the design load, for overlays ranging from 0 to 8 inches (0 - 203.2-mm). Note that the effect of the overlay is to reduce the deviator stress at the top of the subgrade for a given pavement structure and subgrade modulus. The subgrade resilient modulus for a specific overlay thickness and S_{SG} value can be determined from Fig II-2.17. At the point where the subgrade resilient modulus versus deviator stress curve, for the pavement with the specified overlay thickness, intersects the adjusted laboratory curve, with the specified S_{SG} value, the subgrade resilient modulus corresponding to the deviator stress at the top of the subgrade, under design load conditions, can be determined.

Figure II-2.18 indicates the calculated maximum horizontal tensile stress at the bottom of the existing layer, after overlay, versus subgrade resilient modulus for different overlay thicknesses. The maximum allowable horizontal tensile stress at the bottom of the existing pavement, for 7 million 18-kip (80-kN) equivalent single axle loads, has been calculated for different percentages of remaining life, using Eq I-3.3 and I-3.5. The relationship between percentage of remaining life and S_{SG} was obtained from Fig II-2.4, which was derived for the same existing pavement. These maximum allowable tensile stress values are also indicated on Fig II-2.18.

Through a process of interpolation between Figs II-2.17 and II-2.18, overlay thicknesses have been determined, taking into account the reduction in subgrade stress level due to the overlay. The results are indicated in Table II-2.7.

<u>Analysis for Pavement with Class 3 and 4 Cracking</u>. In the analysis of the pavement with class 3 and 4 cracking, the pavement structure is the

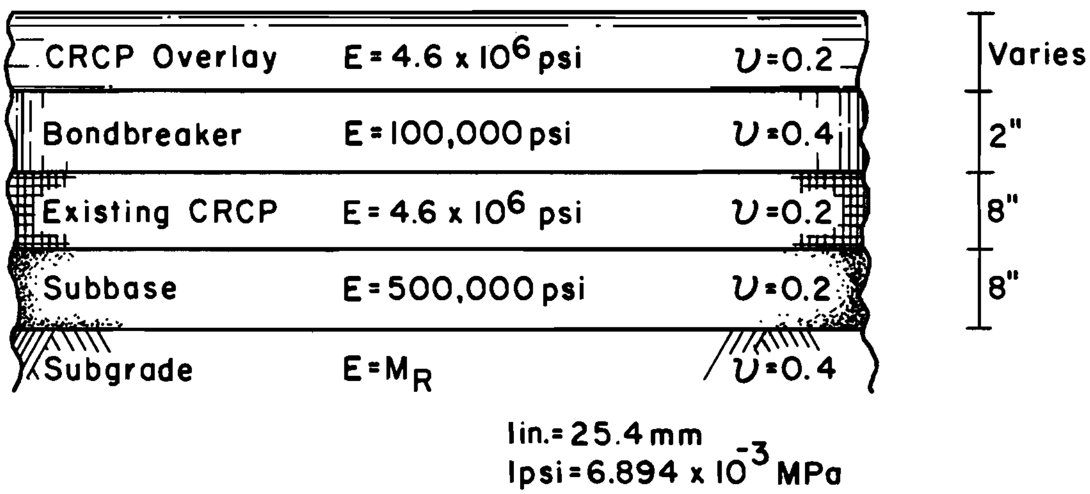


Fig II-2.16. Pavement structure used to evaluate effect of reduction of stress level due to overlay on overlay thickness.

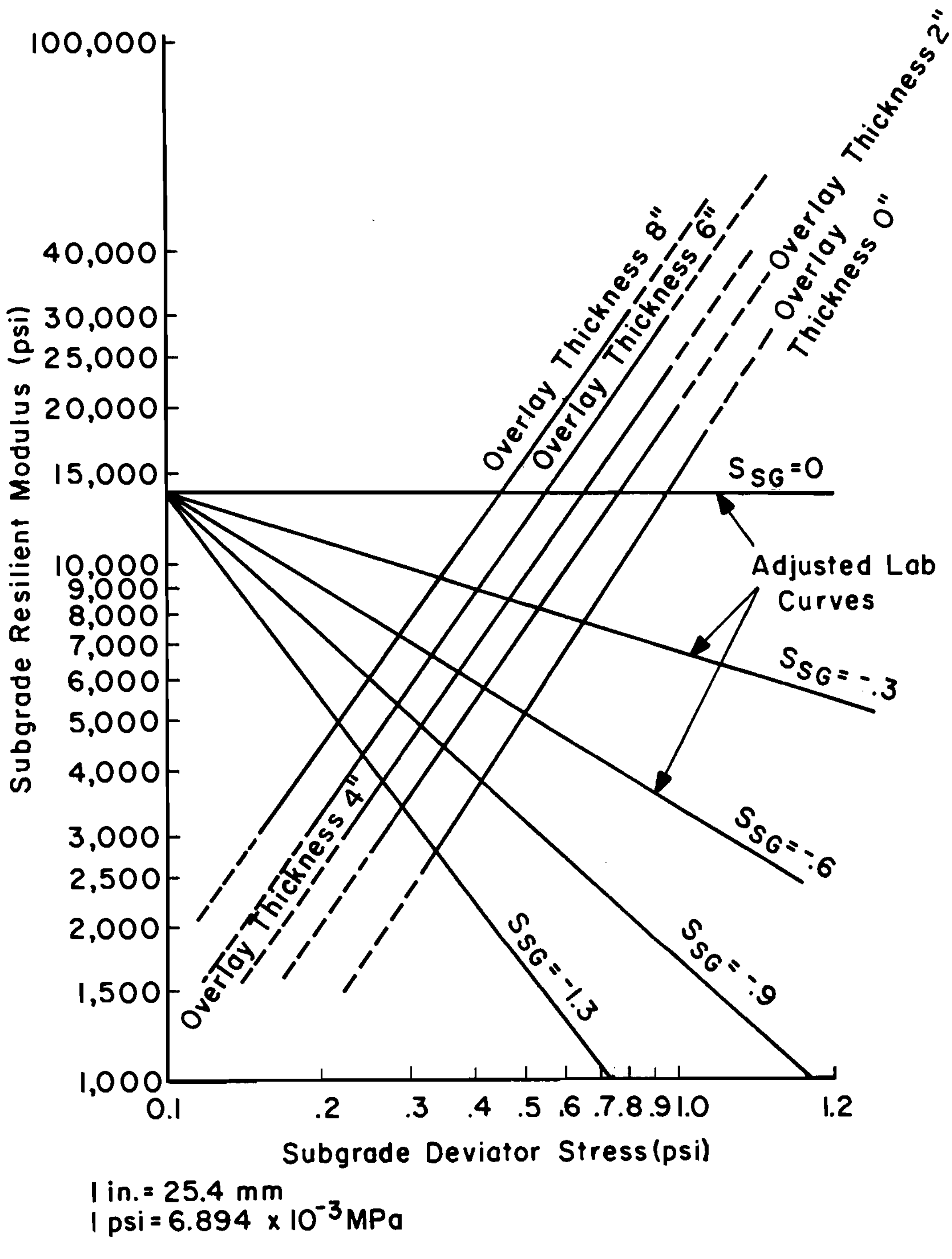


Fig II-2.17. Chart for obtaining resilient modulus versus overlay thickness relationship for various subgrade materials - existing pavement with remaining life.

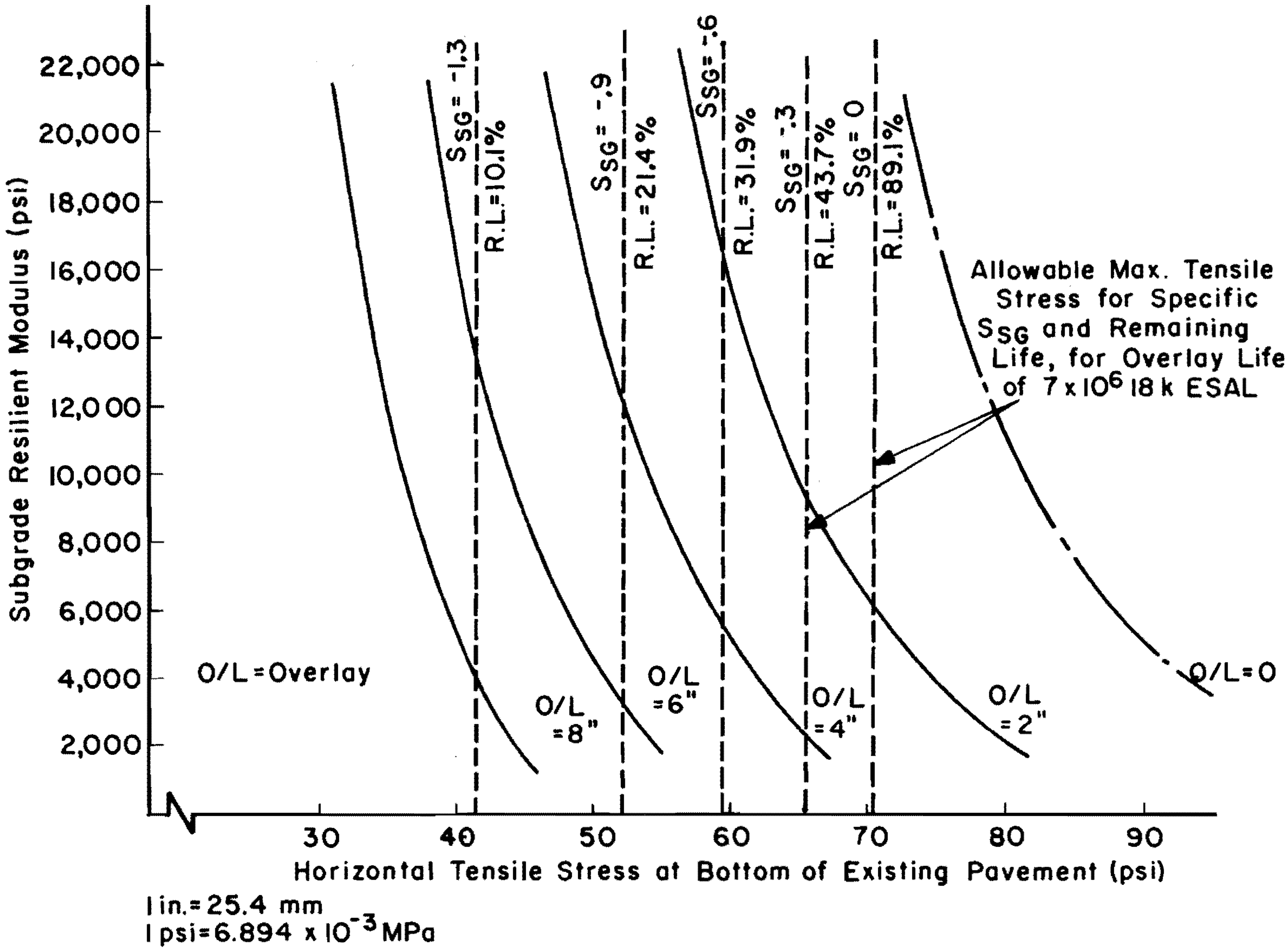


Fig II-2.18. Relation between subgrade modulus and maximum horizontal tensile stress at bottom of existing pavement for different overlay thicknesses - existing pavement without cracking.

TABLE II-2.7. COMPARISON OF RPOD1 PREDICTED OVERLAY THICKNESSES WITH THICKNESSES PREDICTED, TAKING REDUCTION IN SUBGRADE STRESS LEVEL DUE TO THE OVERLAY INTO ACCOUNT FOR EXISTING PAVEMENT WITH REMAINING LIFE

S_{SG}	RPOD1		Method with Reduced σ_{dev} due to Overlay	
	Overlay Thickness, inches	Remaining Life, %	Overlay Thickness, inches	Remaining Life, %
0	1.5	55.6	1.5	57.1
-0.3	2.6	41.7	2.1	43.7
-0.6	4.3	29.2	3.9	31.9
-0.9	6.2	18.3	5.5	21.4
-1.3	7.3	6.6	6.9	10.1

1 in. = 25.4 mm

These values are plotted on Fig II - 2.19.

same as that used for the pavement with remaining life (Fig II-2.16). Using the same procedure as before, design curves were derived for this condition. Again by a process of iteration, the required overlay thicknesses for subgrade materials with different stress sensitivities were obtained, taking into consideration the reduction in subgrade stress level due to the overlay. The results obtained are given in Table II-2.8.

Discussion. Table II-2.8 indicates that neglecting the reduction in subgrade stress level due to the overlay, for an existing pavement with class 3 and 4 cracking, results in a slightly conservative design. The greatest difference in overlay thickness indicated in Table II-2.8 is, however, only 0.3 inch (7.6-mm).

For pavements with remaining life the design is also conservative if the effect of the overlay on the subgrade stress level is not considered. (See Fig II-2.19). Table II-2.7 indicates that the RPOD1-predicted thicknesses are up to .7 inch (15.8-mm) thicker than when the reduction in subgrade stress level is taken into account. This can, however, be considered as a built-in safety factor which is not inappropriate because of the sensitivity of the predicted overlay thickness to percentage of remaining life of the existing pavement and subgrade resilient modulus.

A further iteration process in the program, to take this reduction in stress level into account, will increase computer cost considerably, and is not considered to be worth the effort.

The discussion mentioned above pertains only to subgrade soils with a negative value for S_{SG}. Since RPOD1 cannot handle positive S_{SG} values, in which case the assumption needs to be made that S_{SG} is zero, there will be no danger of under design because of this factor.

Asphalt Concrete Overlays On Portland Cement Concrete Pavements

The fatigue cracking analysis of asphaltic concrete overlays, using the RPOD1 program, did not pose any problem for existing portland cement concrete pavements with remaining life. Nayak et al. (Ref 7) included a pavement of this nature in their sensitivity analysis without any difficulty. The governing stress in this case is considered to be at the bottom of the existing pavement.

TABLE II-2.8. COMPARISON OF RPOD1 PREDICTED OVERLAY THICKNESSES WITH THICKNESSES PREDICTED, TAKING REDUCTION IN SUBGRADE STRESS LEVEL DUE TO THE OVERLAY INTO ACCOUNT FOR EXISTING PAVEMENT WITH CLASS 3 AND 4 CRACKING

S_{SG}	RPOD1 Predicted Overlay Thicknesses, inches	Overlay Thickness, inches (with reduced σ_{dev} due to overlay)
0	7.0	6.8
-0.3	7.1	6.9
-0.6	7.2	6.9
-0.9	7.2	7.0
-1.3	7.3	7.0

1 in. = 25.4 mm

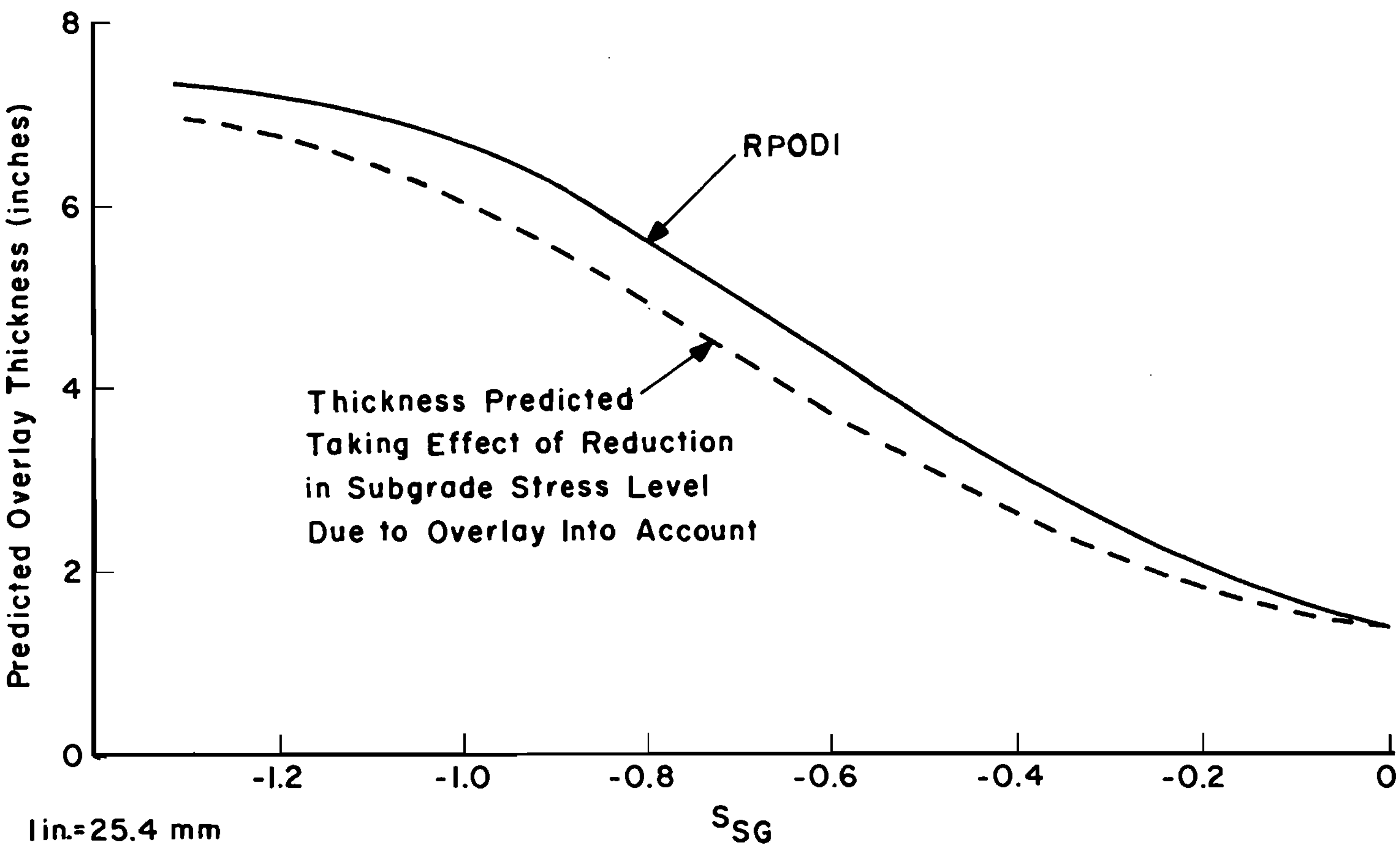


Fig II-2.19. Comparison of RPOD1 predicted thicknesses with manually calculated thicknesses taking reduction in subgrade stress level due to overlay into account.

With asphaltic concrete overlays on rigid pavements without remaining life, a problem has been encountered using layered theory. This type of overlay has not been implemented in the RPOD1 program. In this case, the existing pavement effective modulus of 500,000 psi (3447 MPa) can easily be higher than the modulus of the overlay. The governing stress is considered to be at the bottom of the overlay, which can, in some instances, according to layer theory solutions, even be in compression. Considerable tensile stresses are predicted in the existing cracked pavement.

To study this phenomenon, two pavements have been considered, as indicated in Fig II-2.20. Pavement A had a cement stabilized subbase and pavement B a granular subbase. The loading conditions, material properties, and layer thicknesses used are indicated on Fig II-2.20. Indicated in Fig II-2.21 are the maximum horizontal stresses at the bottom of various layers for the two pavements considering different overlay thicknesses.

Discussion. It can be seen on Fig II-2.21 that layered theory predicted only compressive stresses in the overlay for both pavements. The existing pavement, in the case of pavement B, and the subbase, in the case of pavement A, experienced considerable tensile stresses, which it would not be able to withstand, since the existing pavement is considered to be a cracked pavement. It is clear that this is not an easy problem to deal with using layered theory. A solution to this problem is discussed later in this chapter under "Modifications to the Fatigue Cracking Program."

Comparison Of RPOD1 With a Simplified Method Using Westergaard Equations For Calculations Of stresses and Deflections

Since using linear elastic layer theory is not the traditional way of analyzing rigid pavements, the purpose of this study was to use the basic procedures in RPOD1 but to use Westergaard equations (Ref 19) instead of layer theory, to determine stresses and surface deflections of the pavement slabs. Results obtained using this simplified method are compared to RPOD1 results here.

The basic procedure used in RPOD1 was followed for this simplified method with the exception that stresses and deflections were computed by Westergaard equations as follows:

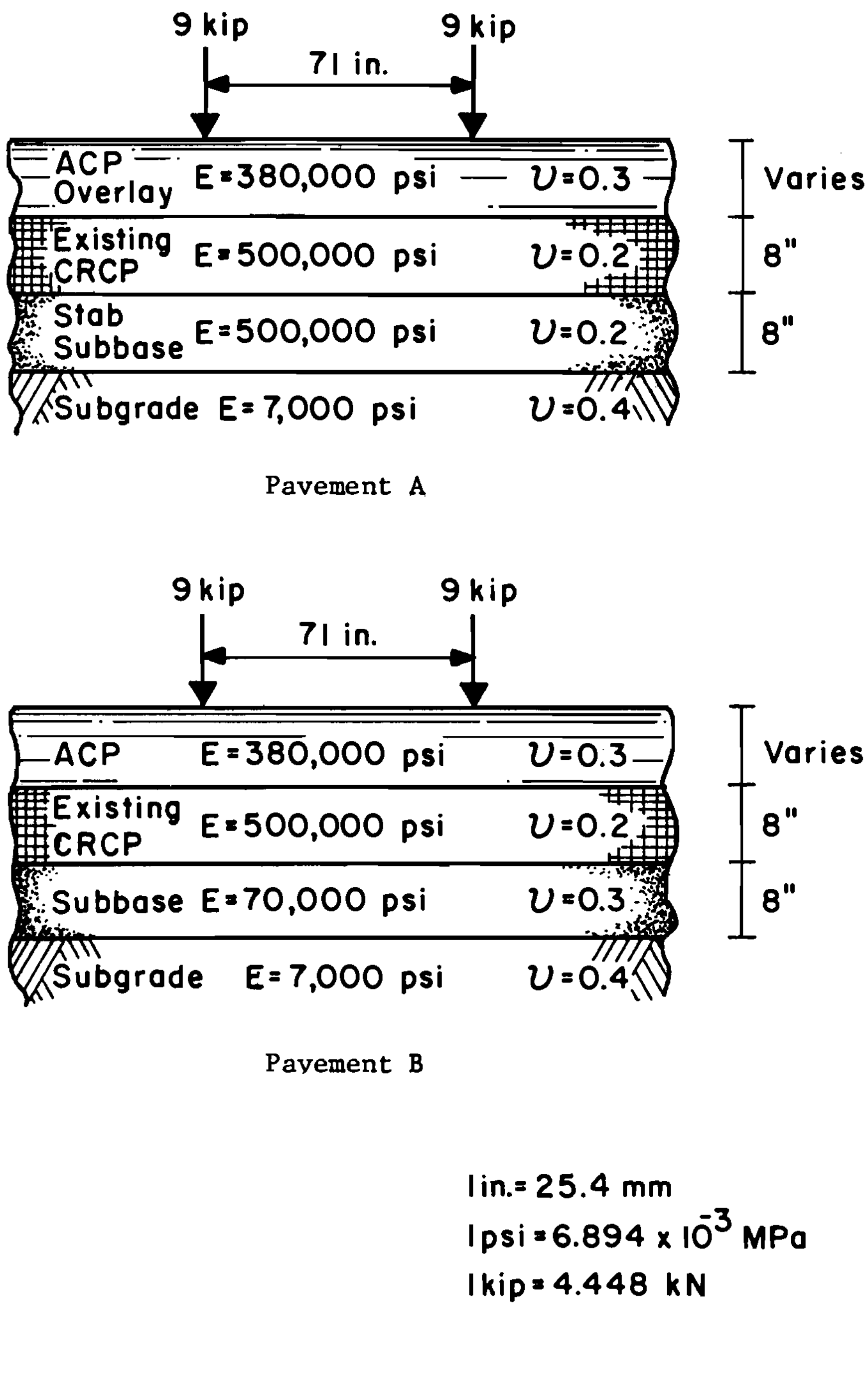


Fig II-2.20. Pavements considered for AC overlay on cracked existing pavement.

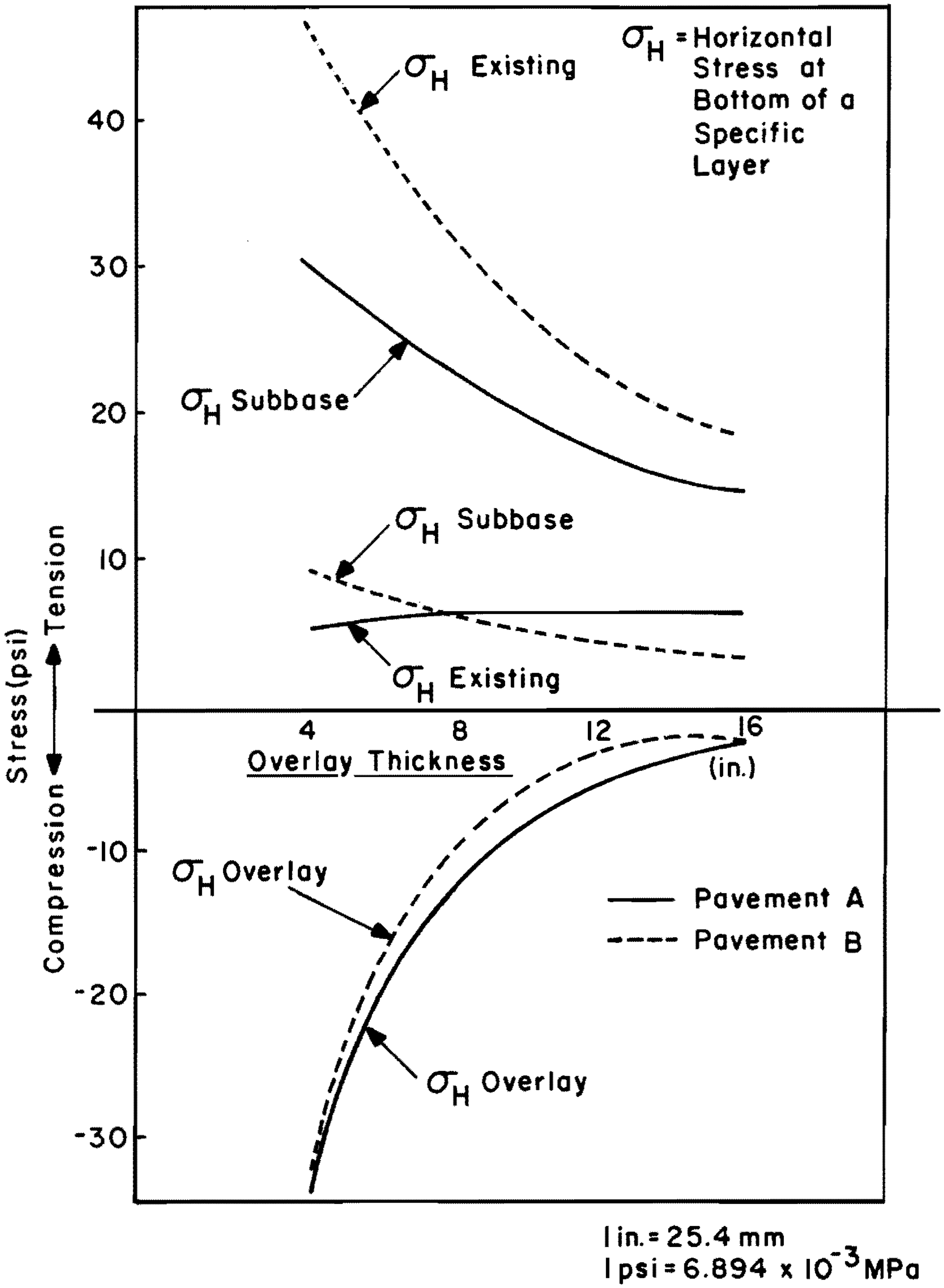


Fig II-2.21. Relations between stresses and overlay thicknesses for pavements A and B.

(1) Westergaard interior deflection equations were used to determine k-values under the existing pavement for the design deflection. (For deflection equations used see Appendix 2.)

(2) With this k-value, the original life of the existing pavement could be determined, using Westergaard stress equations as well as the fatigue equation. These equations are given in Appendix 3.

(3) Taking the traffic prior to overlay into account, the remaining life of the existing pavement was determined.

(4) By using an "effective" thickness concept (see Appendix 3), the overlay thickness was determined. In the case of pavements with remaining life the governing stress was taken to be at the bottom of the existing layer, taking remaining life into consideration. For pavements without remaining life, the existing pavement was considered to be a stabilized subbase. A composite k-value was determined and the overlay was designed as a new pavement on this "subbase."

(5) Stresses were computed using the Westergaard corner stress equation for jointed pavements and the edge stress equation for continuous pavements.

As an alternative, the modulus of subgrade support was determined using layered theory, and a correlation between resilient modulus and modulus of subgrade reaction was determined. Results of both analyses are given in this section.

Deflection, as well as design load, was taken as a 9-kip (40-kN) load.

Relationship Between Resilient Modulus and Modulus of Subgrade Reaction. By using the composite modulus of subgrade reaction equation discussed in Appendix 3 (Eq A3.8) and setting $E_3 = E_4$, a relationship between resilient modulus and modulus of subgrade reaction was established. Figure II-2.22 shows this relationship.

Pavements Considered in this Analysis. Two pavements were considered in this analysis:

(1) 9-inch (228.6-mm), continuously reinforced concrete pavement with a design deflection of .008 inch (0.2-mm) under a 9-kip (40-kN) load; and

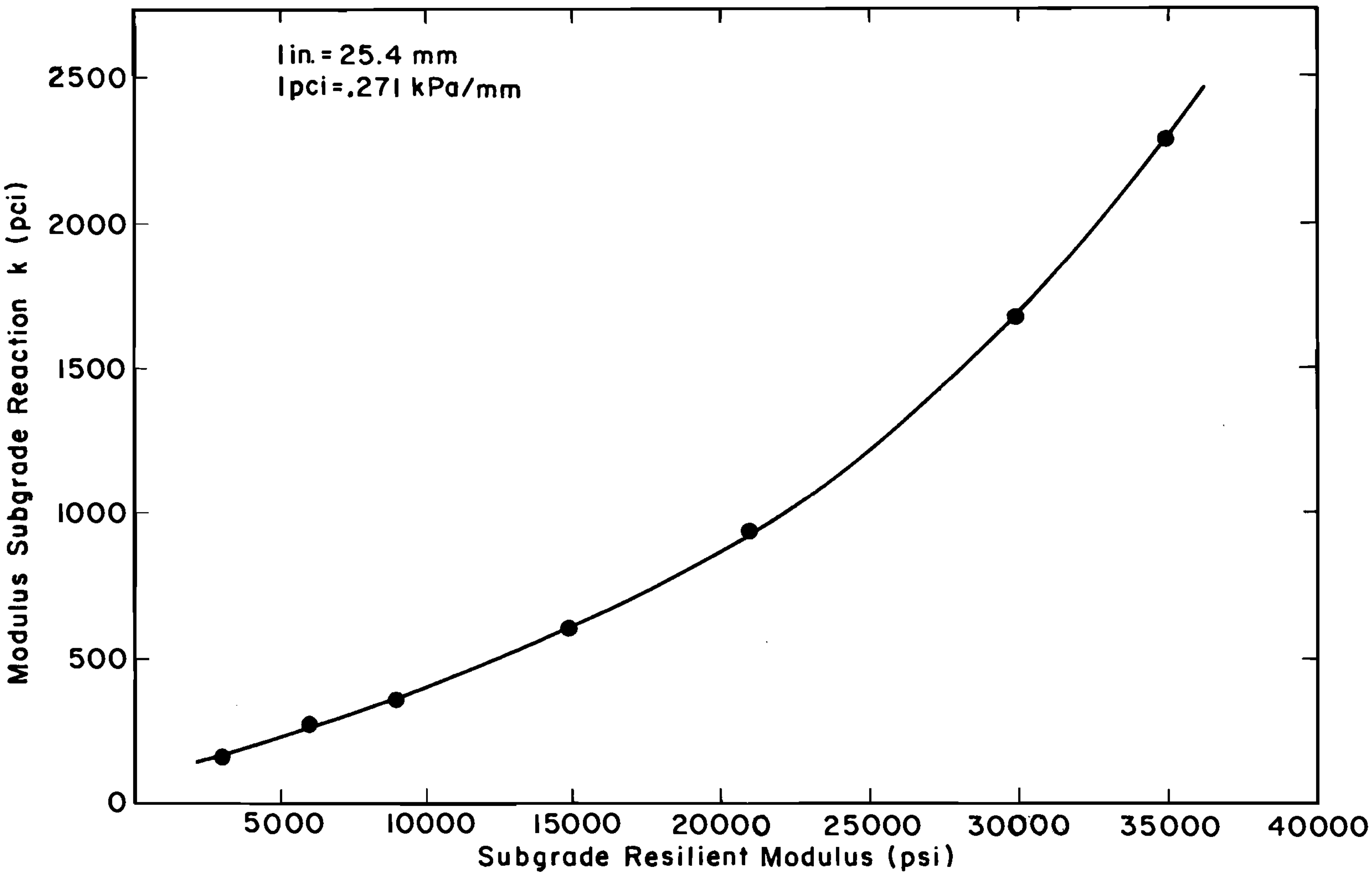


Fig II-2.22. Relationship between subgrade resilient modulus and modulus of subgrade reaction.

(2) 10-inch (254-mm), jointed concrete pavement with a design deflection of .008 inch (0.2 mm) under a 9-kip (40-kN) load.

Concrete elastic modulus, flexural strength, and Poisson's ratio values were assumed to be 4 x 10^6 psi (27576 MPa), 700 psi (4.8 MPa) and 0.2, respectively.

Effect of the Remaining Life on Overlay Thickness. Overlay thicknesses were predicted using

(1) RPOD1;

(2) the simplified method using Westergaard equations to calculate stresses and the Westergaard interior deflection equation to characterize the subgrade material; and

(3) the simplified method using Westergaard equations to calculate stresses and layer theory to characterize the subgrade material.

These calculations were made for both the 10-inch (254-mm) JCP existing pavement with JCP overlay and the 9-inch (228.6-mm) CRCP existing pavement with CRCP overlay. The deflection used in both cases was .008 inch (0.2-mm) under a 9000-pound (40-kN) wheel load. A design traffic for the overlay of 7 x 10^6 18-kip (80-kN) equivalent single axle loads has been assumed.

In order to study the effect of the remaining life on predicted overlay thicknesses, the traffic prior to the overlay was varied keeping all other factors constant. This resulted in a variation in percentage of remaining life.

Figure II-2.23 shows the comparison of results obtained for overlay thickness using the three methods discussed above for the 9-inch (228.6-mm) CRC existing pavement, and Fig II-2.24 is a similar plot for the 10-inch (254-mm) JC existing pavement.

Effect of Subgrade Support on Overlay Thickness. In this section the effect of the subgrade support value (resilient modulus for layered theory and modulus of subgrade reaction for Westergaard theory) on the overlay thickness was studied.

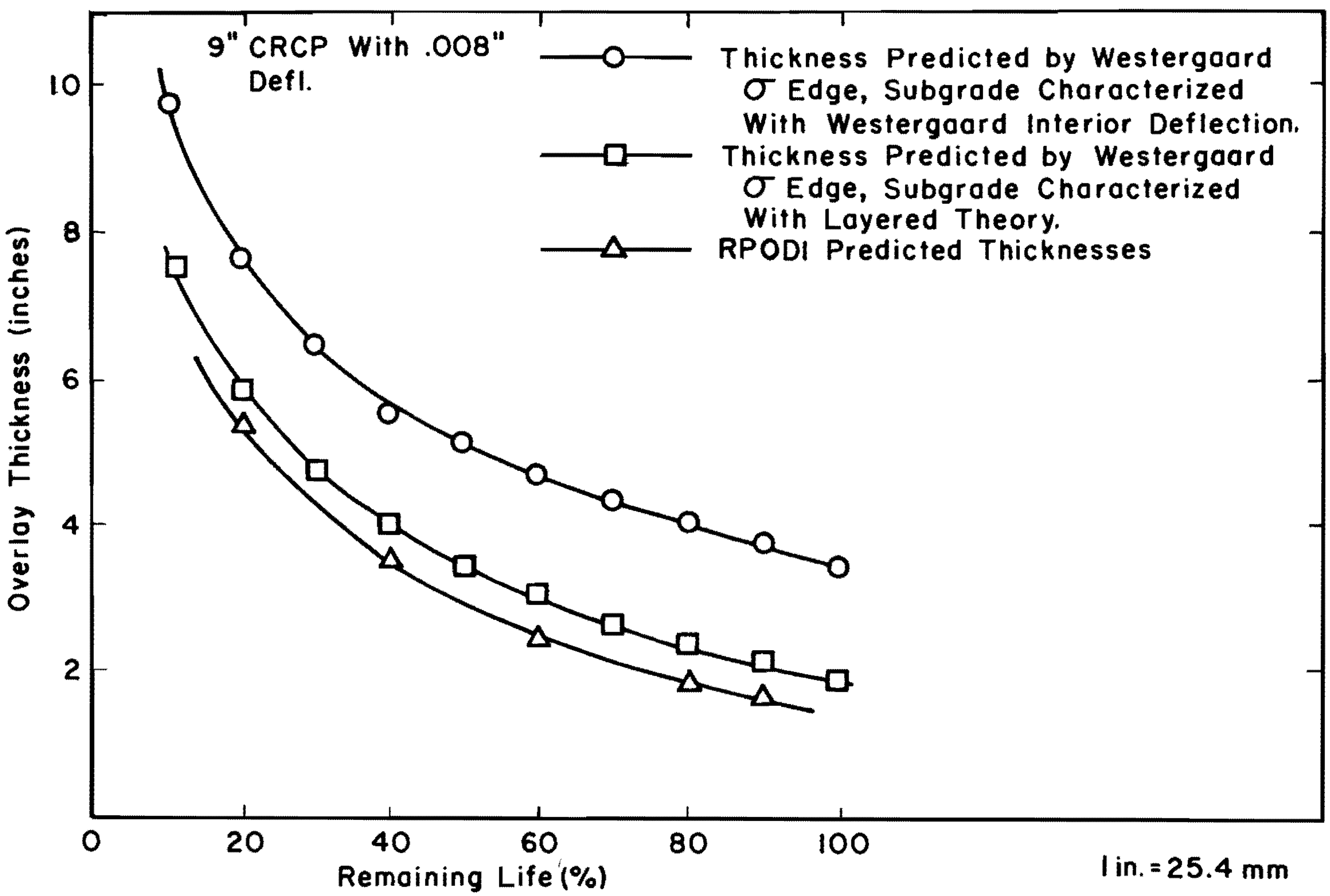


Fig II-2.23. Effect of remaining life on overlay thickness for 9" CRCP existing pavement.

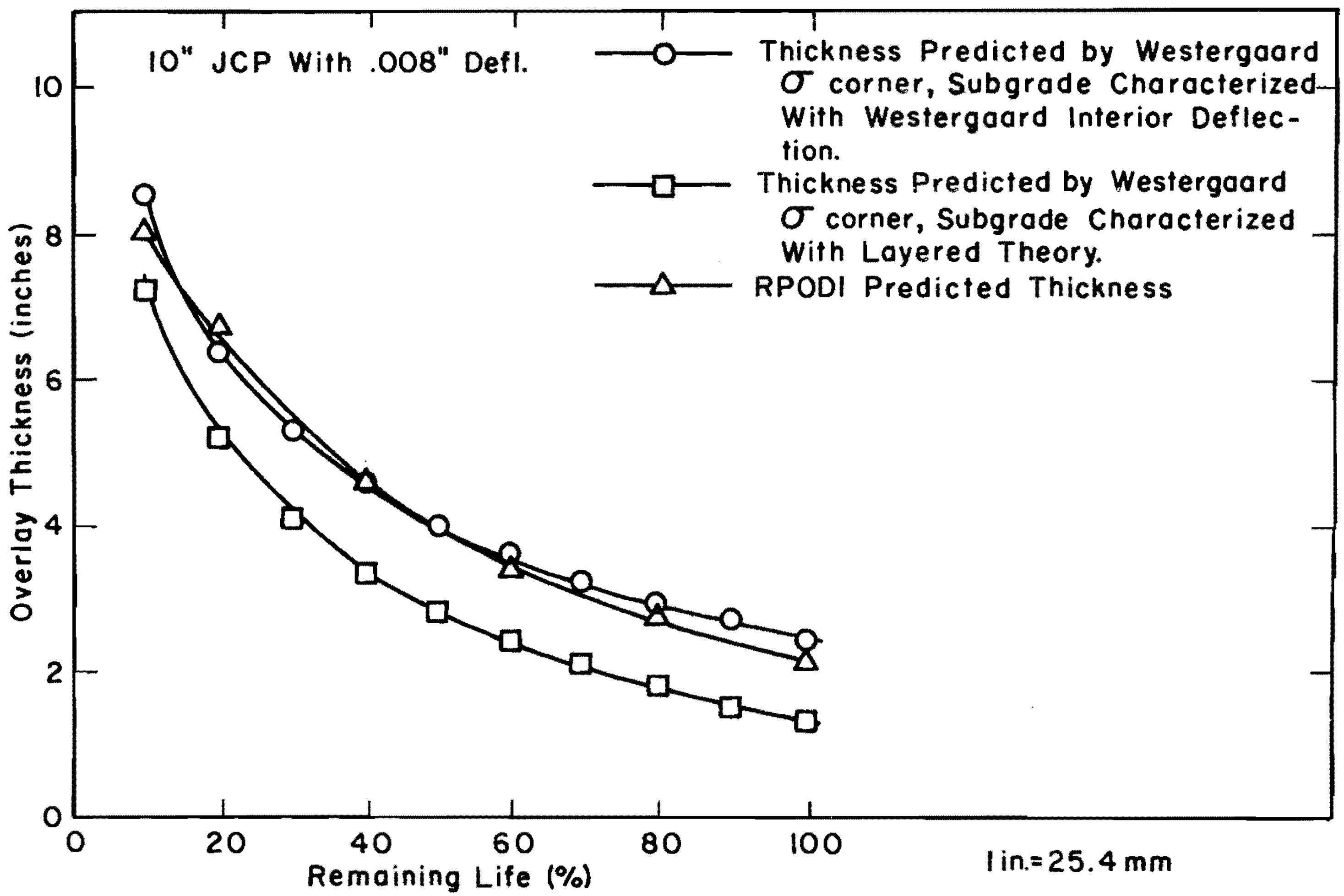


Fig II-2.24. Effect of remaining life on overlay thickness for 10" JCP existing pavement.

Here, RPOD1 and the simplified method were used to obtain overlay thicknesses. Subgrade resilient modulus and modulus of subgrade reaction were correlated using the relationship of Fig II-2.22.

In this case the subgrade support values were varied, and overlay thicknesses predicted for an overlay design traffic of 7 million 18-kip (80-kN) equivalent single axle loads were determined. A value of 3 million 18-kip (80-kN) equivalent single axle loads was assumed for traffic prior to overlay. These claculations were made for both the 9-inch (228.6-mm) CRCP and the 10-inch (254-mm) JCP.

Two sets of calculations were made: one taking the remaining life of the existing pavement into consideration and the other not.

Comparisons of overlay thicknesses determined by the various methods for the 9-inch (228.6-mm) CRC existing pavement with a CRCP overlay are indicated on Fig II-2.25 and Fig II-2.26 gives the same information for the 10-inch (254-mm) JC existing pavement with JCP overlay.

Comparison of Calculated Stresses and Deflections Using Westergaard Equations. Table II-2.9 shows calculated Westergaard stresses and deflections for various slab thicknesses and k-values.

Discussion. Linear elastic layered theory is traditionally not used for designing rigid pavements because it assumes among other things that all layers are uniform, homogeneous, and infinite in the horizontal direction, a requirement which rigid pavements with joints or cracks clearly do not meet. It can however be assumed that interior stresses can be calculated with layered theory where the slab is assumed to be homogeneous in all directions (Ref 1).

Deflections are greatly influenced by the subgrade layer, and this layer can contribute 70 to 95 percent to the deflection, depending on the pavement structure (Ref 20). Since, with layered theory, the influence of subgrade and subbase layers can be accounted for better than with plate theory, characterization of the subgrade by means of deflection measurements can be done more accurately. With layered theory, it is relatively easy to take stress dependency of subgrade support (resilient modulus) into account. This can be done by the laboratory resilient modulus test. It is,

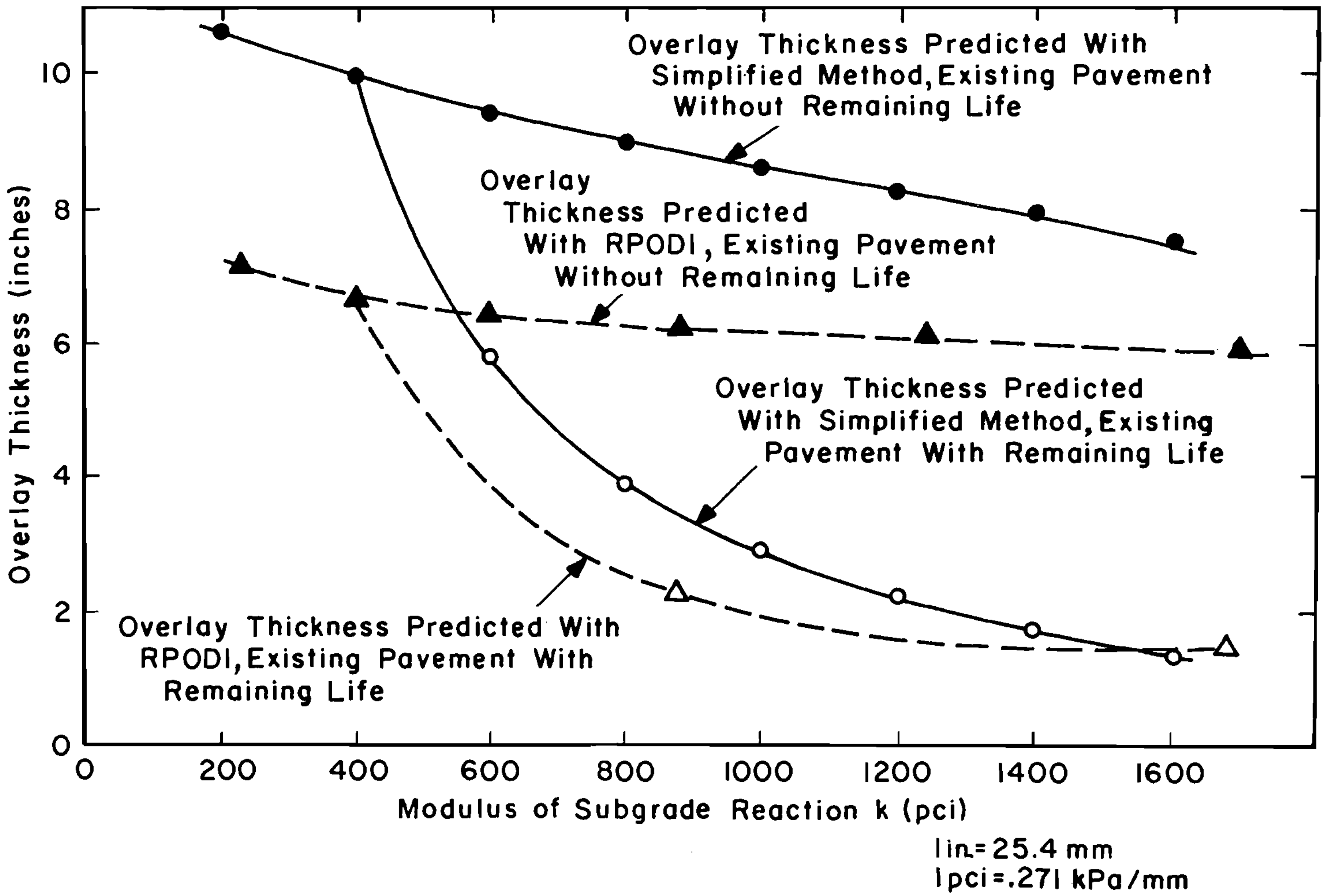


Fig II-2.25. Effect of subgrade support (k) on overlay thickness for 9" CRCP existing pavement.

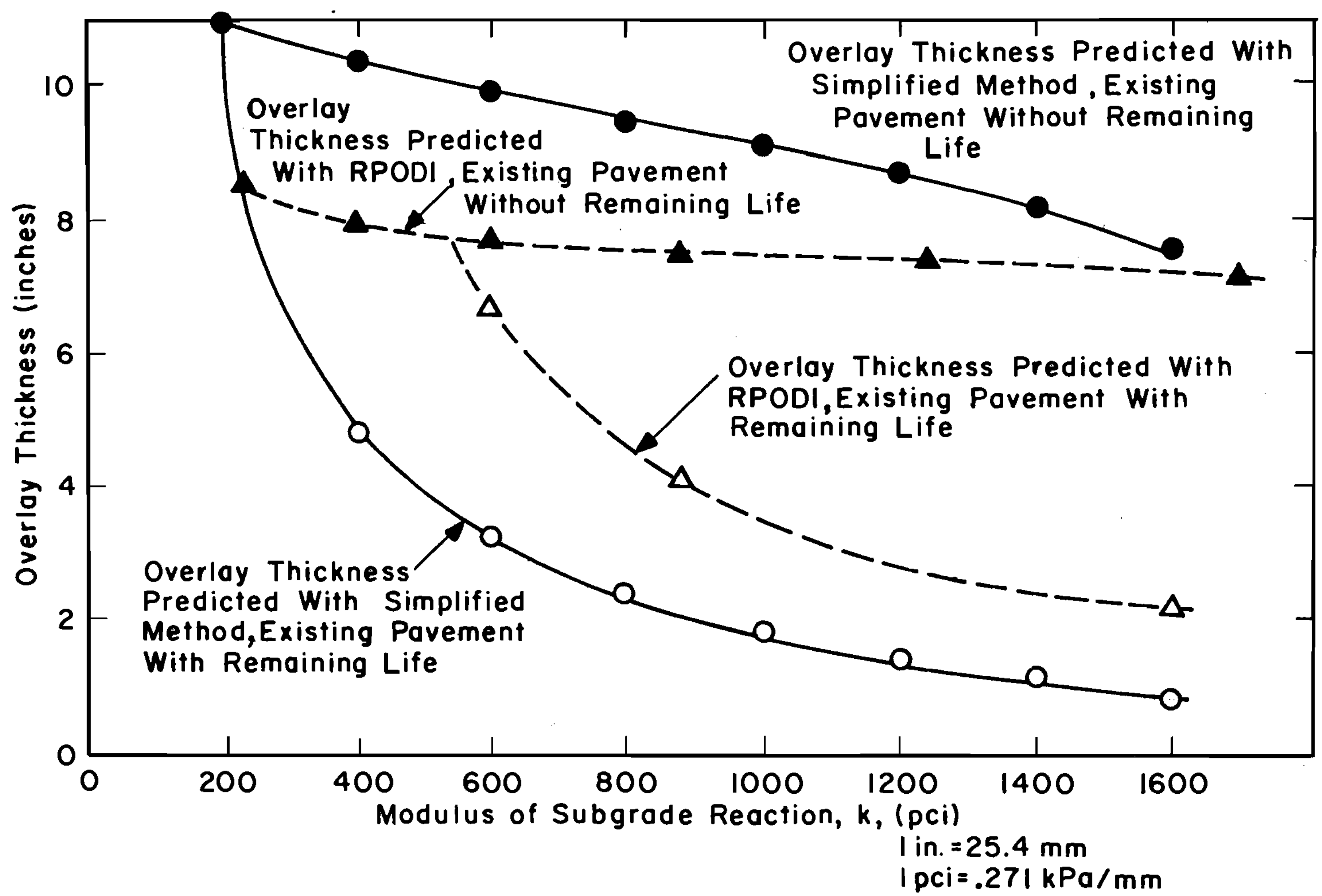


Fig II-2.26. Effect of subgrade support (k) on overlay thickness for 10" JCP existing pavement.

TABLE II-2. 9. CALCULATED WESTERGAARD STRESSES AND DEFLECTIONS

Position	Slab Thickness	k pci	Deflection (in.)	Stress psi
Interior	9"	85	.008	135.5
Interior	10"	62	.008	117.7
Edge	9"	85	.028	200.0
Edge	10"	62	.028	176.4
Corner	9"	85	.058	202.5
Corner	10"	62	.060	173.6

Slab Thickness	Edge Stress / Interior Stress	Corner Stress / Interior Stress
9"	1.48	1.49
10"	1.50	1.47
Average	1.49	1.48

1 in = 25.4 mm   1 psi = 6.894×10^{-3} MPa

however, not so easy to take stress dependency of the material into account when dealing with Westergaard modulus of subgrade reaction.

When studying the effect of the remaining life of the pavement on overlay thickness (Figs II-2.23 and II-2.24), it can be seen that all three methods show a drastic decrease in thickness if remaining life is taken in consideration. For the 9-inch (228.6-mm) CRCP, it can be noted that the RPOD1 predictions were close to the thicknesses predicted by the simplified method using layered theory for the characterization of the subgrade material. For the 10-inch (254-mm) JCP, however, it can be seen that the RPOD1 predictions were close to that of the simplified method using the Westergaard interior deflection equation to characterize the subgrade material.

Table II-2.9 indicates that the average Westergaard edge to interior stress ratio was 1.49 which is higher than the 1.2 used in RPOD1 (Table I-3.2) in the case of CRCP overlays on CRCP. The corner to interior stress ratio of 1.48 is within the range used in RPOD1 (Table I-3.2) and is in fact very close to the default value of 1.5, used for this ratio, in the RPOD1 program (Refs 1 and 6). McCullough (Ref 5) compared Westegaard's interior equations to layered theory and concluded

> The deflections predicted by the two models differ considerably especially for poor soils and normal concrete. Although this latter factor presents a discrepancy between the two models, the comparison does indicate that the two models may be used interchangeably with approximately the same degree of confidence.

In general the deflections predicted by layered theory were found to be higher than those predicted by Westergaard equations. This is why the support values predicted with the Westergaard interior deflection equation were 85 pci (23.0 KPa/mm) and 62 pci (16.8 KPa/mm) for the 9-inch (228.6-mm) and 10-inch (254-mm) pavements, respectively, while layered theory predicted k-values of 420 pci (113.8 KPa/mm) and 380 pci (103.0 KPa/mm). McCullough (Ref 5) also points out that predicted deflections are often higher than measured, partly due to the assumption of a semi-infinite subgrade thickness. This can be overcome by reducing the subgrade thickness. The RPOD1 program is capable of simulating the presence of bedrock at a depth to be specified by the user. McCullough points out that for subgrade thicknesses of less than 12 foot (3.7-m) a variation in subgrade thickness has a significant effect on deflection, especially for lower soil support values.

The effect of the subgrade support on the predicted overlay thickness can be seen on Figs II-2.25 and II-2.26 for the 9-inch (228.6-mm) CRC existing pavement and the 10-inch (254-mm) JC existing pavement, respectively. Although the absolute values of the predicted overlay thicknesses are not in good agreement for reasons previously mentioned, it can be seen that for both methods the subgrade support value had a relatively small effect on the overlay thickness for pavements without remaining life, whereas the effect of the value of the subgrade support is much greater in the case of pavements with remaining life. This effect can be seen for both the RPOD1 design method and the simplified method. Thus, it can be concluded that using the remaining life concept, makes the overlay thickness design much more sensitive to the subgrade support value.

Since the stress factors used in the RPOD1 program (Ref 1) were derived using discrete element theory, as well as Westergaard and Pickett theory, and field measured deflections, it is felt that the predictions by the RPOD1 program can be used with confidence. The thickness of the subgrade layer should be recognized in design if a stiff layer occurs at a depth of less than 12 foot (3.7-m).

SUMMARY OF STUDY ON RPOD1

In summary, it can be said that the RPOD1 program, which is used for the fatigue cracking analysis in this overlay design procedure, is a sound program, based on the most up-to-date theories and experience. Some modifications have been made, however, for use in the Texas method.

The findings of this evaluation study of RPOD1 are as follows:

(1) In general, the most important input variables are design deflection, elastic moduli and thicknesses of the various layers.

(2) In the analysis by Nayak et al. (Ref 7) the elastic modulus of concrete has been correlated with the flexural strength, which suggests that the flexural strength is also an important variable.

(3) It is desirable to use fixed Poisson's ratio values for different pavement materials, rather than determining it for each individual project.

(4) This study indicates, in general, that RPOD1 results compare very well with manual calculations using ELSYM5 to determine stresses, strains and deflections.

(5) For existing pavements with class 3 and 4 cracking or for mechanically broken up pavements, an effective modulus is being used. This eliminates the necessity for field determination of elastic modulus of the existing pavement concrete.

(6) For pavements with remaining life, overlay thickness is sensitive to the subgrade resilient modulus. It is also sensitive to the stress sensitivity of the subgrade soil, if the deflection load differs from the design load. Overlays on pavements with class 3 and 4 cracking or on mechanically broken up pavements, on the other hand, are relatively insensitive to subgrade modulus, which indicates that less effort is necessary in determining the subgrade modulus for these classes of existing pavements.

(7) A practical range for S_{SG} of 0 to -1.2 has been determined from field data. With the information available, S_{SG} could not be correlated to soil type.

(8) For pavements with remaining life, the percentage of remaining life has a great effect on overlay thickness. The percentage of remaining life is directly determined from the traffic prior to overlay, which indicates that this information should be as accurate as practically possible. For pavements with class 3 and 4 cracking and for mechanically broken up pavements, this information is not needed.

(9) It has been illustrated that on rigid pavements all four of the wheels on a standard single axle could have an effect on the overlay thickness because of the large deflection basin. In RPOD1 the two loads used as design loads corresponded to those used in developing the fatigue equation, so that reasonable results could be expected.

(10) Under certain conditions the thickness predicted taking the remaining life of the existing pavement into account could be greater than when the existing pavement is considered not to have remaining life. For this reason RPOD1 considers pavements with less than 12 percent remaining life as not having remaining life. Here a modification has been made to RPOD1, which is discussed in the next section.

(11) The fact that the decrease in subgrade stress level due to the overlay is not taken into account in RPOD1 leads to a somewhat conservative design. It is, however, believed that this simplification is reasonable.

(12) There is a need to develop a design procedure for asphaltic concrete overlays on portland cement concrete pavements without remaining life. These overlay designs have not been fully implemented in RPOD1. The procedure used in the Texas method is discussed in the next section.

(13) In comparing RPOD1 with a simplified method using Westergaard equations instead of layered theory, both methods indicated sensitivity of the overlay thickness to percentage of remaining life and subgrade modulus, for pavements with remaining life. For pavements without remaining life both methods indicated the subgrade support value to be relatively unimportant.

MODIFICATIONS TO THE FATIGUE CRACKING PROGRAM

Certain modifications made to the RPOD1 program for the Texas SDHPT procedure are discussed in this section. The modified program is called RPOD2.

Modification In Calculation Of Overlay Thicknesses For Pavements With 1 to 25 Percent Remaining Life

Under certain circumstances, as indicated on Fig II-2.8, it is possible that taking the remaining life of the existing pavement into account can result in a greater predicted overlay thickness than when the pavement is not considered to have remaining life. The reason for this can be seen by studying the remaining life and fatigue equations (Eqs I-3.5 and I-3.3). To predict the design life of the overlay, the following equation can be written:

$$N_o = RL \times N_p \qquad \text{(II-2.4)}$$

where

N_o = design life of the overlaid pavement system,

RL = remaining life of the existing pavement,

N_p = allowable number of stress applications determined out of the fatigue equation, using the horizontal tensile stress at the bottom of the existing pavement after overlay.

It will be noted that if RL decreases, N_p must increase in order to give the desired design life after overlay. For N_p to increase, the stress at the bottom of the overlay must be reduced, which will result in a thicker overlay. For very low percentages of remaining life, the required overlay thickness could become very large using this concept. For the Texas method, in the range of 1 to 25 percent remaining life, overlay thicknesses are computed and printed out for both the remaining life and the no remaining life cases. It is recommended that the more economical of the two thicknesses be used. Below one percent remaining life, the pavement is analyzed as if it has no remaining life.

Modification To Facilitate the Specification Of Both the Overlay and Existing Pavement Concrete Flexural Strengths

In RPOD1, it is only possible to input one flexural strength for concrete and this value is then used in the fatigue equation. This would not pose a problem for pavements with remaining life, in which case the flexural strength of the existing pavement concrete would be specified since the governing stress is at the bottom of the existing pavement. Likewise, for pavements with class 3 and 4 cracking or for mechanically broken up pavements, the flexural strength of the overlay concrete could be used as an input. For uncracked pavements with remaining life in the range of 1 to 25 percent both the flexural strength of the existing pavement concrete and that of the overlay material is needed (if they are significantly different). The program has been modified in such a way that in RPOD2 both the existing pavement and overlay concrete flexural strength values must be specified.

The Fatigue Cracking Analysis for Asphaltic Concrete Overlays on Rigid Pavements With No Remaining Life

As previously mentioned, the fatigue cracking analysis of asphalt concrete overlays on rigid pavements with no remaining life has not been implemented in RPOD1. To overcome the problems mentioned in the previous section, and illustrated in Fig II-2.21, the following procedure is being used in RPOD2:

(1) Determine the existing pavement structure as usual.

(2) Characterize the subgrade material as before using laboratory data and deflection measurements (see Chapter I-3).

(3) Determine the surface deflection of the existing pavement under the design load, using layered theory.

(4) Determine the modulus of a semi-infinite halfspace that would result in the same deflection as determined under 3 above.

(5) Design overlay thickness on this semi-infinite halfspace to keep the horizontal tensile stress at the bottom of the overlay within tolerance, using fatigue concepts.

For uncracked pavements or pavements that exhibit class 1 and 2 cracking but have no remaining life, the characterization of the subgrade material is to be done using the original modulus of the existing layer. For pavements with class 3 and 4 cracking and for mechanically broken up pavements, the subgrade material should be characterized using the effective modulus of the existing pavement [500,000 psi (3447 MPa) and 70,000 psi (483 MPa), respectively]. In determining the deflection under the design load (step 3 above), the effective modulus of the existing pavement should be used.

In order to gain confidence in this approach, a comparison study has been done to compare thicknesses for CRCP overlays calculated using RPOD1 with asphaltic concrete thicknesses calculated using this procedure and with asphaltic concrete thicknesses calculated using the AASHTO Interim Guide (Ref 9). The existing pavement structure used in this study is as indicated in Fig II-2.1. A subgrade modulus of 5000 psi (34.5 MPa) has been used. Thicknesses calculated are given in Table II-2.10.

This method seems to be a reasonable approach. The stress sensitivity of the subgrade material is still taken into account and the problem of modeling cracked layers with layered theory has been overcome. Table II-2.10 indicates that overlay thicknesses predicted by RPOD2 are reasonable in comparison with those predicted using the AASHTO Interim Guide and in comparison with predicted CRCP overlay thicknesses.

Maximum Limit On Subbase Modulus For Pavements With Class 3 and 4 Cracking and For Mechanically Broken Up Pavements

Since it is unlikely that the subbase of a pavement that has been mechanically broken up would still be intact, an upper limit has been set in the RPOD2 program on the modulus of the subbase. This maximum limit is the effective modulus of the existing pavement, which is in this case 70,000 psi (483 MPa). Likewise, for pavements with class 3 and 4 cracking an upper limit for subbase modulus of 500,000 psi (3447 MPa) has been used.

Default Value For Deflection Loads

Since the Dynaflect is the device most frequently used in Texas for deflection measurements, "Dynaflect loads" has been used as a default value in the RPOD2 program. This makes it unnecessary for the user of the program to specify the loads. Dynaflect loads used are two 500-pound (2.2-kN) loads 20 inches (508-mm) apart, with the position of deflection measurement between the two loads. The load pressure for Dynaflect loads is 167 psi (743 MPa) (Ref 6).

Alternative Method To Specify Stress Sensitivity Of Subgrade Material

Since it is possible to estimate S_{SG} by using two different deflection loads, as indicated in Appendix 4, an alternative way to input the resilient

TABLE II-2.10. COMPARISON OF OVERLAY THICKNESSES CALCULATED BY DIFFERENT METHODS

Method of Analysis / Type of Overlay / Pavement Condition	RPOD2	RPOD1	AASHTO
	AC	CRCP	AC
30% remaining life	8.8"	4.4"	7.9"
Class 3 & 4 cracking	9.7"	6.6"	8.9"
Mechanically broken up	11.6"	11.2"	11.9"

modulus versus deviator stress relationship has been provided. For RPOD2 it is possible to input S_{SG} directly should it be determined in some other way than resilient modulus testing in the laboratory.

CHAPTER II-3. REFLECTION CRACKING ANALYSIS (RFLCR1)

As pointed out in Part I, the RFLCR1 computer program, which is used for the reflection cracking analysis in this design procedure, fulfills a need that has been in existence for a long time. It attempts to design overlays against reflection cracking, or at least analyze for the possible occurrence of reflection cracking. This analysis procedure also provides the designer with theoretical evidence on the effectiveness of bondbreakers and/or interlayers he might consider in his design. Previously, decisions of this nature have been made on experience and engineering judgement (Ref 1). For more information on the theoretical background to the RFLCR1 program, the reader is referred to Part I and the work by Treybig et al. (Refs 1 and 6).

An evaluation of the RFLCR1 computer program has been attempted by means of a limited sensitivity analysis, which is given here.

SENSITIVITY ANALYSIS ON THE RFLCR1 COMPUTER PROGRAM

An extensive sensitivity analysis, such as conducted on the RPOD1 computer program by Nayak et al. (Ref 7) is beyond the scope of this study. Cost and time limitations would make such a study prohibitive. A limited sensitivity analysis has, however, been conducted on the RFLCR1 program to establish reasonableness of solutions and relative importance of input variables. The objectives of this sensitivity study were

(1) to evaluate the RFLCR1 computer program in order to adapt it for use by the Texas SDHPT,

(2) to establish confidence in the reliability of the model,

(3) to obtain an indication of the relative importance of the different input variables into the program, and

(4) to assist the designer in determining the relative amount of time and effort he should spend in determining the different input variables.

EXPERIMENT DESIGN

A full multiple factorial experiment is one in which response values are determined by combining all levels of each variable with all levels of every other variable (Ref 7). A fractional factorial experiment requires only a part of the full factorial observations. In a single factorial experiment all variables are kept constant at a certain level and the response values for several levels of one selected variable are taken. Another variable is then chosen and the process continued until all variables have been considered.

Work by Sutaria and Hudson (Ref 21) and Nayak et al. (Ref 7) indicates clearly that the better procedure for conducting a sensitivity analysis is to use a multiple factorial experiment design. Limitations to the single factorial experiment are:

(1) In the multiple factorial design, the effect of a design variable is estimated at more than one level of the other variables, and the conclusions are more reliable than in single factorial experiments.

(2) The rankings of variables obtained from a single factorial experiment are not absolute rankings.

(3) Ignoring interactions (as in a single factorial experiment) might result in misleading conclusions.

Bearing in mind the limitations to a single factorial sensitivity analysis, but also, on the other hand, considering the fact that full, or even fractional, factorial experiments would become prohibitive time as well as cost wise, it has been decided to conduct only a limited sensitivity analysis based on single factorial experiments. It is felt that the objectives, as outlined above, could be met reasonably well in this way. Some indication of the relative importance of variables will be obtained, although the rankings might not be absolute rankings. In using the results of this sensitivity study the limitations will be borne in mind.

Figure I-3.2 indicated that as many as twenty different analyses could be conducted using the RFLCR1 program. In this study, the number of different analyses was limited to two, as follows:

(1) an asphaltic concrete overlay on an uncracked JC existing pavement with a bondbreaker and without any overlay reinforcement, and

(2) an asphaltic concrete overlay on a cracked CRC existing pavement without a bondbreaker or overlay reinforcement.

Figure II-3.1 indicates the two analyses investigated, namely 14 and 20. Low medium and high numerical values for the different input variables were determined; and in these single factorial experiments all variables except one were set at medium values and the response values for high and low levels of the selected variable were determined. The next variable was then chosen and the process repeated until all variables had been considered. The effect of the independent variable was determined from the difference in response between the low and high value of that variable.

PROCEDURE FOR SELECTION OF NUMERICAL VALUES FOR INPUT VARIABLES

There are several bases for selection of numerical values for the independent variables in a sensitivity analysis (Refs 7 and 21).

Unit Change

In this case, each variable is changed by one unit, say one inch, one millimeter, etc., and the effect of this change on the response (dependent variable) determined. It is clear that a change of one inch, for instance, in the thickness of a pavement will have a much larger effect on the response than a change of one millimeter. This method of "unit change" will not give meaningful results if used in a sensitivity analysis.

Range

A range is the absolute difference between the largest and the smallest values of the independent variable (Ref 21). Selecting a range is a complicated problem and it is arbitrary. The results of the sensitivity analysis are, however, greatly effected by the range selected. It would, therefore, be undesirable to use such an arbitrary method.

Standard Deviation

There are many uncertainties associated with pavement design, and it is necessary to consider the stochastic nature of many of these variables.

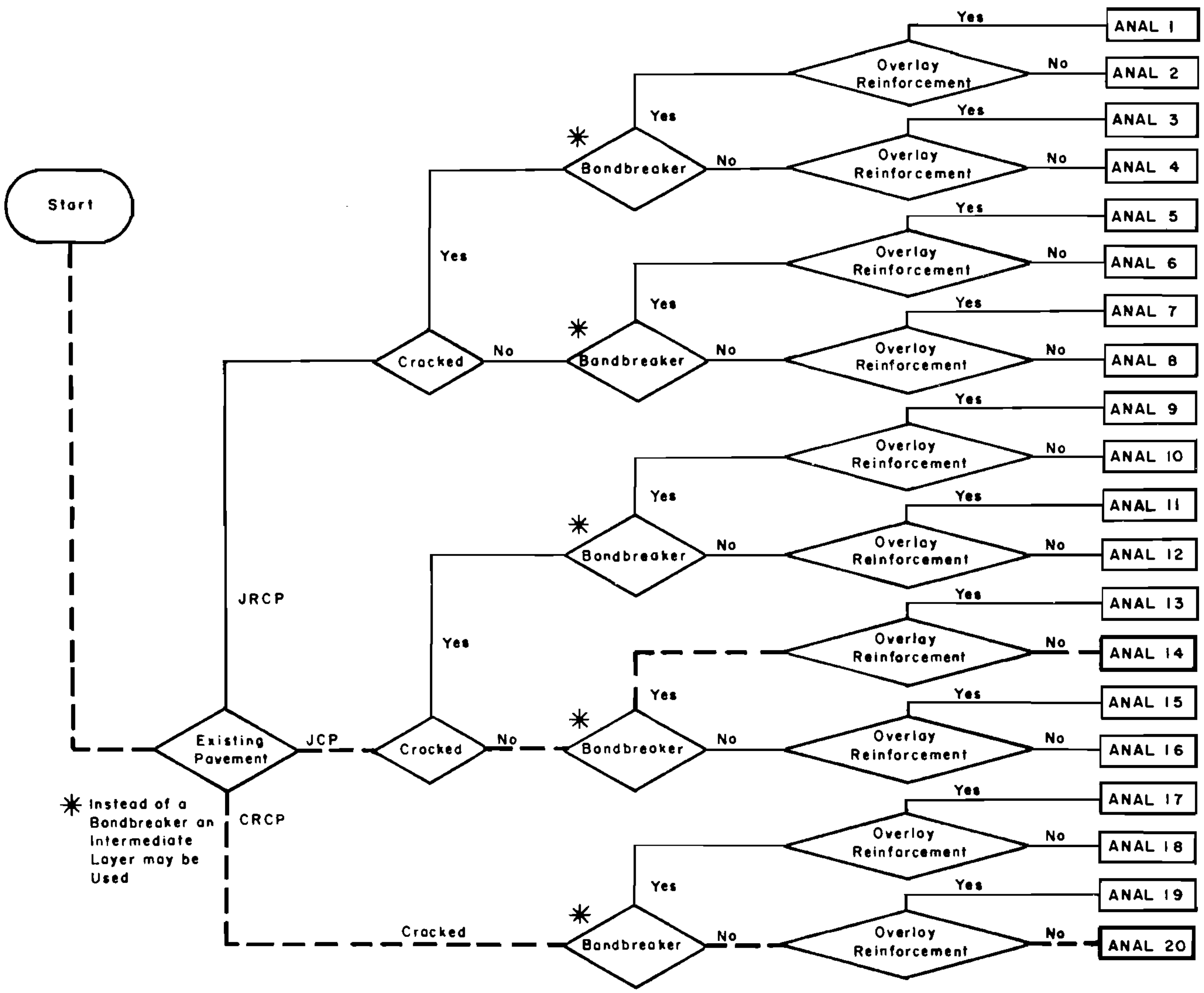


Fig II-3.1. Analysis system for prevention or minimizing reflection cracking.

It is, therefore, meaningful to select standard deviation, which is a measure of variation of the individual observations about their own arithmetic mean, as the unit for the sensitivity study (Ref 21), rather than any of the other two methods mentioned above.

The standard deviation represents the smallest change in a variable that can be measured, or controlled, with confidence in practice. It is calculated as follows:

$$\sigma = \sqrt{\frac{\Sigma(x_i - \bar{x})^2}{n}} \qquad \text{(II-3.1)}$$

where

σ = standard deviation,

$\bar{x}$ = arithmetic mean of observations,

x_i = individually observed value, and

n = number of observations.

There are two types of variability associated with pavement design variables (Ref 21):

(1) within-project variability, which is associated with the variations about their means of input parameters within the same pavement section, and

(2) between-project variability, which is the variability between assumed design average values and those actually constructed.

The total variation which is necessary for a sensitivity analysis, can be calculated as follows:

$$[\text{total standard deviation}]^2 = [\text{within-project standard deviation}]^2 + [\text{between-project standard deviation}]^2 \qquad \text{(II-3.2)}$$

In this study it was decided to use basically the "standard deviation" method in selecting numerical values for the input variables. One standard deviation on the positive side and one on the negative side of the mean values of the independent variables have been used to determine high and low level values of the independent variables, as follows:

$$x_{iL} = \bar{x}_i - \sigma_{x_i} \tag{II-3.3}$$

and

$$x_{iH} = \bar{x}_i + \sigma_{x_i} \tag{II-3.4}$$

where

x_{iL} = low level of independent variable,

x_{iH} = high level of independent variable,

$\bar{x}_i$ = mean value of the independent variable, and

σ_{x_i} = total standard deviation of independent variable.

MEAN VALUES AND STANDARD DEVIATIONS OF INPUT VARIABLES

Table I-3.3 in Chapter I-3 lists all the input variables for the RFLCR1 program. Those variables not applicable to a specific problem are not required; for instance, if no bondbreaker is to be used, the bondbreaker information is deleted from the list.

Mean values and standard deviations have been determined for those variables considered in this sensitivity analysis. Details on the determination of these values are given in Appendix 5. All variables used for the JC existing pavement as well as the CRC existing pavement are listed in Table II-3.1, and mean values, standard deviations, and low and high levels of variables are also indicated on the same table.

Analysis

The sensitivity analysis has been conducted by using the values for the input variables listed on Table II-3.1 for analysis 14 (JC existing pavement) and for analysis 20 (CRC existing pavement)

For each case studied, all variables have been put at medium level values and variables have been varied one at a time from their low levels to their high level values with all other variables at medium level. The effect of each variable has been determined, as previously discussed, by subtracting the RFLCR1 response (horizontal tensile strain and vertical shear strain in the overlay) for the high level of the variable from the response for the low value of that variable. Effects of variables have heen listed on Table II-3.2 for analysis 14 (JC existing pavement) and on Table II-3.3 for analysis 20 (CRC existing pavement) for

(1) horizontal tensile strain in the overlay, and
(2) vertical shear strain in the overlay.

These tables also summarize the RFLCR1 response values for each variable at its low level as well as at its high level. The variables have also been ranked according to their relative effects on the RFLCR1 responses. It should be noted that these rankings are not absolute rankings as previously indicated, since interactions have not been considered here. These rankings will also be very much dependent on the low, medium and high level values selected for the different variables.

In the case of the CRC existing pavement (analysis 20), it is also interesting to study the effect of the variables on the concrete stress as well as on the steel stress before overlay. These effects can be seen in Table II-3.4.

The results of the sensitivity analysis are presented in graphical form in Figs II-3.2 to II-3.5. Figures II-3.2 and II-3.4 show differences in horizontal tensile strain if the variables, for analyses 14 and 20

TABLE II-3.1. INPUTS FOR SENSITIVITY ANALYSIS

Variable Number	Layer	Variable	Mean Value $\bar{x}_i$	Total Standard Deviation σ_i	Low Value of Variable x_{iL}	High Value of Variable x_{iH}
Input Common to Both Analyses 14 and 20 (see Fig II-3.1)						
1	Existing Pavement	Elastic modulus (psi)	4.6×10^6	0.4×10^6	4.2×10^6	5×10^6
2	Existing Pavement	Thermal coefficient (in./in./°F)	5.2×10^{-6}	1.4×10^{-6}	3.8×10^{-6}	6.6×10^{-6}
3	Existing Pavement	Thickness (inches)	8.0	0.5	7.5	8.5
4	Existing Pavement	Density (pcf)	140.0	4.0	136.0	144.0
5	Existing Pavement	Movement at sliding (inches)	.135	.115	.02	.25
6	Existing Pavement	Minimum temperature observed since construction, °F	5.5	5.5	0	11
7	Existing Pavement	Load transfer (percent/100)	.8	.15	.65	.95
8	Existing Pavement	Design temperature change °F	94	4	90	98
9	Overlay	Creep modulus (psi)	320,000	180,000	140,000	500,000
10	Overlay	Thermal coefficient (in./in./°F)	1.2×10^{-5}	Not Varied	1.2×10^{-5}	1.2×10^{-5}
11	Overlay	Thickness (inches)	8	0.5	7.5	8.5
12	Overlay	Density (pcf)	136	7.5	128.5	143.5
13	Overlay	Poisson's Ratio	0.3	0.05	0.25	0.35
14	Overlay	Dynamic modulus (psi)	6.75×10^6	2.25×10^{-6}	4.5×10^6	9×10^6
15	Overlay	Overlay to existing surface bonding stress (psi)	850	350	500	1200
16	Overlay	Design temperature change °F	105	5	100	110
17	--	Design load weight (pounds)	18,000	2,000	16,000	20,000
18	--	Width of design load (inches)	24	4	20	28
Additional Input for Analysis 14 (see Fig II-3.1)						
1	Existing Pavement	Joint spacing (feet)	13.5	1.5	12.0	15.0
2	Existing Pavement	Change in joint width for temperature change from 80°F to 70°F (inches)	3.5×10^{-3}	1.5×10^{-3}	2×10^{-3}	5×10^{-3}
3	Existing Pavement	Mean joint width (inches)	.04	.01	.03	.05
4	Bondbreaker	Width of bondbreaker (feet)	1.0	0.5	0.5	1.5
Additional Input for Analysis 20 (see Fig II-3.1)						
1	Existing Pavement	*Crack spacing	6 feet	2 feet	4 feet	8 feet
2	Existing Pavement	*Change in crack width for temperature change from 80°F to 70°F (inches)	3.2×10^{-3}	1.95×10^{-3}	1.3×10^{-3}	5.2×10^{-6}
3	Existing Pavement	Mean crack width (inches)	.018	.01	.008	.028
4	Existing Pavement	Elastic modulus of steel (psi)	29×10^6	Not Varied	29×10^6	29×10^6
5	Existing Pavement	Steel thermal coefficient (in./in./°F)	5.75×10^{-6}	$.75 \times 10^{-6}$	5×10^{-6}	6.5×10^{-6}
6	Existing Pavement	Area of steel/foot-width (in.^{2})	.508	.073	.435	.581
7	Existing Pavement	Perimeter of steel (in./ft width)	3.49	1.17	2.32	4.65
8	Existing Pavement	Steel to concrete bonding stress (psi)	260	90	170	350

1 in./in./°F = 1.8 mm/mm/°C
°C = (°F − 32) x 5/9
1 inch = 2.54 mm,
1 psi = 6.894 KPa

TABLE II-3.2. SUMMARY OF SINGLE FACTORIAL EXPERIMENT OF AC OVERLAY ON UNCRACKED JCP WITH BONDBREAKER

Number	Layer	Variable	Horizontal Tensile Strain (x 10^{-3} in./in.) Strain at Low Value of Variable	Strain at High Value of Variable	Effect	Rank of Variables	Vertical Shear Strain (x 10^{-6} in./in.) Strain at Low Value of Variable	Strain at High Value of Variable	Effect	Rank of Variables
1	Existing Pavement	Elastic modulus	2.405	2.431	+0.026	10	7.222	7.222	0	
2	Existing Pavement	Thermal coefficient	2.369	2.450	+0.081	6	7.222	7.222	0	
3	Existing Pavement	Thickness	2.409	2.428	+0.019	11	7.222	7.222	0	
4	Existing Pavement	Density	2.419	2.419	0		7.222	7.222	0	
5	Existing Pavement	Joint spacing	2.398	2.436	+0.038	9	7.222	7.222	0	
6	Existing Pavement	Movement at sliding	2.419	2.419	0		7.222	7.222	0	
7	Existing Pavement	Change in joint width from 80°F to 70°F	2.206	2.674	+0.468	3	7.222	7.222	0	
8	Existing Pavement	Minimum temperature observed since construction	2.419	2.419	0		7.222	7.222	0	
9	Existing Pavement	Mean joint width	2.419	2.419	0		7.222	7.222	0	
10	Existing Pavement	Load transfer	2.419	2.419	0		12.64	1.806	-10.834	1
11	Existing Pavement	Design temperature change	2.384	2.454	+0.070	7	7.222	7.222	0	
12	Overlay	Creep modulus	2.830	2.209	-0.621	1	7.222	7.222	0	
13	Overlay	Thickness	2.451	2.389	-0.062	8	7.704	6.797	-0.907	5
14	Overlay	Density	2.419	2.419	0		7.222	7.222	0	
15	Overlay	Poisson's Ratio	2.419	2.419	0		6.944	7.500	+0.556	6
16	Overlay	Dynamic modulus	2.419	2.419	0		10.83	5.417	-5.413	2
17	Overlay	Overlay to existing surface bonding stress	2.233	2.533	+0.300	4	7.222	7.222	0	
18	Overlay	Design temperature change	2.359	2.479	+0.120	5	7.222	7.222	0	
19	Bondbreaker	Bondbreaker width	2.709	2.200	-0.509	2	7.222	7.222	0	
20	--	Design load weight	2.419	2.419	0		6.420	8.025	+1.605	4
21	--	Design load width	2.419	2.419	0		8.667	6.190	-2.477	3
All Variables at Medium Values				2.419				7.222		

TABLE II-3.3. SUMMARY OF SINGLE FACTORIAL EXPERIMENT OF AC OVERLAY ON CRACKED CRCP WITHOUT BONDBREAKER

Number	Layer	Variable	Horizontal Tensile Strain ($\times 10^{-3}$ in./in.) Strain at Low Value of Variable	Strain at High Value of Variable	Effect	Rank of Variables	Vertical Shear Strain ($\times 10^{-6}$ in./in.) Strain at Low Value of Variable	Strain at High Value of Variable	Effect	Rank of Variables
1	Existing Pavement	Elastic modulus	2.869	2.899	+ .030	8	7.222	7.222	0	
2	Existing Pavement	*Thermal coefficient *Crack spacing *Change in crack width for temperature change from 80°F to 70°F	2.316	3.344	+1.028	2	7.222	7.222	0	
3	Existing Pavement	Thickness	2.884	2.896	+0.012	9	7.222	7.222	0	
4	Existing Pavement	Density	2.891	2.891	0		7.222	7.222	0	
5	Existing Pavement	Movement at sliding	2.891	2.891	0		7.222	7.222	0	
6	Existing Pavement	Minimum temperature observed since construction	2.889	2.893	+ .004	10	7.222	7.222	0	
7	Existing Pavement	Mean joint width	2.891	2.891	0		7.222	7.222	0	
8	Existing Pavement	Load transfer	2.891	2.891	0		12.64	1.806	-10.83	1
9	Existing Pavement	Design temperature change	2.858	2.923	+ .065	6	7.222	7.222	0	
10	Existing Pavement	Thermal coefficient	2.891	2.891	0		7.222	7.222	0	
11	Existing Pavement	Area of steel/foot width	2.890	2.891	+ .001	11	7.222	7.222	0	
12	Existing Pavement	Perimeter of steel	2.873	2.905	+ .032	7	7.222	7.222	0	
13	Existing Pavement	Steel to concrete bonding stress	2.876	2.906	+ .030	8	7.222	7.222	0	
14	Overlay	Creep modulus	4.001	2.464	-1.537	1	7.222	7.222	0	
15	Overlay	Thickness	2.965	2.824	-0.141	4	7.704	6.797	-.907	5
16	Overlay	Density	2.891	2.891	0		7.222	7.222	0	
17	Overlay	Poisson's Ratio	2.891	2.891	0		6.944	7.500	+.556	6
18	Overlay	Dynamic modulus	2.891	2.891	0		10.830	5.417	-5.413	2
19	Overlay	Overlay to existing surface bonding stress	2.461	3.202	+ .741	3	7.222	7.222	0	
20	Overlay	Design temperature change	2.831	2.951	+ .120	5	7.222	7.222	0	
21	--	Design load weight	2.891	2.891	0		6.420	8.025	+1.605	4
22	--	Design load width	2.891	2.891	0		8.667	6.190	-2.471	3
All Variables at Medium Values				2.891				7.222		
With Friction Curve Switch > 0 (for non-plastic soils) (Refs 5 and 6)				2.838				7.222		

*Variables varied together

TABLE II-4.4. SUMMARY OF EFFECT OF VARIABLES ON CONCRETE AND STEEL STRESS IN EXISTING CRCP BEFORE OVERLAY

Number	Layer	Variable	Concrete Stress Before Overlay psi			Steel Stress Before Overlay psi		
			Low Value of Variable	High Value of Variable	Effect	Low Value of Variable	High Value of Variable	Effect
1	Existing Pavement	Elastic Modulus	320	365	+45	36983	36927	-56
2	Existing Pavement	*Thermal coefficient *Crack spacing *Change in crack width for temperature change from 80°F to 70°F	462	475	+13	32970	39490	+6520
3	Existing Pavement	Thickness	348	338	-10	36974	36933	-41
4	Existing Pavement	Density	343	343	0	36953	36953	0
5	Existing Pavement	Movement at sliding	343	343	0	36953	36953	0
6	Existing Pavement	Minimum temperature observed since construction	360	325	-35	37795	36098	-1697
7	Existing Pavement	Mean joint width	343	343	0	36953	36953	0
8	Existing Pavement	Load transfer	343	343	0	36953	36953	0
9	Existing Pavement	Design temperature change	343	343	0	36953	36953	0
10	Existing Pavement	Steel thermal coefficient	343	343	0	35333	38573	+3240
11	Existing Pavement	Area of steel/foot width	337	348	+11	39661	34776	-4885
12	Existing Pavement	Perimeter of steel	327	356	+29	30653	42237	+11584
13	Existing Pavement	Steel to concrete bonding stress	327	356	+29	30424	42441	+12017
14	Overlay	Creep modulus	343	343	0	36953	36953	0
15	Overlay	Thickness	343	343	0	36953	36953	0
16	Overlay	Density	343	343	0	36953	36953	0
17	Overlay	Poisson's ratio	343	343	0	36953	36953	0
18	Overlay	Dynamic modulus	343	343	0	36953	36953	0
19	Overlay	Overlay to existing surface bonding stress	343	343	0	36953	36953	0
20	Overlay	Design temperature change	343	343	0	36953	36953	0
21	Overlay	Design load weight	343	343	0	36953	36953	0
22	Overlay	Width of design load	343	343	0	36953	36953	0
	All Variables at Medium Values			343			36953	
	With Friction Curve Switch > 0 (for plastic soils)			343			36953	

*Variables varied together

1 psi = 6.894×10^{-3} MN/m^2

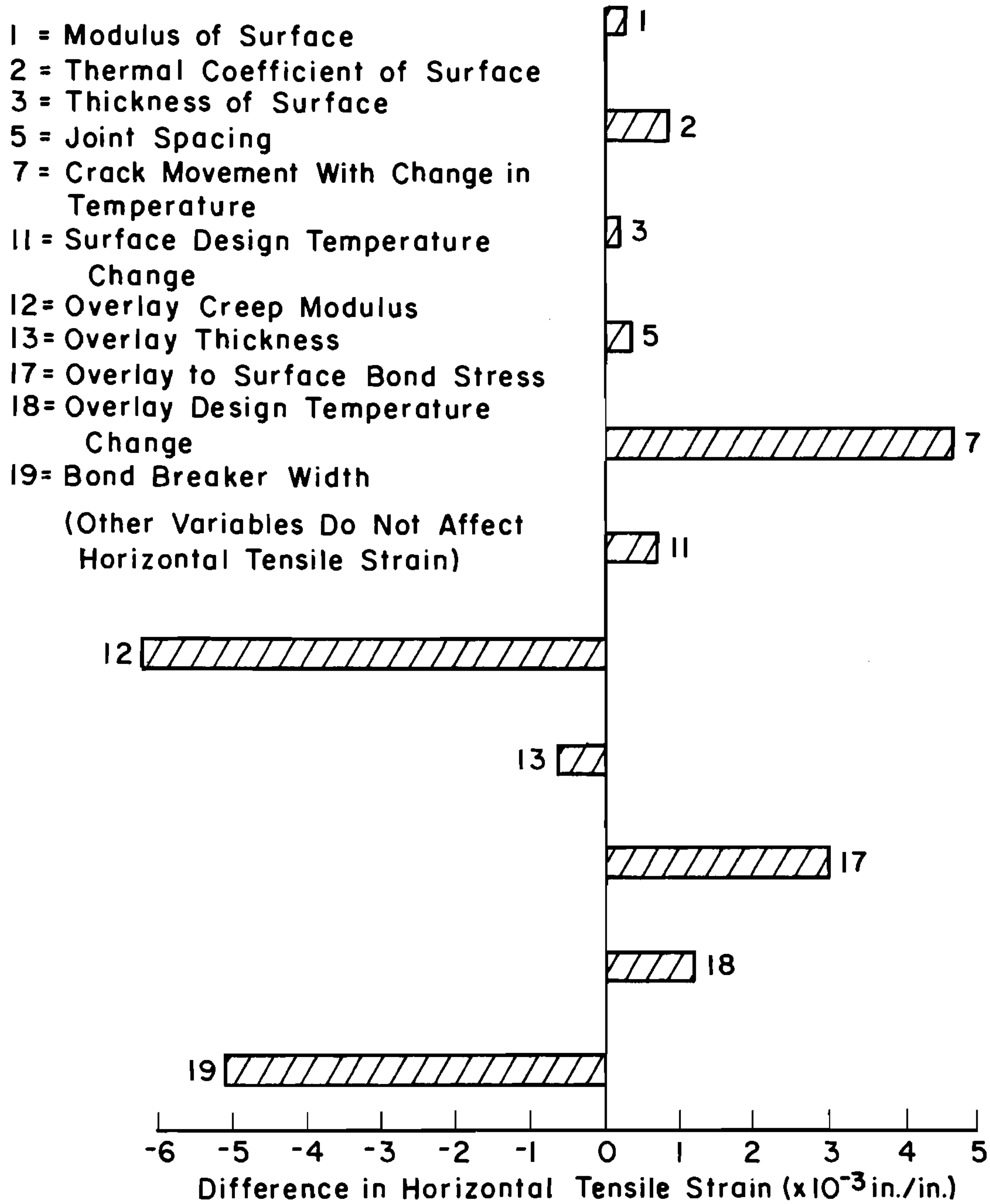


Fig II-3.2. Sensitivity study data illustrating change in horizontal tensile strain for $\pm$ one standard deviation of variable - AC on uncracked JCP with bondbreaker.

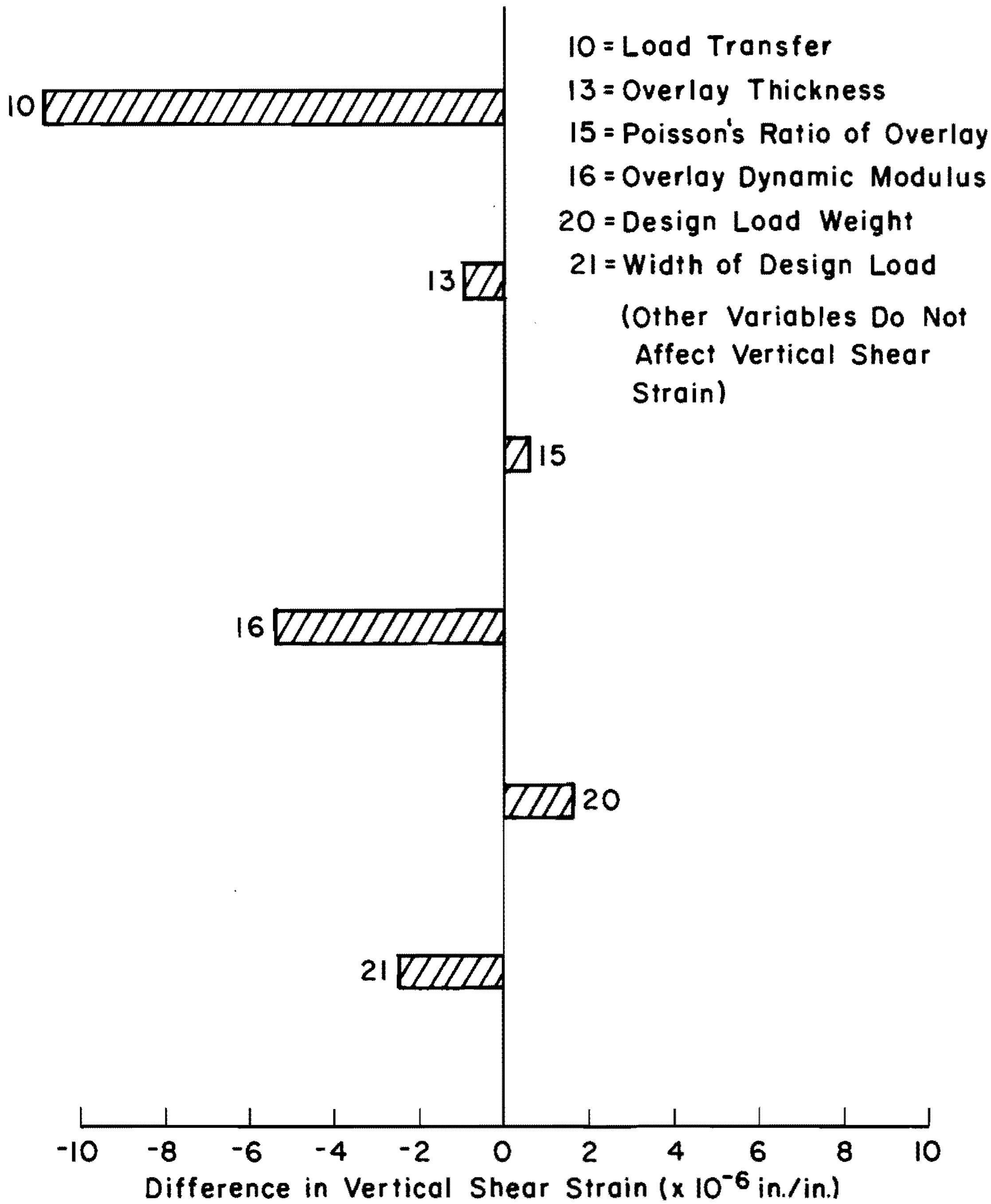


Fig II-3.3. Sensitivity study data illustrating change in vertical shear strain for ± one standard deviation of variable - AC on uncracked JCP with bondbreaker.

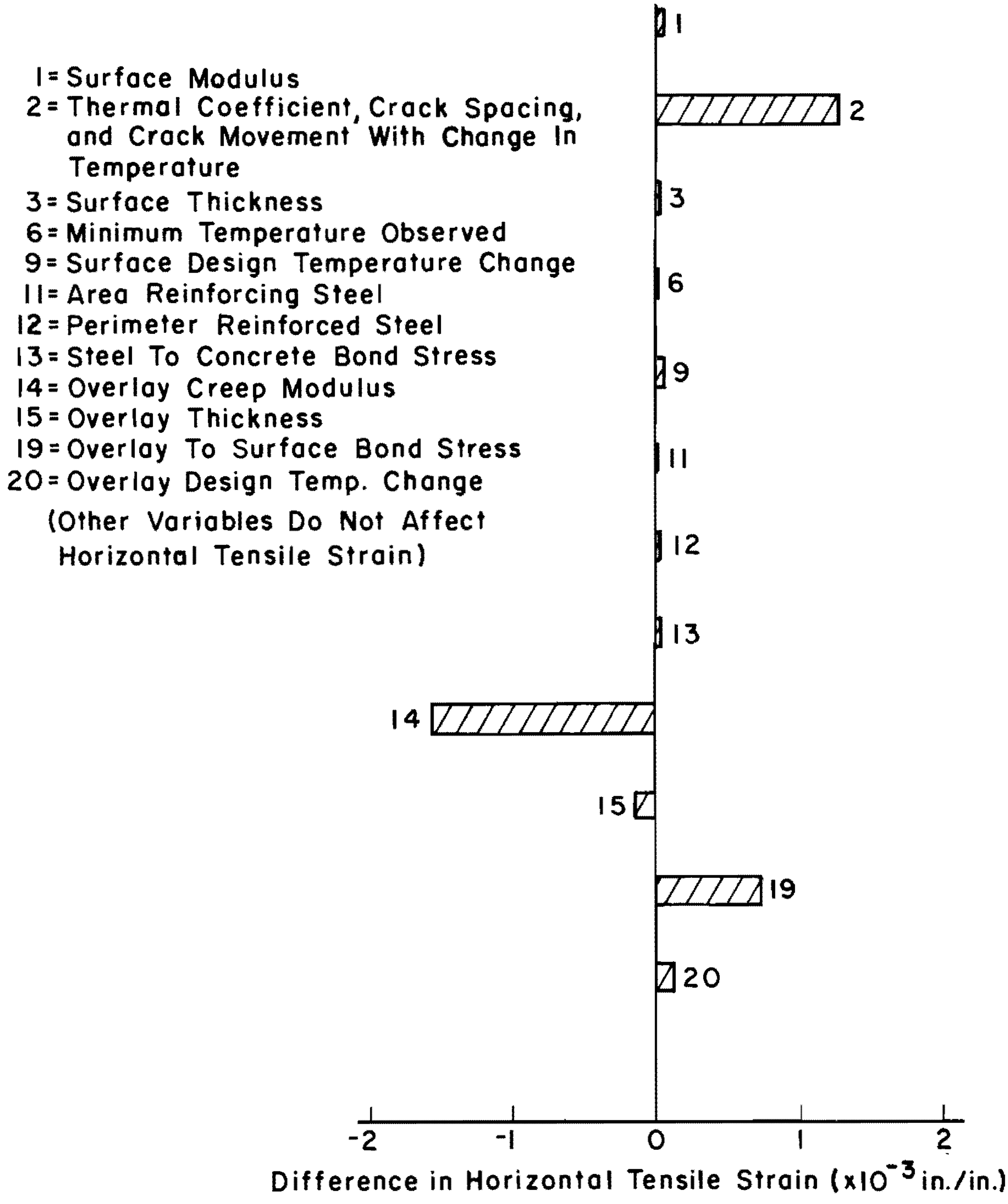


Fig II-3.4. Sensitivity study data illustrating change in horizontal tensile strain for $\pm$ one standard deviation of variable - AC on cracked CRCP without bondbreaker.

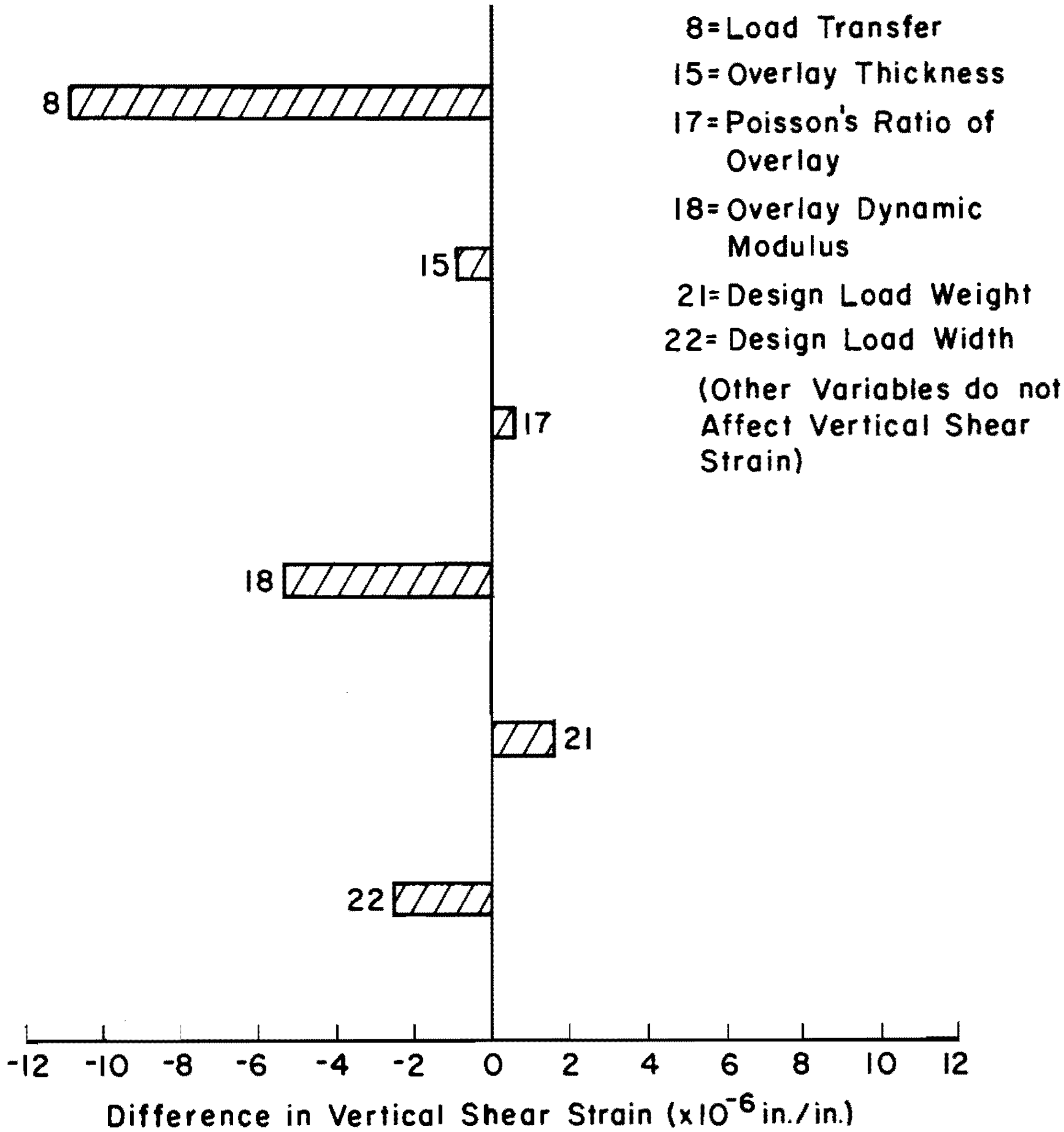


Fig II-3.5. Sensitivity study data illustrating change in vertical shear strain for $\pm$ one standard deviation of variable - AC on cracked CRCP without bondbreaker.

respectively, are varied by plus and minus one standard deviation from the variable mean value. Similarly the differences in vertical shear strain are indicated in Figs II-3.3 and II-3.5 for analyses 14 and 20 respectively.

DISCUSSION OF RESULTS

Results obtained from the analysis of reflection cracking for the pavement structures considered are discussed separately in this section.

AC Overlay On Uncracked JCP With Bond Breaker

In studying Table II-3.2 and Figs II-3.2 and II-3.3, it has been noted that 11 of the 21 input variables had an effect on the horizontal tensile strain in the overlay. They are, in descending order of importance,

(1) overlay creep modulus,
(2) width of bondbreaker
(3) change in joint width with change in temperature,
(4) overlay to existing surface bonding stress,
(5) overlay design temperature change,
(6) existing concrete thermal coefficient,
(7) existing pavement design temperature change,
(8) overlay thickness,
(9) joint spacing,
(10) concrete elastic modulus, and
(11) existing pavement thickness.

The following variables had an effect on the vertical shear strain:

(1) load transfer,
(2) overlay dynamic modulus,
(3) width of the design load,
(4) design load weight,
(5) overlay thickness, and
(6) overlay Poisson's ratio.

Only one variable, overlay thickness affected both the horizontal tensile strain and the vertical shear strain.

These results make sense since the horizontal tensile strain is expected to be affected by factors such as temperature changes, thermal coefficients, slab lengths, material properties, and overlay thickness. The creep modulus is used in this program in relation to horizontal tensile strains, which are caused by temperature movements, the rate of application of which are much longer than of traffic associated loads.

On the other hand, it could be expected that vertical, load associated strains will result from a lack of load transfer and from magnitude and width of applied loads. Elastic material properties of the overlay and the thickness of the overlay could also be expected to be important, which is indeed the case.

Some factors did not appear to have an effect at all. In this study, the subbase was considered to be a plastic soil, for which use of an adjusted friction curve which takes the increased overburden pressure into account is not recommended (Refs 1 and 6). This has been done by specifying a value of 0 for the friction curve switch (see RFLCR1 input guide). Variables such as the density of the existing pavement and the density of the overlay are used for calculation of this adjusted friction curve and, therefore, did not have any effect in this study.

The minimum temperature observed since construction is used in characterization of a reinforced pavement (Ref 5). This variable is not used for unreinforced pavements and did not have any effect in this case. The design temperature change, which is used in calculating the horizontal strain in the overlay, did show an effect, as already mentioned.

Movements at sliding did not appear to have any effect on the response. The reason for this is as follows. In this program it is assumed that the parabolic friction curve (Ref 1) can be estimated with a constant slope line (Fig II-3.6). The force f_F is the force at which sliding occurs. Below this limit, there is a linear relationship between force and displacement. If the specific parameters in this analysis have been selected in such a way that the displacement would never be greater than Y_s, sliding would not occur and the movement at sliding would not show an effect. In studying the input variables in Tables II-3.2 and II-3.3, it may be seen that this has been the case for both pavements studied.

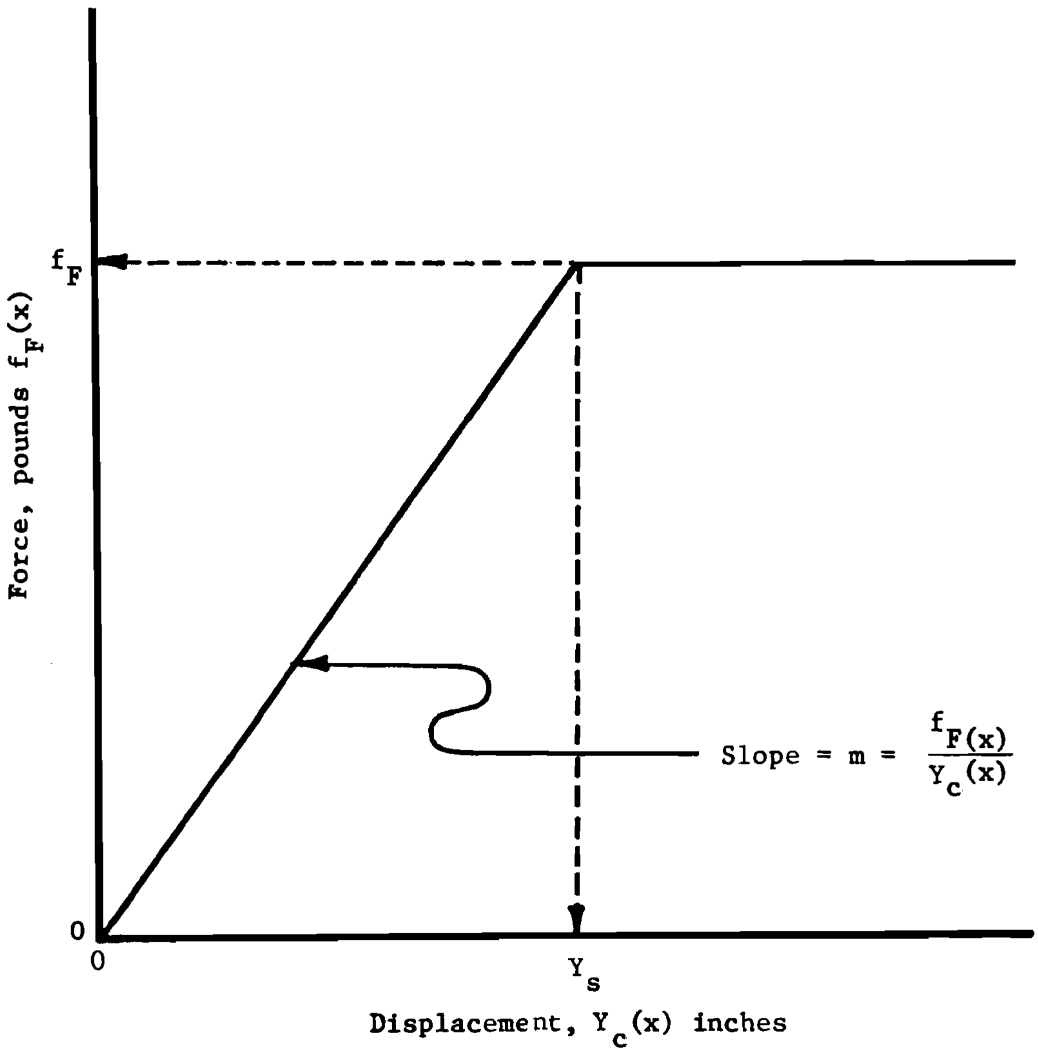


Fig II-3.6. Theoretical force-displacement relationship between concrete slab and underlaying layer assumed in the model (Ref 1).

The mean joint width did not show any effect on the RFLCR1 response. The bondbreaker did not affect the vertical shear strain in the overlay.

AC Overlay On Cracked CRCP Without Bondbreaker

Results of the study of AC overlay on cracked CRCP without a bondbreaker can be seen in Tables II-3.3 and II-3.4 and Figs II-3.4 and II-3.5.

As mentioned previously, it has been deemed necessary to vary three variables, concrete thermal coefficient, crack spacing, and change in crack width associated with a specific temperature change, together so that only the combined effect of these variables will be evaluated in this study.

Similarly to the study with the JC existing pavement, certain variables affected the horizontal tensile strain, certain variables the vertical shear strain, and certain variables did not have an effect on either one of the responses. Again, the thickness of the overlay was the only variable with an effect on both RFLCR1 responses.

Factors affecting the horizontal tensile strain, in descending order of importance are,

(1) overlay creep modulus,

(2) concrete thermal coefficient plus crack spacing and change in crack width with temperature change,

(3) overlay to existing surface bonding stress,

(4) overlay thickness,

(5) overlay design temperature change,

(6) existing pavement design temperature change,

(7) perimeter of reinforcing steel,

(8) steel to concrete bonding stress and existing pavement elastic modulus,

(9) existing pavement thickness,

(10) minimum temperature observed since construction, and

(11) area of reinforcing steel.

Vertical shear strain has been affected by

(1) load transfer,

(2) dynamic modulus of overlay,

(3) width of design load,

(4) magnitude of design load,

(5) overlay thickness, and

(6) Poisson's ratio of overlay.

As previously mentioned, the minimum temperature observed since construction is used in characterizing the reinforced pavement, and it did have a slight effect on the horizontal tensile strain in this case. In Table II-3.4 it will be noted that the minimum temperature observed affected the concrete, as well as steel stress, before overlay, significantly.

For the same reasons as mentioned for the previous case, the existing pavement density and the density of the overlay did not have an effect on the responses. In this case, the effect of specifying a non-plastic soil, with all variables at medium level, has been determined and can be seen in Table II-3.3. Specifying a non-plastic soil caused a decrease in horizontal tensile strain.

The mean joint spacing did not show an effect on the RFLCR1 response and, as previously discussed, the movement at sliding did not show any effect in this study either.

Steel reinforcement properties as well as the concrete thermal coefficient seem to be the most important factors affecting the steel stress before overlay.

DEVELOPMENT OF THE REFLECTION CRACKING ANALYSIS FOR TEXAS

The sensitivity analysis on the RFLCR1 program provided an opportunity to become acquainted with this program, to evaluate the reasonableness of its results, and to develop some sense for the relative importance of the various input variables.

No modifications to the RFLCR1 program are suggested here, and the program is recommended for use in the Texas SDHPT procedure as a very useful tool.

The overlay creep modulus, used in calculation of horizontal tensile strains, seems to be a very important variable, and consideration should be given to determining this material property directly rather than using nomographs as suggested in the FHWA method (Ref 1). This characterization should be done at the low temperature related to the design temperature change. The loading time should be in the order of 6 to 12 hours. There is presently no standard test for determination of creep modulus of asphalt concrete.

The vertical shear strain is load associated and, therefore, the temperature for determining the dynamic modulus of the overlay material should be the same as that used in the fatigue cracking analysis. This shear strain is, however, a repeated strain and should be considered as such in this procedure. Appendix 6 gives a suggested method for determining the maximum allowable value for this shear strain due to traffic loads.

CHAPTER II-4. SUMMARY AND RECOMMENDATIONS

This part of this report presents the development of the rigid pavement overlay design procedure for Texas SDHPT. In this process the FHWA method has been evaluated thoroughly. It was found to be a sound method based on the most up-to-date pavement design technology and was, therefore, used as the basis of the Texas SDHPT procedure. Some modifications have been made in the Texas procedure. Modifications to the fatigue cracking analysis are discussed in Chapter II-2. The revised computer program is called RPOD2. The study on the reflection cracking analysis program, RFLCR1, has been conducted mainly through a limited sensitivity analysis. The RFLCR1 program has been accepted, unmodified, for the Texas method. A tentative method to determine a maximum allowable value for repeated shear strain is, however, suggested in Chapter II-3.

Recommendations

From the studies outlined in this part of the report, it is recommended that the revised Texas SDHPT overlay procedure for rigid pavements be implemented for use in Texas as soon as possible. Further research suggestions are:

(1) It would be advisable to revise the fatigue equation used in the fatigue cracking analysis (RPOD2) by developing it for the four loads of the standard axle (see Fig II-2.5a). The design load in RPOD2 should then be revised accordingly. This would lead to a more accurate prediction of overlay thickness, and the fatigue equation would then also be useful outside the RPOD2 context.

(2) Since the load associated vertical shear strain in the reflection cracking program is repetitive in nature, it is assumed that it would cause fatigue of the asphaltic concrete. Although some tentative allowable shear strain values are suggested here, some more research is needed.

PART III

USER'S MANUAL FOR THE TEXAS SDHPT RIGID PAVEMENT OVERLAY DESIGN PROCEDURE

CHAPTER III-1. INTRODUCTION

This part of the report deals with the User's Manual prepared for the Rigid Pavement Overlay Design Procedure and is for the use of Texas State Department of Highways and Public Transportation. The User's Manual is based on FHWA report No. FHWA-RD-77-67 (Ref 6) and includes the changes to the RPOD1 computer program. It also incorporates some modifications to the input guides to meet the requirements of the Texas SDHPT.

This manual is intended to be a self-explanatory guide to the use of the procedure for thickness design of both rigid and flexible overlays on rigid pavements. All the elements necessary to enable the user to perform a thickness design analysis of overlays on rigid pavements are covered.

The basic concepts encompassed by this procedure are discussed in Chapter I-3 and modifications to the program RPOD1, which performs the fatigue cracking analysis, are dealt with in Chapter II-2. The modified program is called RPOD2.

This user's manual is divided into the following sections:

(1) evaluation of the existing pavement,

(2) fatigue cracking analysis,

(3) reflection cracking analysis, and

(4) selection of overlay thickness.

In the pavement evaluation process two computer programs, PLOT2 and TVAL2, are available for use.

The overlay thickness analysis is accomplished by using two computer programs;

(1) RPOD2 which performs the fatigue cracking analysis and

(2) RFLCR1 which performs the reflection cracking analysis.

In general, the input guides have been written so that certain values are "fixed" by the way the general input guide is set up, allowing the program to use a default value. For more specialized work or non-typical designs, the user is allowed to alter these "fixed" values by the use of a supplement to the input guide.

CHAPTER III-2. EVALUATION OF THE EXISTING PAVEMENT

The existing pavement is evaluated by a deflection survey and a condition survey. In the following sections, the procedures for each of these survey types are outlined.

DEFLECTION SURVEY

Deflection test results are used to divide the project into design sections that behave differently under loads and to characterize the subgrade material.

Equipment

This method is not limited to a specific deflection measuring device. Since the Dynaflect is frequently used by the Texas SDHPT, Dynaflect loadings have been fixed in the RPOD2 input guide, but a supplement to the input guide provides a means for inputting any other deflection loads. Appendix 4 gives some consideration to load measuring devices such as the Benkelman beam, Falling Weight Deflectometer and Dynaflect, each of which have certain advantages and limitations.

Testing Conditions

In this method, it is assumed that measurements are taken during the season of the year that yields maximum deflection. This manual offers no seasonal adjustment factors for converting measurements made in any other season to maximum deflections, but is is suggested that such correlations be developed in the future.

Sampling Procedure and Frequency of Testing

It is recommended that at least one deflection profile along the outer wheelpath be obtained for each roadway. For ease of traffic handling, it is desirable to take these profiles as close as possible to the outside wheelpaths but no closer than 3 feet (914-mm) to the edge of the road. The reason for this limitation is that the procedure requires interior deflections as an input, and work by Treybig et al. (Ref 1) indicated that, at 3 feet (914-mm) from the edge, the deflections would be very close to interior deflections. On each line, measurements should be taken at an average of 100 feet (30.5-m) apart but spaced in such a way that they are far enough from joints or cracks to represent an interior condition. The measurements of the two profiles should be staggered to provide data at 50-foot (15.25-m) intervals. For very uniform soil conditions on level terrain (few cut to fill transitions), the measurement spacing may be increased to 250 feet (76.2-m).

In addition to the interior deflection measurements, it is also necessary to make measurements of corner deflections on jointed pavements. These measurements are to be taken simultaneously with the interior deflections in order to save time and money but are to be kept separate from the profile measurements. The corner measurements are used to determine a corner to interior deflection ratio which is used for estimating the degree of load transfer. Figure III-2.1 is a plan view of a JCP devided highway indicating the deflection locations.

Deflection Profiles

Data obtained from interior deflection measurements are to be plotted in the form of deflection profiles. On undivided highways deflections of separate lanes should be combined. For divided highways, it is recommended that profiles be plotted separately and that each of the two roadways be designed to reflect its needs.

Plotting of Profiles

Profiles can be plotted manually or by using the computer program PLOT2 (Ref 6).

The PLOT2 program makes a printer plot of deflections vs. distance along the roadway. The deflection is represented by the Y-value of the graph and the distance by the X-value.

Appendix 7 contains an input guide for the PLOT 2 computer program.

The output of this program is a line print plot in which only the Y or horizontal axis is scaled. On the X-axis, the deflection locations are

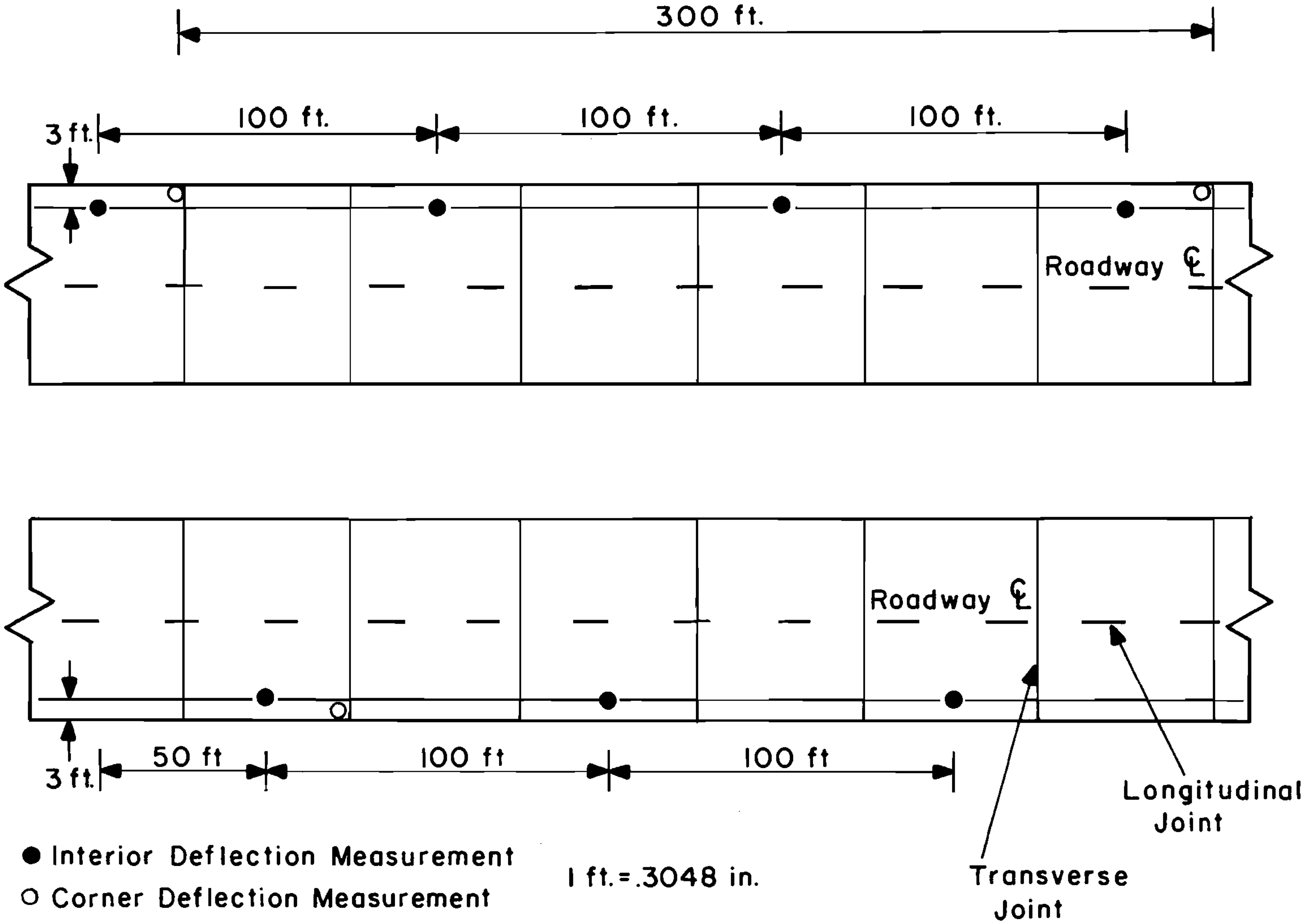


Fig III-2.1. Sampling plan for deflection survey.

plotted on consecutive lines down the page regardless of interval between two specific measurements. The coordinates of each point are printed on either side of the plot. The number of X-Y value cards submitted is also counted by this program and is useful information necessary for use with the TVAL2 program.

CONDITION SURVEY

As part of the pavement evaluation, the condition of the existing pavement should be carefully documented. It is recommended that this information be obtained simultaneously with the deflection survey. The inventory should include such things as the types and amount of cracking, spalling, joint condition, faulting, pumping, blowups, presence of voids, roughness measurements, and drainage. This information should be related to the positions of deflection measurements for future reference. Figure III-2.2 is a suggested condition survey data form. The station limits are selected as base elements, normally 100 foot (30.5-m) long. If the pavement condition is uniform, the length of the base elements may be increased.

Specifically, the cracking classification and whether or not voids are present under the existing pavement are used directly in the overlay design procedure. The rest of the condition survey information will enable the designer, among other things, to explain variations on deflection profiles and to decide whether improvement in drainage might be a viable alternative to increased overlay thickness. Condition survey information is also used as a guideline for selecting design sections.

Cracking

Definition of cracking in this procedure is according to the AASHO definitions for rigid pavements (Ref 8).

Class 1. Class 1 cracking includes those cracks not visible under dry surface conditions to a man with good vision standing at a distance of 15 feet (4.6-m).

Class 2. Class 2 cracks are visible at a distance of 15 feet (4.6-m) but exhibit only minor spalling at the surface. The opening at the surface is less than 1/4 inch (6-mm).

County ____________________

Highway ____________________

Control and Section No. ____________________

Roadway Direction ____________________

Measurement Lane ____________________

Date ____________________

Stations		Cracking					Faulting	Spalling	Pumping	Drainage		Gradeline			Other Comments
From	To	Uncr.	Cl. 1	Cl. 2	Cl. 3	Cl. 4				Good	Poor	Cut	Fill	Natural	

The rater should place a check mark for each observed distress in the appropriate box.

Fig III-2.2. Sample condition survey data form.

Class 3. Class 3 cracks are cracks that opened or spalled at the surface to a width of 1/4 inch (6-mm) or more over a distance equal to at least one-half the crack length. Any portion of the crack opened less than 1/4 inch (6-mm) at the surface for a distance of 3 feet (914-mm) or more is classified separately.

Class 4. A Class 4 crack is defined as any crack which has been sealed.

Recording of Data

Data are to be recorded on the form on Fig III-2.2 as follows:

Stations. The same identification system for stations should be used as for the deflection survey in order to relate information on this sheet to deflection profiles.

Cracking. Each section with base length of 100 foot (30.5-m) should be classified according to the cracking definitions given above. The general type of cracking present should be recorded by a check mark in the appropriate box. If more than 5 percent of the next (more severe) class of cracking occurs in a certain section, that particular class should be checked.

Faulting, Spalling, Pumping. It is recommended that these factors be checked on the form only for presence.

Drainage. Check whether drainage at that particular section is good or poor.

Grade. Check whether cut, fill or natural.

Other Comments. This might include such things as a change in type of construction, a change in pavement width, type of shoulders or an obvious change in soil type.

Reduction of Data

Pavement condition should be classified according to its general type of cracking and should fit into one of the following three classes:

(1) uncracked, class 1 and class 2 cracking,

(2) class 3 and 4 cracking, and

(3) pavements so severly cracked that it should be mechanically broken up into small pieces.

It is suggested that design sections be classified according to the most severe class of cracking that occurs and is checked for more than 25 percent of the deflection survey sections, of 100 foot (30.5-m) length, as indicated on Fig III-2.2. If three or more adjacent deflection survey sections are classified differently from the rest in a particular design section, consideration may be given to treating it as a separate design section.

It is also necessary to record whether voids exist under the existing pavement.

SELECTION OF DESIGN SECTIONS

Projects are to be divided into design sections, using deflection information as well as condition survey information. Sections with different cross sections or that exhibit clearly different cracking patterns should be treated as different sections.

Profiles plotted as discussed in the previous section are to be studied for dividing the project into design sections. A design section would be an area, or section of roadway, that exhibit similar deflection over its length. This first division of the project into design sections is done by the designer through visual examination of the deflection profiles (see Fig I-3.3).

Statistical Hypothesis Testing

The design sections should be checked by statistical methods to see if they are significantly different. This may be accomplished using the computer program TVAL2. Appendix 8 contains an input guide for this program. The student's t-test is used in the TVAL2 program.

The following are the steps and formulas used to model this test (Refs 6 and 22):

a, b = individual measurements of variates in sections designated 1 or 2 respectively,

$\bar{a}$, $\bar{b}$ = mean value of measurements of variates in sections 1 and 2,

n_a, n_b = number of variates in sections 1 and 2, and

d_f = number of degrees of freedom.

Step 1: Calculate the mean ($\bar{a}$) from the section 1 data:

$$\bar{a} = \frac{\Sigma a}{n_a} \tag{III-2.1}$$

Step 2: Calculate the mean ($\bar{b}$) from the section 2:

$$\bar{b} = \frac{\Sigma b}{n_b} \tag{III-2.2}$$

Step 3: Calculate the "pooled estimate of the standard deviation" (S) for the two sections:

$$S = \sqrt{\frac{\Sigma(a-\bar{a})^2 + \Sigma(b-\bar{b})^2}{n_a + n_b - 2}} \tag{III-2.3}$$

This insures that the standard deviation determined is not affected by any difference which may exist between the means of the two sections.

Step 4: Determine the best estimate of the standard deviation of the mean of samples of n_a variates for section 1 ($S_{\bar{a}}$):

$$S_{\bar{a}} = \frac{S}{\sqrt{n_a - 1}} \tag{III-2.4}$$

Step 5: Determine the best estimate of the standard deviation of the mean of samples of n_b variates for section 2 ($S_{\bar{b}}$):

$$S_{\bar{b}} = \sqrt{\frac{S}{n_b - 1}} \qquad \text{(III-2.5)}$$

Step 6: From steps 4 and 5 calculate

$$S_{(\bar{a}-\bar{b})} = \sqrt{S_{\bar{a}}^2 + S_{\bar{b}}^2} \qquad \text{(III-2.6)}$$

Step 7: Hypothesize $M_1 - M_2 = 0$, where M_1 and M_2 are means of two normally and independently distributed sections. Calculate t-value for student t-distribution:

$$t = \frac{(\bar{a}-\bar{b}) - M_{\bar{a}-\bar{b}}}{S_{(\bar{a}-\bar{b})}} \qquad \text{(III-2.7)}$$

Since the hypothesis was made that the means of the two sections were equal, Eq III-2.7 reduces to

$$t = \frac{\bar{a} - \bar{b}}{S_{(\bar{a}-\bar{b})}} \qquad \text{(III-2.8)}$$

Step 8: Obtain t-value from students' t-distribution in Alder and Roessler (Ref 22) or other statistics tables to check hypothesis.

Step 9: Compare computed t-value with table t-value. If computed value is larger than table value, the two sections are significantly different.

Adjacent sections that are not significantly different are to be combined into one section and then that section is to be checked against the next adjacent sections. Each significantly different design section becomes a separate design problem in this procedure.

The designer has to select a level of significance for this test and a five percent level is recommended (Ref 6). If the designer so desires, he may select some other value (see card 3.1 in the TVAL2 input guide - Appendix 8).

The computer program TVAL2 performs the above statistical test and also determines the design deflection for each design section.

Determination of Design Deflection

The TVAL2 program computes for each design section the mean and standard deviations of the deflections and also prints out the design deflection for each section computed as follows (Ref 6):

$$W\alpha = \bar{w} + z\, S_{dw} \tag{III-2.9}$$

where

$W\alpha$ = design deflection, inches,

$\bar{w}$ = mean deflection, inches,

S_{dw} = standard deviation of deflection, inches, and

z = distance from mean to selected significance level on a normal distribution curve.

Table III-2.1 is a list of z values corresponding to design confidence or reliability levels (Ref 6).

TABLE III-2.1. Z VALUES FOR VARIOUS CONFIDENCE LEVELS

Design Confidence Level	Reliability (R)	Z Value
50	50	0
75	25	0.674
90	10	1.282
95	5	1.645
97.5	2.5	1.960
99	1	2.330

CHAPTER III-3. FATIGUE CRACKING ANALYSIS

To perform the overlay thickness design, taking fatigue criteria into account, the RPOD2 computer program is to be used. For pavements where reflection cracking is not considered to be a problem, RPOD2 will predict the required overlay thickness. For portland cement concrete overlays on cracked or jointed pavements, RPOD2 will also include a strain relieving course in the design. In addition, the designer can specify a strain relieving course between a PCC overlay on an uncracked PCC existing pavement if he so desires.

Generally, for asphaltic concrete overlays on cracked or jointed pavements, the reflection cracking analysis, also discussed in Chapter II-4, should be performed in addition to the fatigue analysis.

The RPOD2 computer program, which is a modified version of RPOD1, which is used in the Federal Highway Administration overlay design procedure (Refs 1 and 6), is discussed in Part I. Part II indicates the modifications made in the RPOD2 version.

INPUT VARIABLES FOR RPOD2

Appendix 9 consists of an input guide for the RPOD2 program and indicates all the required input variables. They will be discussed here in detail. The studies discussed in Chapter II-2 indicate that under certain conditions certain variables might either be more important or not required. Table II-3.1 summarizes the input variables for different existing pavement conditions.

As pointed out before, certain variables have been "fixed" in the general fixed-order input guide (Appendix 9), but may be altered using the supplement to the input guide. Table III-3.1 is intended as a guide to the designer in planning his investigation.

Input variables will be discussed in the same way they appear in the fixed-order input guide. Testing procedures are specified in this guide

TABLE III-3.1. INPUT VARIABLES FOR RPOD2 FOR DIFFERENT EXISTING PAVEMENT CONDITIONS

RPOD2 Variables		Pavement Condition			
		With R.L.	Uncracked, Cl. 1 & 2 Cracks, No R.L.	Class 3 & 4 Cracks	Mechanically Broken Up
Traffic prior to overlay		R	R	-	-
Existing pavement:	concrete flexural strength	R	R	-	-
	condition	R	R	R	R
	modulus	R	R	F	F
	Poisson's ratio	F	F	F	F
	thickness	R	R	R	R
Subbase:	modulus	R	R	R	R
	Poisson's ratio	F	F	F	F
	thickness	R	R	R	R
Subgrade:	modulus	-	-	-	-
	Poisson's ratio	F	F	F	F
	thickness	R*	R*	R*	R*
	lab data (M_R vs. σdev)	R	R	E	E
Design deflection		R	R	R	R
Deflection load magnitude		F	F	F	F
Deflection load positions		F	F	F	F
Corner to interior stress ratio		R**	R**	R**	R**
Overlay:	modulus	R	R	R	R
	Poisson's ratio	F	F	F	F
	concrete flexural strength	-	R	R	R
	bonding condition	R	R	-	-
Bondbreaker:	modulus	R+	R	R	R
	Poisson's ratio	F	F	F	F
	thickness	R	R	R	R
Design traffic		R	R	R	R

R = required
F = fixed (can be changed using the supplement to the input guide)
* = subgrade thickness required if bedrock is specified
E = estimate - a good estimate of this value would be sufficient
** = required if existing pavement is JCP
+ = required if bondbreaker is specified

for each material property to be tested. Since the design sections are selected to represent pavements with homogeneous behavior under load and changes in soil conditions and pavement cross sections are taken into account, it is recommended that at least one boring be made in each design section. For extremely long sections, more than one boring may be desirable. If it is impossible to obtain this many borings, then, as an absolute minimum, materials sampling should include borings in the design sections with the highest and lowest deflection.

As pointed out in the random order input guide (Appendix 10), instructions are supplied to the program in the form of directives. The first eight characters of each directive contain a keyword identifying the type of information being entered. All keywords may be abbreviated to their first four characters. Either one of the two input guides may be used, but the general fixed-order input guide is intended to be a more convenient form for general use.

It should be noted that this program has not been metricated; therefore, inputs must be in the British System.

Traffic Prior to Overlay

This input is required for pavements which may have remaining life (uncracked pavements or pavements with class 1 and 2 cracking). It is used to calculate the percentage of remaining life of the existing pavement and should be estimated as accurately as possible, since the percentage of remaining life has a direct influence on overlay thickness (See Chapter II-2).

This traffic information is input into the program as the number of 18-kip (80-kN) equivalent single axle load applications since construction of the facility until the time of overlay. It is recommended that AASHTO equivalency factors be used to convert mixed traffic to equivalent 18-kip (80-kN) single axle loads. These equivalency factors can be found in the AASHTO Interim Guide (Ref 9). Information for this input may be obtained from the Planning Survey Division, File D-10, at the Texas SDHPT.

Should any doubt exist as to the accuracy of this estimate, it is recommended that the designer use a conservative estimate.

Existing Pavement Concrete Flexural Strength

The flexural strength of the existing pavement concrete is required for uncracked existing pavements or pavements that exhibit class 1 and 2 cracking. These pavements have the potential to have remaining life, in which case the governing stress is considered to be at the bottom of the existing pavement and the flexural strength of the existing pavement concrete is to be used in the fatigue equation (Eq I-3.3).

Since it is not practical to cut beams from the existing pavement for standard flexural tests, it is recommended that the flexural strength of the concrete in the existing pavement be determined by the Indirect Tensile Test method as outlined by Anagnos and Kennedy (Ref 23). An approximate correlation between indirect tensile values and flexural strength, as required by this procedure, is given in Fig III-3.1 (Ref 24).

Existing Pavement Condition

In order for the program to determine whether a void factor should be applied, and in what location to consider the governing stress (or strain) (see Part I), it is necessary to specify the condition of the existing pavement. This rating of the existing pavement is obtained from the section on condition survey. Table I-4.8 is an example coding form indicating how a CRC existing pavement with class 1 and 2 cracking and no voids is to be coded (cards 3 and 4).

Elastic Modulus of Pavement Layers

Since ELSYM 5, a linear elastic layer program, is used to determine stresses and strains in RPOD2, material properties such as elastic modulus and Poisson's ratio are required.

Elastic Modulus of Existing Pavement. In this method the existing pavement is portland cement concrete. The modulus of elasticity of portland cement concrete may be determined according to ASTM C469. As an alternative method of determining elastic modulus, the indirect tensile test outlined by Anagnos and Kennedy (Ref 23) is recommended.

As indicated in Table III-3.1, it is necessary to determine the modulus of the existing pavement only if the pavement is uncracked or exhibits class 1 and 2 cracking. For pavements with class 3 and 4 cracking, a modulus value of 500,000 psi (3447 MPa) is to be specified. This input value

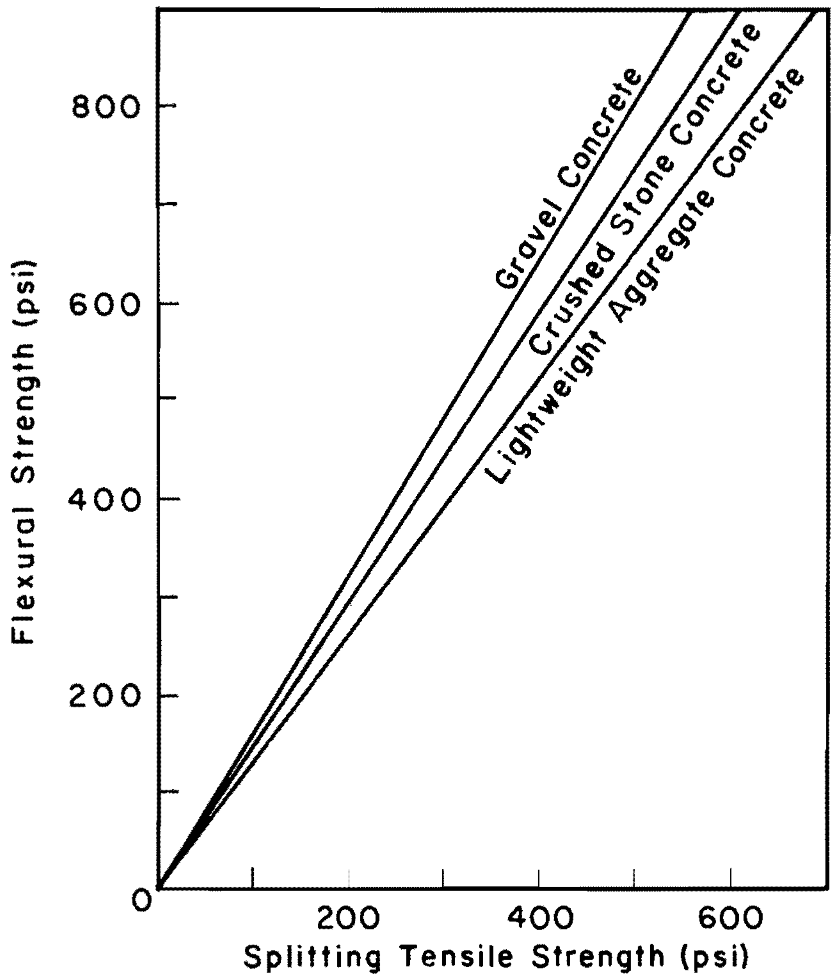


Fig III-3.1. Relationship between flexural strength and splitting tensile strength for concrete made with three different aggregates. (Ref 24) will be used in characterizing the subgrade material. The automatic default value of 500,000 psi (3447 MPa) for this category of pavement applies only to overlay thickness calculations. For mechanically broken up pavements, the automatic default is 70,000 psi (483 MPa). It is suggested that, if the deflection measurements are taken prior to breaking up the pavement, a modulus value of 500,000 psi (3447 MPa) be specified for use in characterization of the subgrade material. If the pavement is broken up first the modulus value to be specified is 70,000 psi (483 MPa).

Elastic Modulus of Bound Subbase Materials. Bound subbase materials will generally be either asphalt or cement treated. Cement materials must be characterized for a modulus of elasticity using ASTM 469 or the indirect tensile test method mentioned in the previous section. Tests should be conducted on undisturbed samples.

Asphalt treated subbase materials should be tested by the dynamic modulus test (Ref 6), as described for asphalt concrete in Appendix 11. At this time, there is no ASTM procedure for this test. Appendix 12 outlines a procedure obtaining the dynamic modulus of asphalt structures using the indirect tensile test method. A characterization temperature of 70°F (21°C) is suggested.

Since it is unlikely that a mechanically broken up pavement, considered to have an effective modulus of 70,000 psi (483 MPa), would still have beneath it a subbase with a greater effective modulus, it is suggested that a maximum value of 70,000 psi (483 MPa) be used for the subbase for mechanically broken up pavement.

By the same token, a maximum modulus value of 500,000 psi (3447 MPa) is suggested for subbases of pavements with class 3 and 4 cracking.

Elastic Modulus of Unbound Subbase Materials. The use of linear elastic layer theory for prediction of stresses, strains, and deflections requires an accurate determination of the modulus of elasticity of the subbase and subgrade materials. Since the modulus of most soils is stress sensitive and varies with repeated loading, the resilient modulus test is the most appropriate test to use for determination of this material property. The resilient modulus is the ratio of stress to resilient strain, in a repetitive loading triaxial test, after an appropriate number of cycles of loading at a specific stress level. In general, unbound subbase samples will be disturbed samples. Recompaction in the laboratory should be done to obtain the inplace density and moisture content. Subbase materials should be tested at confining pressure equal to the overburden pressure and if that is less than one psi the test should be unconfined (Ref 6). It is recommended (Ref 6) that deviator stresses of 20 psi (138 kPa) be used if the total concrete thickness is greater than 6 inches (152-mm). Appendix 13 includes recommended procedures for this test.

Elastic Modulus of Subgrade Materials. The Resilient Modulus Test is also to be conducted on subgrade materials. Subgrade samples will usually be undistrubed samples. If this is not the case, they should be treated similarly to subbase samples in which confining pressures are equal to the overburden pressure. The repeated overburden pressures should be over a range of 2 to 12 psi (13.8 to 82.7 kPa). At least four levels of deviator stress should be used; 2, 5, 8 and 12 psi (13.8, 34.5, 55.2 and 82.7 kPa) are recommended values. The resilient modulus values and corresponding deviator stresses are input to the RPOD2 program on the "lab data" card. The elastic modulus value for subgrade to be specified on the "layer" card is only a value to start iteration on and can be only a rough estimate.

Poisson's Ratio Values for Existing Pavement Layers

Values of Poisson's ration for pavement layers have been "fixed" through the general input guide as follows:

(1) portland cement concrete - 0.15,
(2) asphaltic concrete - 0.30,
(3) stabilized subbases - 0.20,
(4) granular subbases - 0.40, and
(5) subgrade - 0.45.

Should a designer, however, desire to specify Poisson's ratio values for a specific project, it is possible through use of the supplement to the fixed-order input guide (Appendix 9) or the random order input guide (Appendix 10).

Thickness of Existing Pavement Layers

Thicknesses of the different layers in the existing pavement are inputs to the program. These thicknesses are to be determined at the time sampling is being performed for elastic modulus testing.

If a uniform thickness is indicated on the construction plans for a specific section of roadway, it is suggested that all thickness determinations for that section be lumped together and a thickness value be selected in such a way that there is a 90 percent probability of not having a thinner thickness than the selected thickness, as follows:

$$D_{des} = \bar{D} - z\, S_{dD}$$

where

D_{des} = thickness to be used in design, inches;

$\bar{D}$ = mean thickness, inches;

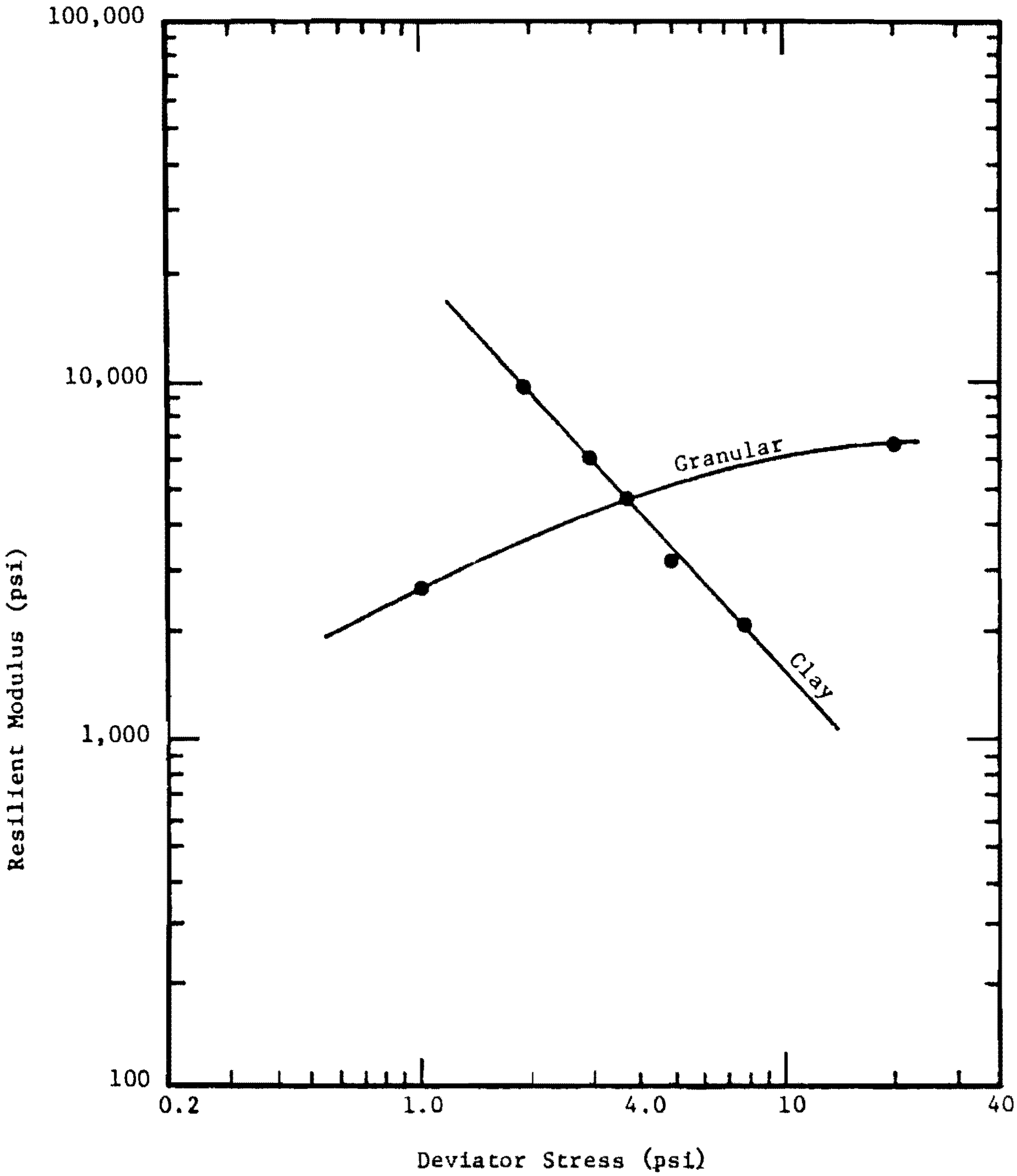
S_{dD} = standard deviation of thickness, inches;

z = distance from mean to selected significance level on a normal distribution curve.

For a 90 percent confidence level $z = 1.28$.

Subgrade "Lab Data"

Subgrade "lab data" is a necessary input if the deflection load differs significantly from the design load (as will be the case when Dynaflect is used). Data required here are resilient modulus and corresponding deviator stress values, as described under "Elastic Modulus of Subgrade Materials." Figure III-3.2 is an example plot showing the relationship between resilient modulus and deviator stress for different types of soils. As an alternative to this, the slope of the log resilient modulus versus the log deviator stress line, S_{SG}, can be used. A method to obtain a value



1 psi = 6.894 KPa

Fig III-3.2. Relationship between resilient modulus and stress for typical clay and granular soils (Ref 6).

for S_{SG} is suggested in Appendix 6 and ranges of values of S_{SG} observed for different soil types are included in Table III-3.2.

Design Deflection

The design deflection is used to characterize the subgrade material, as discussed in Part I. Determination of the value of design deflection has been discussed under the section dealing with selection of design sections. Dynaflect deflection loads are "fixed" when the general fixed-order input guide is used. To input deflections other than Dynaflect deflections, the supplement to the fixed-order input guide or the random order input guide should be used.

Corner to Interior Stress Ratio

In order to obtain the stress adjustment factor, if the existing pavement is JCP, the measured ratio of corner deflection to interior deflection is required. This information is obtained through the deflection survey.

Overlay Elastic Modulus

Overlays considered in this design procedure are either portland cement concrete or asphaltic concrete. The elastic modulus of portland cement concrete can be determined according to Texas SDHPT procedure Tex-421-A, or by using the indirect tensile test (Ref 23). For asphaltic concrete it is necessary to determine the dynamic modulus, as described in Appendix 11 or 12.

Overlay Concrete Flexural Strength

In the case of pavements with no remaining life, the governing stress is considered to be at the bottom of the overlay. For portland cement concrete overlays, it is necessary to input the flexural strength of the overlay concrete. This value is used in the fatigue equation (Eq I-3.3) in predicting fatigue life. Since it is possible to obtain flexural test specimens of the concrete to be used for overlay construction, this flexural strength value should be determined using the center point loading method according to Texas SDHPT method Tex-420-A. Since the flexural strength determined by the third point loading method is required in this method,

TABLE III-3.2. PRACTICAL RANGES OF S_{SG} OBSERVED FOR DIFFERENT TYPES OF SOILS

Soil Type	Range of S_{SG}		
Clay	-.25	to	-1.30
Silty clay	-.08	to	- .66
Clayey silt	-.32	to	-1.00
Sandy silt	-.02	to	- .81
Sand (fine grained)	-.30	to	- .38
Non-plastic, gravelley sand	+.18	to	+ .51

the flexural strength values obtained from the center point loading method should be multiplied by 0.9.

Poisson's Ratio of Overlay

Poisson's ratio for the overlay material has been dealt with in the same way as discussed under "Poisson's Ratio Values for Existing Pavement Layers."

Modulus of Bond Breaker

Generally, if a bondbreaker is used, it will be an asphaltic concrete layer, and so the dynamic modulus as described in Appendix 11 or 12, is to be determined. The program provides a default value of 100,000 psi (689 MPa) for this variable.

Poisson's Ratio of Bond Breaker

Poisson's ratio of bondbreaker has been dealt with as discussed under "Poisson's Ratio Values for Existing Pavement Layers."

Thickness of Bond Breaker

On distressed pavements, where bond breakers will often be used, the bondbreaker layer will also serve as some form of a levelling course, with varying thickness. It is suggested that this course be applied as thin as possible. An average thickness for this layer has to be estimated. The program provides a default value of one inch.

Overlay Design Traffic

The design traffic should be specified in terms of the total number of equivalent 18-kip (80-kN) single axle loads expected in the design lane during the design period for the overlay. It is recommended that AASHTO equivalency factors be used to convert mixed traffic to 18-kip (80-kN) equivalent single axle loads. For information on these equivalency factors, see the AASHTO Interim Guide (Ref 9).

The program calculates fatigue lives for 3, 6, 9 and 12-in (76, 152, 229 and 305-mm) overlays. If a design traffic is specified, it will interpolate from the overlay thickness versus fatigue life relationship a design thickness for the specified design traffic. A maximum of 5 values may be specified, which makes it easy to study different alternatives.

Information on this input can be obtained from the Planning Survey Division, File D-10, at the Texas SDHPT.

RPOD2 OUTPUT

The RPOD2 program claculates fatigue lives for 3, 6, 9 and 12-inch (76, 152, 229 and 305-mm) overlays and prints them out in a table. It also makes a plot of overlay thickness versus fatigue life. In addition, if a design life is specified, the program will also interpolate, from the above mentioned relationship, a thickness of overlay.

In the case of existing pavements of between 1 and 25 percent remaining life, the program will consider the existing pavement both to have remaining life and not to have remaining life. Both thicknesses are then printed out and the designer can select the more economical thickness.

In cases of portland cement concrete pavements with no remaining life, the program would automatically call for the use of a bondbreaker.

In the case of asphaltic concrete overlays on jointed pavements, or cracked pavements, the reflection cracking analysis must be conducted in addition to the fatigue analysis.

CHAPTER III-4. REFLECTION CRACKING ANALYSIS

The RFLCR1 computer program is used to do the reflection cracking analysis. It is intended for asphaltic concrete overlays, but other materials can be analyzed by reviewing the procedure and recognizing the assumptions made in developing the program (Ref 6).

In general, it is suggested that this model be used for asphaltic concrete overlays when reflection cracking is anticipated to be a problem. This analysis, however, is not applicable to mechanically broken up pavements (Ref 6).

This program characterizes the existing pavement through in-field measurements and calculates horizontal tensile strains and vertical shear strains in the overlay. These strains can then be used to design against reflection cracking.

ADDITIONAL CONDITION SURVEY INFORMATION REQUIRED FOR PAVEMENTS WITH A REFLECTION CRACKING PROBLEM

In order to use the RFLCR1 program, it is necessary to obtain some information additional to that required for the fatigue analysis, during the condition survey. The following information is required:

(1) crack or joint spacing,

(2) horizontal movement of the slab at different air temperatures, and

(3) differential vertical deflections.

The horizontal movement information is necessary for horizontal characterization of the pavement, and differential vertical deflections to determine the amount of load transfer at joints or cracks. Methods to obtain this information are discussed under the appropriate headings in the next section.

INPUT VARIABLES FOR RFLCR1

Appendix 14 is a fixed order input guide for the RFLCR1 program. All the input variables required for this program are included in this input guide and are discussed here in detail in the order they appear in the input guide. No variables have been fixed in this input guide, since it was felt that the nature of the sensitivity analysis was too limited. Some default values are used.

Existing Pavement Properties

Elastic Modulus. The elastic modulus of the existing pavement concrete is to be determined as required for the RPOD2 program.

Thermal Coefficient. The thermal coefficient is used in characterizing the existing pavement and also affects the horizontal tensile strain in the overlay (Fig II-3.2 and II-3.4). In general, this variable would not be tested for. Table III-4.1 (Ref 6) gives suggested values for thermal coefficient based on coarse aggregate type. These values correlate with thermal coefficients reported by Ma (Ref 25).

Thickness. The existing pavement thickness is to be determined in the same way as required for RPOD2.

Density. Density of the existing pavement concrete, together with the thickness of that layer, is used to determine overburden pressure, which has an influence on the friction curve when non-plastic subbases are used (Ref 1). Density of concrete can be determined from cores during materials sampling.

Joint or Crack Spacing. The joint or crack spacing on the existing pavement has an influence on the horizontal tensile strain in the overlay.

For continuously reinforced concrete existing pavements, the average crack spacing is the input into the program. Crack spacing is to be determined by an accurate inventory of the cracks present. This can be achieved either by using the photographic techniques suggested by Strauss et al. (Ref 26) or by measuring and recording the distances of cracks from a known reference point with a rolatape or measuring wheel.

TABLE III-4.1 THERMAL COEFFICIENT OF THE EXISTING CONCRETE PAVEMENT (Ref 6)

Coarse Aggregate	Thermal Coefficient* ($x10^{-6}$ in./in./^{o}F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

* 1 in./in./^{o}F = 1.8 mm/mm/^{o}C

The joint spacing, in jointed reinforced concrete pavements, will represent a more critical condition than the crack spacing. It is recommended that the joint spacing be used in this case. With cracked JRCP, the program can be used to evaluate crack measurements, but, in this case, joints should not be included as cracks (Ref 6).

In the case of unreinforced jointed concrete pavements, the joint spacing should be used unless thermal cracks extend through the full thickness of the slab.

For this procedure, it is recommended that a 90 percent confidence level be selected for design, as follows:

$$C_\alpha = \bar{C} + z\,S_{dC} \tag{III-4.1}$$

where

C_α = design crack or joint spacing, feet,

$\bar{C}$ = mean crack or joint spacing, feet,

S_{dC} = standard deviation of crack or joint spacing, feet, and

z = distance from mean to selected significance level on normal distribution curve.

For a 90 percent confidence level, $z = 1.282$ should be used.

C_α is the value to be entered in the field "joint or crack spacing" on card 3 in the input guide.

<u>Movement at Sliding</u>. In order to characterize the existing pavement, the friction curve between the concrete and the subbase is calculated (Ref 6). To complete this calculation the slab movement at which sliding occurs must be known. Very little information exists on friction curves and therefore, Treybig et al. (Ref 6) suggest the values in Table III-4.2, which correlate well with information on friction curves reported by Ma (Ref 25).

TABLE III-4.2. MOVEMENT BETWEEN THE CONCRETE SLAB AND UNDERLAYING LAYER AT WHICH SLIDING OR A CONSTANT FRICTION FORCE OCCURS

Material	Movement at Sliding, inches*
Polyethelene sheeting	0.02
Granular subbase	0.25
Sand	0.05
Sand àsphalt	0.02
Plastic soil	0.05

* 1 inch = 25.4 mm

Existing Pavement Reinforcement Properties

When joint spacings are used in this program, reinforcement properties should not be input. Reinforcement properties are only used with crack spacings (Ref 6).

Elastic Modulus of Steel. It is suggested that the default value for elastic modulus of steel be used. This value is 29,000,000 psi (200 GPa).

Thermal Coefficient. Table III-4.3 gives recommended values for the thermal coefficient of steel.

Area and Perimeter of Steel Per Foot Width. The area and perimeter of steel per foot width of pavement may be calculated from construction records. If no information is available, the bar spacing can be obtained by non-destructive methods. The bar size can be obtained by obtaining some cores that include reinforcing bars.

Steel to Concrete Bonding Stress. For pavements that exhibit no fatigue cracking, the steel to concrete bonding stress can be calculated as follow (Ref 6):

$$\mu = \frac{f_t A_c}{\ell \Sigma} \qquad \text{(III-4.2)}$$

where

μ = bonding stress, psi,

f_t = concrete tensile strength, as determined in accordance with ASTM C-496-71, psi,

A_c = cross-sectional area per foot width of pavement, in.^{2}/ft

ℓ = one half of the crack spacing, inches, and

Σ = perimeter of the steel per foot width of pavement, in/ft width.

TABLE III-4.3. THERMAL COEFFICIENTS OF DIFFERENT TYPES OF STEEL (Ref 6)

Type of Steel	Thermal Coefficient* ($\times 10^{-6}$ in./in./^{o}F)
Steel (1020)	6.5
Steel (1040)	6.3
Steel (1080)	6.0
Steel (18Cr - 8Ni stainless)	5.0

* 1 in./in./^{o}F = 1.8 mm/mm/^{o}C

For cases where fatigue cracking is present, the following equation can be used to determine the bonding stress:

$$\mu = \frac{9.5\sqrt{f_c}}{d} \qquad \text{(III-4.3)}$$

where

f_c = compressive strength of concrete determined in accordance with ASTM C 116-60T, psi, and

d = diameter of reinforcing bar, inches.

Equation III-4.3 is to be used only when fatigue cracking is very prevalent (Ref 6).

Horizontal Characterization of Pavement. As mentioned earlier, characterization measurements are to be taken during the condition survey. Figure III-4.1 is an example of a form which may be used for horizontal characterization of the pavement (Ref 6).

Treybig et al. (Ref 6) suggest the following method to determine horizonatal movements in the pavement. Horizontal movement of the slab is determined by measuring the crack width at different air temperatures. It is necessary to obtain these widths over as nearly the same temperature differentials as possible. For measurement of crack (or joint) widths, a microscope with a graduated eyepiece, capable of measuring to .001 inch (.025-mm) is recommended. Joints or cracks to be measured should be properly marked so that measurements can be taken at the same location at other temperatures. At least three readings should be taken at each location for each specific temperature and the average value should be recorded.

For pavements with variable crack spacing, the crack movements to be used with the design crack spacing must be determined as indicated on Fig III-4.2 (Ref 6). The crack width at the lower temperature, $Y(T_L)$, the crack width at higher temperature, $Y(T_H)$, are input values to the program.

When joint spacings are used Fig III-4.3 (from Ref 6) must be developed to determine the horizontal characterization input data required for the program. A 90 percent confidence level is recommended for design.

Horizontal Movements

County ____________________ Highway ______________________

Description _______________ Measuring Device ______________ Date _________

Measurement Number or Location	Joint or Avg. Crack Spacing, feet L	Air Temperature °F. T_L	*Joint or Crack Width, inches $Y(T_L)$	Air Temperature, °F T_H	*Joint or Crack Width, inches $Y(T_H)$	Temperature Change °F. ΔT_c	Joint or Crack Movement, inches $Y(T_L) - Y(T_H)$

Fig III-4.1. Example of form for recording existing pavement characterization data.

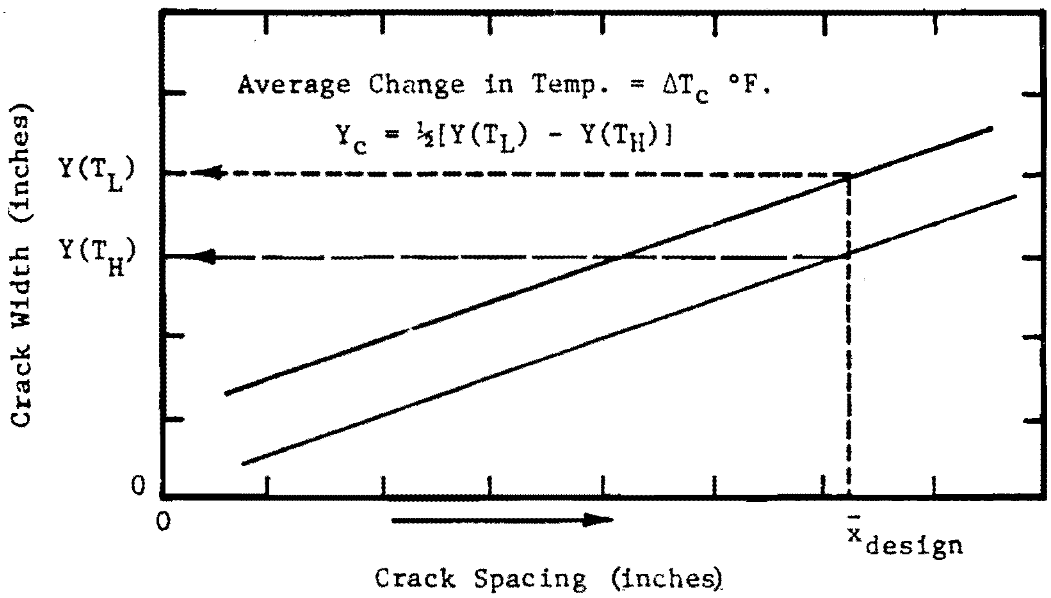
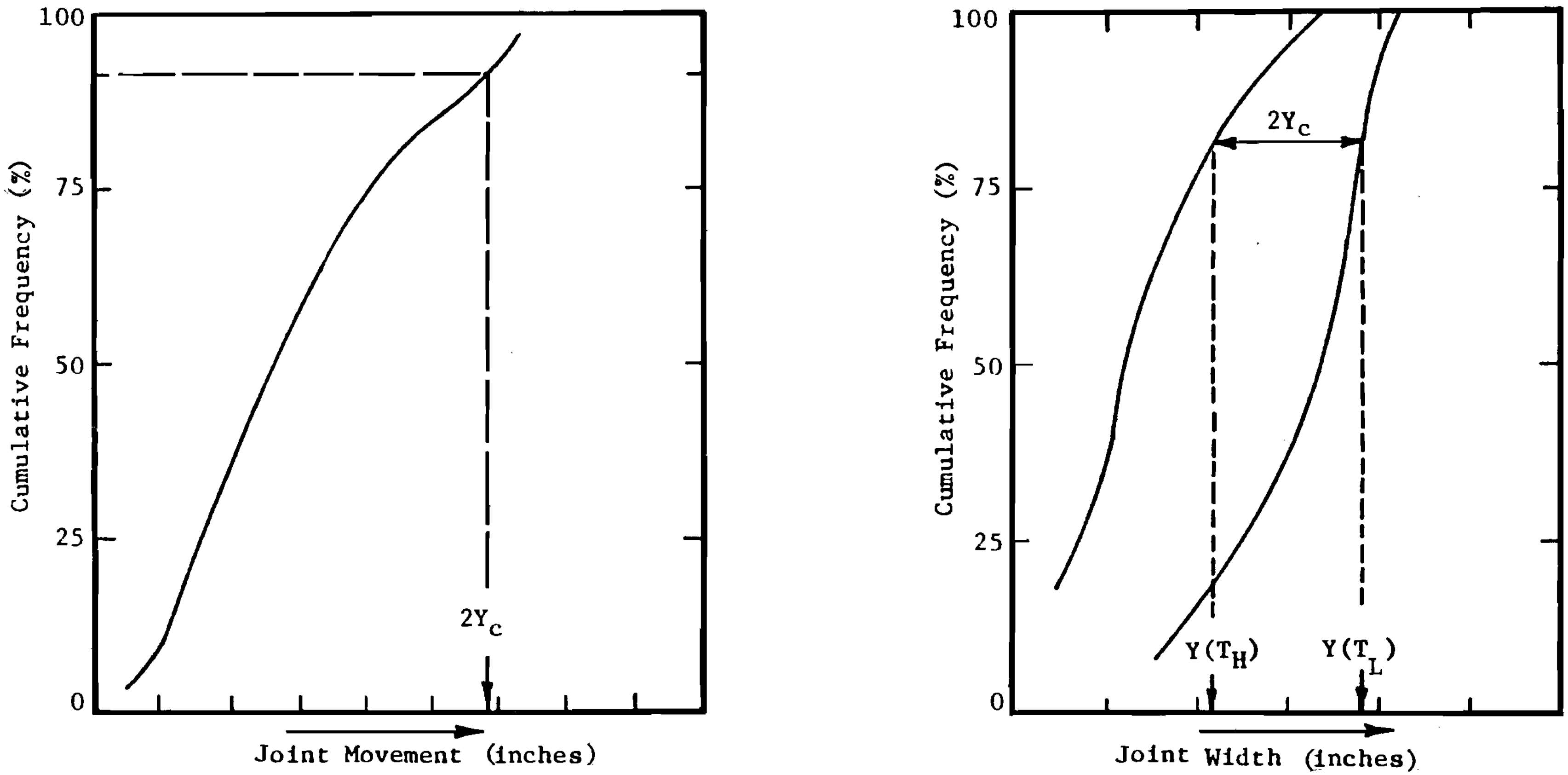


Fig III-4.2. Realtion between crack spacing and concrete movement at a crack for a specific temperature change and location on pavement. (Ref 6)



(a) Determination of the design joint movement.

(b) Estimating maximum joint widths to be input into the program RFLCR1.

Fig III-4.3. Illustration showing how the design joint movement, based on the characterization temperature change, can be obtained for an uncracked pavement. (Ref 6)

Minimum Temperature Observed Since Construction of Pavement. The minimum temperature observed since construction of the existing pavement is used to characterize a reinforced pavement. This variable is not used for unreinforced pavements. This information can be obtained from weather records. Suggested values of minimum temperatures observed in Texas are given in Fig III-4.4.

Vertical Characterization of the Pavement. Vertical characterization of the pavement can be achieved using a regular deflection measuring device, such as a Dynaflect or a Benkelman Beam, or using a differential deflection device that uses a dial gauge to measure the relative movement between two adjacent slabs.

It is necessary to determine the deflections at a joint with the load on one side of it. Deflections are to be taken at the loaded side of the joint, as well as the unloaded side. Alternatively a deflection measurement can be taken at the loaded side of the joint and a differential deflection measurement can be taken at the same position.

The differential deflection can be calculated as follows:

$$w_d = w_L = w_u \qquad \text{(III-4.4)}$$

where

w_d = differential deflection, inches,

w_L = deflection of the loaded joint, inches, and

w_u = deflection of the unloaded joint, inches.

The percent load transfer can be determined as follows:

$$L_t = \left[1 - \frac{w_d}{w_L}\right] x\ 100 \qquad \text{(III-4.5)}$$

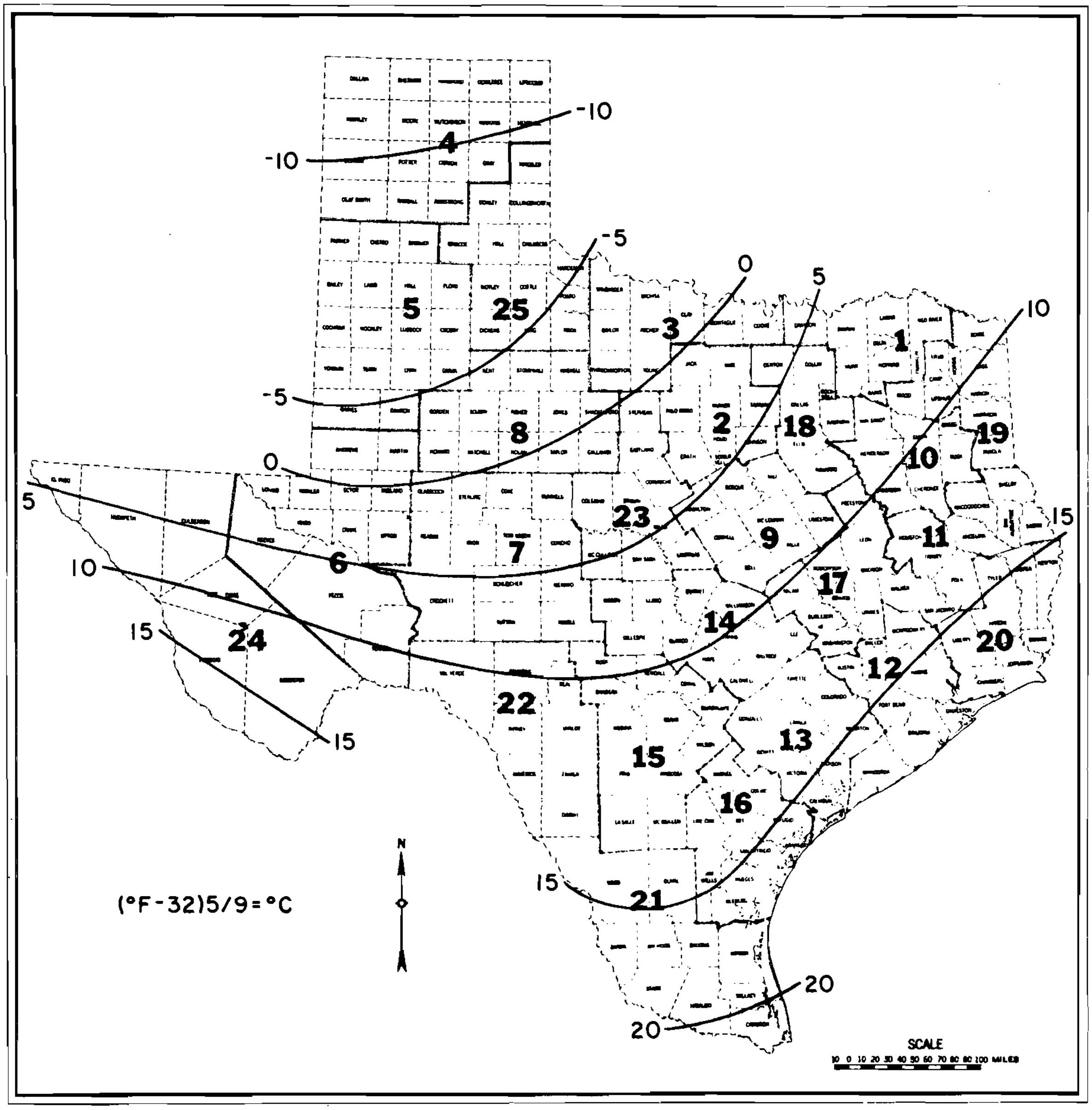


Fig III-4.4. Minimum observed temperatures in Texas.

where

L_t = percent load transfer and

all other variables are as defined above.

Figure III-4.5 is an example of a form that is suggested for use to collect data for the vertical characterization of the pavement (Ref 6). The columns for joint width and temperature are only for physical information. They are used only when it is anticipated that joint width will have a distinct effect on load transfer.

A 90 percent design level is recommended for load transfer. It can be calculated as follows:

$$L_{tdes} = \bar{L}_t - ZS_{dLt} \qquad \text{(III-4.6)}$$

where

L_{tdes} = design load transfer, percent,

$\bar{L}_t$ = mean value of load transfer, percent,

S_{dLt} = standard deviation for load transfer, percent, and

Z = distance from the mean to selected significance level on a normal distribution curve. For a 90 percent design level, a value of 1.28 is to be used.

Should the number of observations be less than 25, it is recommended that the t-statistic rather than the Z-value be used. Figure III-4.6 can be used for determining this value for smaller samples.

Differential Vertical Deflections

County ______________ Highway ______________

Description ______________ Measuring Device ______________ Date ______________

Measurement Number or Location	Joint Width, inches	Temperature, °F	*Deflection, mils		Differential Deflection, inches	Percent Load Transfer %
			Loaded Joint W_L	Unloaded Joint W_U	W_d	L_T

Fig III-4.5. Example of form for determining load transfer at discontinuities in the existing pavement.

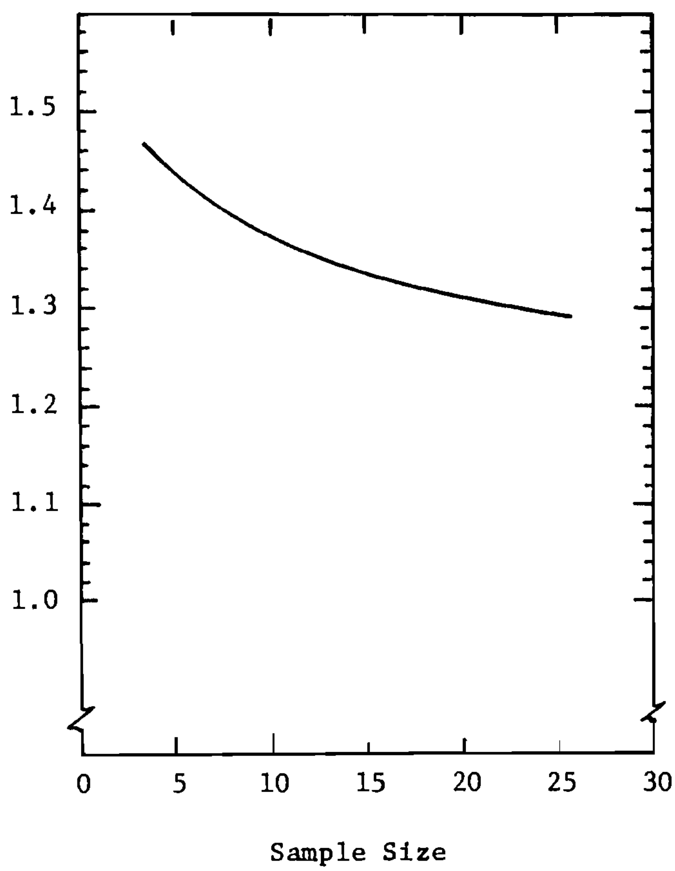


Fig III-4.6. Relationship between t-statistic and sample size for determining L_{tdes} at a 90% design level.

Overlay Properties

Creep Modulus. No standard test exists for determining the creep modulus of the asphaltic concrete. The sensitivity analysis described in Chapter II-3 indicated that this is an important variable, so, even though a standard test is not available, consideration should be given to determining this properly by means of a laboratory experiment rather than by using a nomograph. This should be done at the design temperature for theoverlay and at a loading time of between 6 and 12 hours.

Instead of laboratory testing, the FHWA method (Ref 6) suggests the use of nomographs for determining the creep modulus. Figures III-4.7 to III-4.9 (Ref 6) can be used to estimate this value.

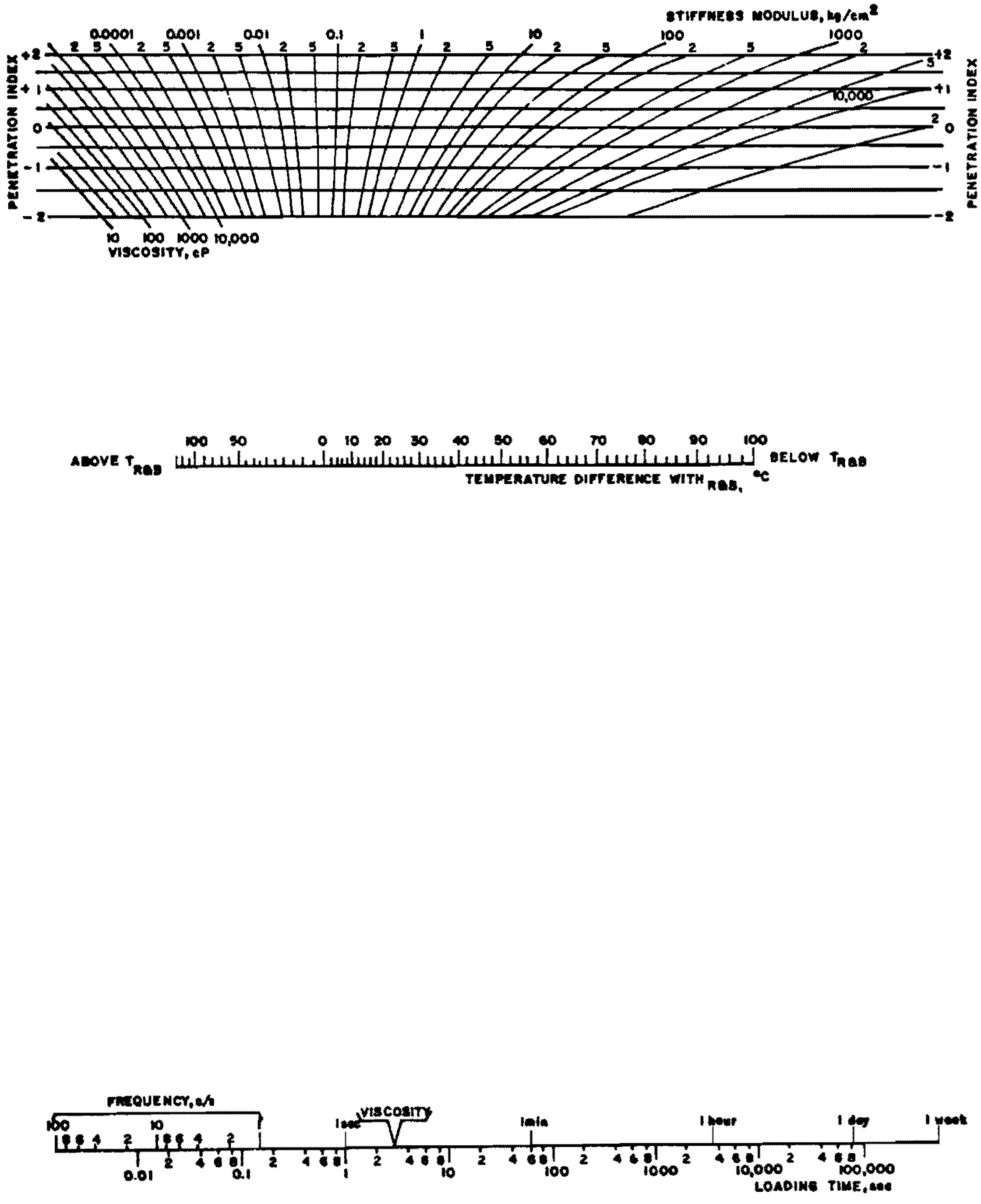
To determine the penetration index from Fig III-4.7, it is necessary to determine the Ring and Ball Temperature (T_o') in accordance with ASTM D 36, and the penetration in accordance with ASTM D5.

Figure III-4.7 is then used to determine the stiffness modulus of the asphalt at a loading times of 6 to 12 hours. The temperature difference represents the difference between the Ring and Ball test temperature (T_o') and the minimum temperature expected to occur for the overlay material, determined as discussed in the section on design temperature changes.

Using Fig III-4.9, the creep modulus of the asphaltic concrete mixture can be determined if the stiffness of the asphalt and the volume concentration of the aggregates (C_v) are known.

Thermal Coefficient. The overlay will generally be of asphaltic concrete and a thermal coefficient for asphaltic concrete of 1.2×10^{-5} in/in/^{o}F (Ref 6) is suggested if this value is not known.

Thickness. The thickness of overlay is required to calculate tensile and shear strains and also to adjust the friction curve if non-plastic subbases are used. In general the thickness predicted by the fatigue analysis RPOD2 will be used as a first trial. Should the strains be larger than the permissible, the thickness has to be increased. As an alternative to increasing the thickness, a bondbreaker, overlay reinforcement or an intermediate layer may be considered for design against reflection cracking.



$$1 \text{ psi} = 14.223 \frac{\text{kg}}{\text{m}^2} = 6.894 \times 10^{-3} \text{ MPa}$$

Fig III-4.7. Nomograph for predicting the stiffness modulus of asphaltic bitumens, after Heukelom and Klomp (Ref 6).

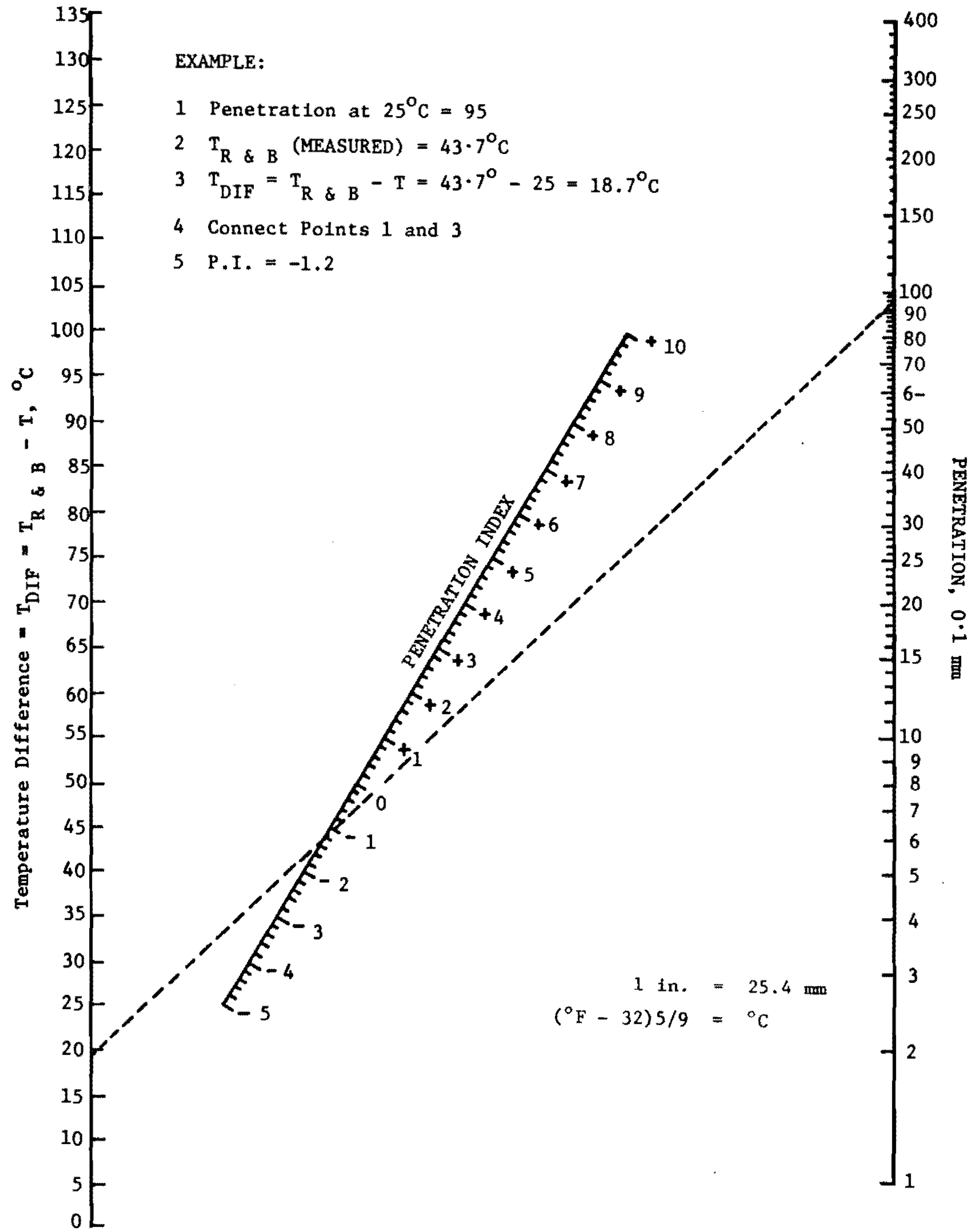


Fig III-4.8. Nomograph for determining Pfeiffer and Van Doormaal's Penetration Index (Ref 6).

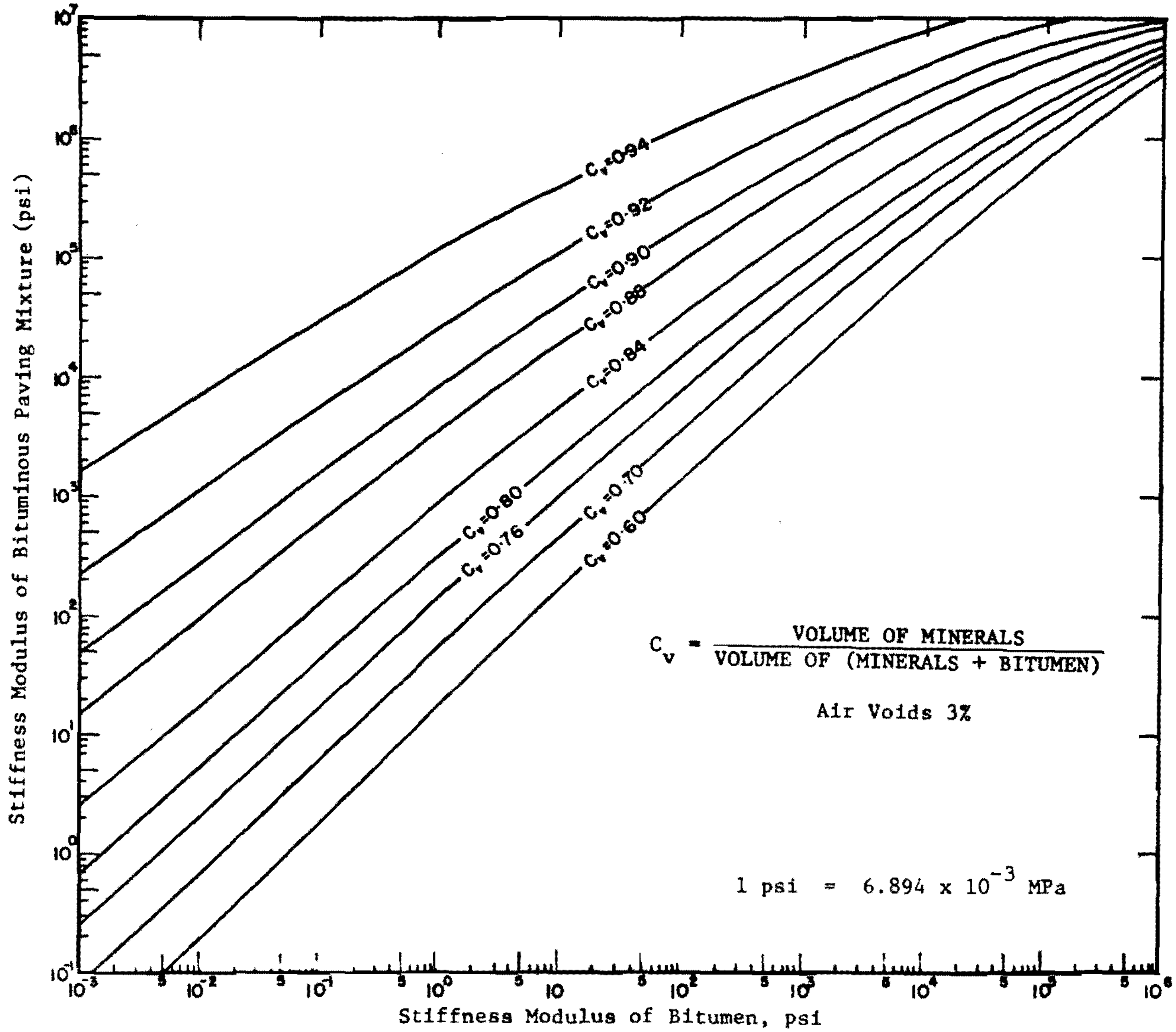


Fig III-4.9. Relationships between moduli of stiffness of asphalt cements and of paving mixtures containing the same asphalt cements (Based on Hukelom and Klomp). (Ref 6)

Density. The overlay density is, together with the thickness, used to adjust the friction curve. It is to be determined from laboratory compacted specimens of the overlay asphaltic concrete. Adjustments should be made to account for compaction anticipated under field conditions.

Poisson's Ratio. It is suggested that a value of 0.3 be used for Poisson's ratio of the asphaltic concrete overlay.

Dynamic Modulus. The dynamic modulus of the overlay is used in calculation of overlay vertical shear strains. Since these shear strains are load associated, it is suggested that the same dynamic modulus used in RPOD2 be used here. A temperature of 70°F (21°C) is suggested for determining dynamic modulus in Texas.

Overlay to Existing Surface Bonding Stress. The overlay to existing surface bonding stress is dependent on the type and condition of the existing surface. Table III-4.4 shows values of bonding stress suggested in the FHWA method (Ref 6).

It is recommended that these values be used until more information becomes available. If no bonding stress is used, it is assumed that the overlay and existing surface are fully bonded.

Overlay Reinforcement Properties

The overlay reinforcement properties are dependent on the type of reinforcement used. They are to be determined as previously mentioned and include

(1) elastic modulus,

(2) thermal coefficient,

(3) area of reinforcement per foot width, and

(4) allowable tensile strain.

TABLE III-4.4. RECOMMENDED VALUES OF ASPHALTIC OVERLAY BONDING STRESS (Ref 6)

Condition of Existing Surface	Average Bonding Stress, psi
Smooth; polished surface; no exposed coarse aggregate	50
Rough; same as for smooth surface but some of the as-constructed texture remains; small amount of coarse aggregate exposed	500
Very rough; worn surface with exposed coarse aggregate; contains aggregate popout; contains surface texture	1200
Jagged; grooves present; numerous aggregate popouts; coarse aggregate highly exposed	semi-infinite

1 psi = 6.89 kPa

Bondbreaker

The program assumes no horizontal force transfer between the overlay and the concrete over an area where a bondbreaker is used. Materials that would be suitable for bondbreakers are sand layers, unbound granular layers, and any smooth frictionless material.

Width or Length in Direction of Traffic. The width or length of the bondbreaker in the direction of traffic is an input to the program.

Intermediate Layer Properties

For a cushion or intermediate layer, the same types of properties are required as for the overlay material, such as:

(1) creep modulus,

(2) thermal coefficient,

(3) thickness,

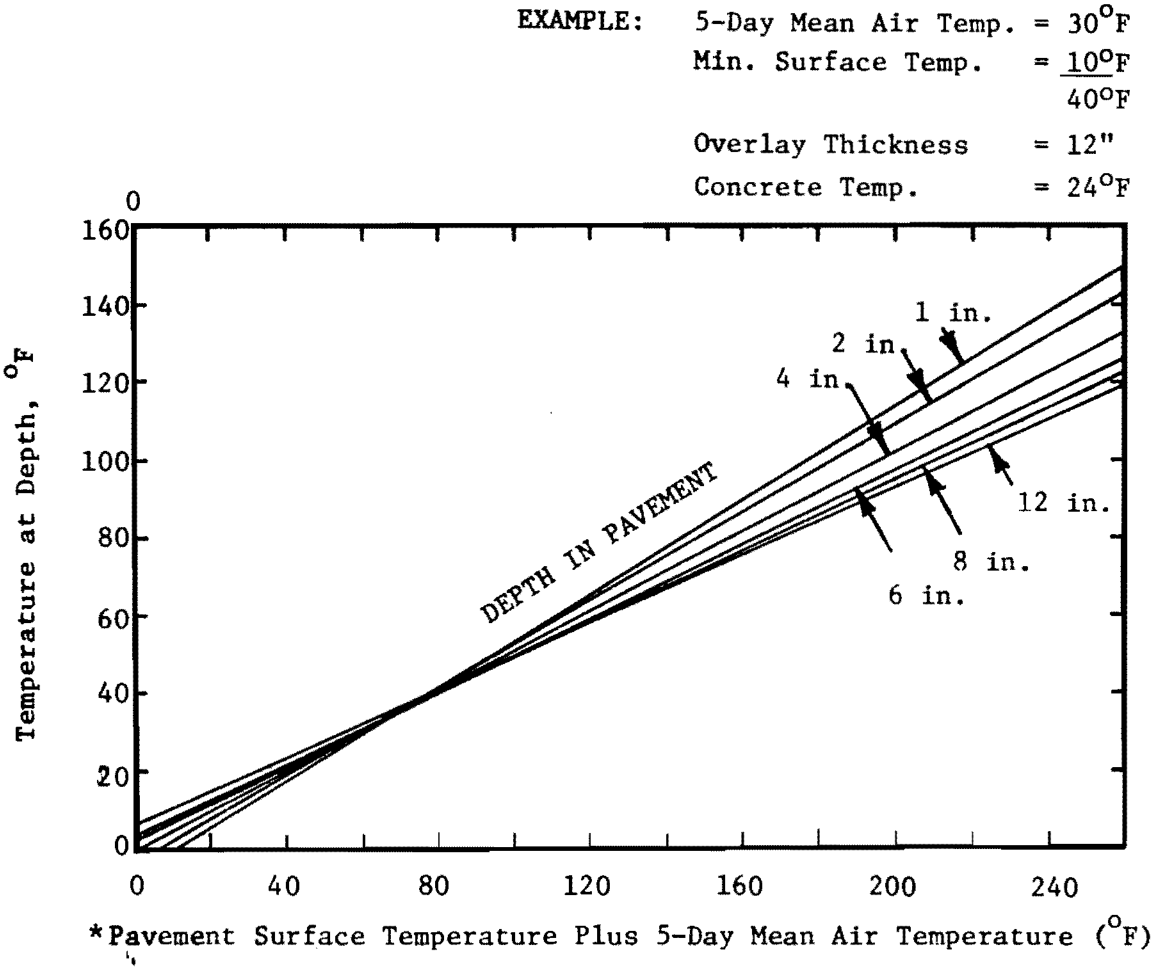
(4) density, and

(5) dynamic modulus.

The allowable strain of the intermediate layer material is also an input to the program.

Design Temperature Changes

Design temperatures for different layers can be determined using Fig III-4.10 (Ref 6). This figure is only applicable to asphalt overlays. For other materials, similar information should be obtained from the producer. To use Fig III-4.10, the minimum five-day average air temperature expected to occur during the design period is added to the minimum surface temperature during those five days. This value is entered on the horizontal axis of Fig III-4.10 and the design temperature at any depth can then be read on the vertical axis. Figure III-4.11 gives suggested values of pavement surface temperature plus five-day mean air temperatures to be used for different districts in Texas.



*See Fig II-4.11.

Fig III-4.10. Predicting pavement temperatures at the bottom of an asphalt overlay. (Ref 6)

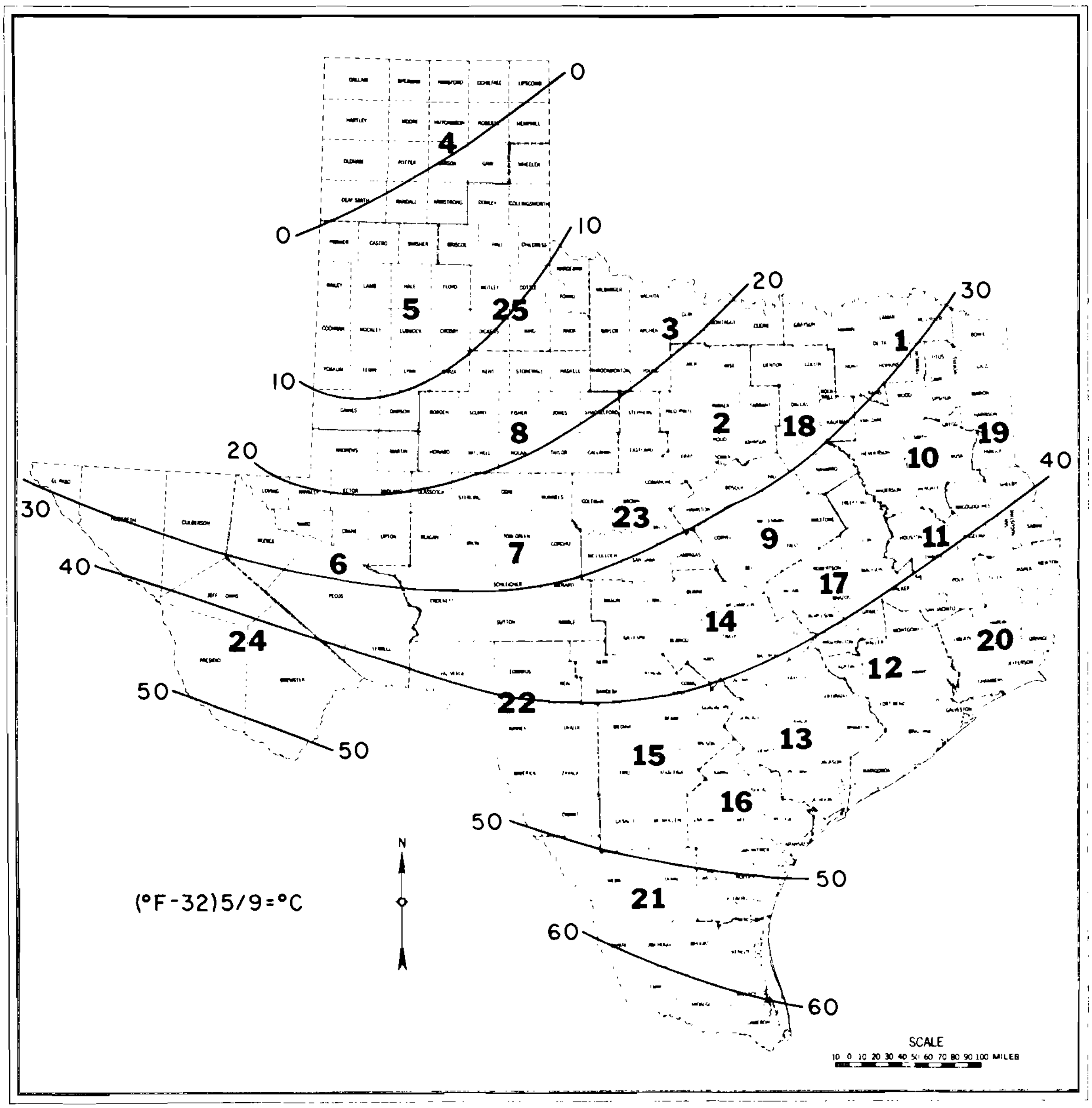


Fig III-4.11. Values for minimum pavement surface temperature plus minimum five-day mean air temperatures, ^{O}F for Texas.

Design Temperature for Existing Pavement. The design temperature for the existing pavement is the temperature at the bottom of the intermediate layer of the overlay if no intermediate layer is used.

Design Temperature for the Intermediate Layer. The design temperature for the intermediate layer is the temperature at the bottom of the overlay.

Design Temperature for the Overlay. The design temperature for the overlay is the temperature at the surface.

Design Load

The design load is to be specified by the designer. The width of the design load is the distance between the outer edges of a set of wheels on the wheel configuration considered for design.

Friction Curve

For some materials, an increase in overburden pressure will increase the slope of the friction curve (Ref 5). These are only non-plastic soils or subbases. It is recommended that the adjusted friction curve not be used for plastic soils. The Friction curve switch (card type 12 in the input guide) designates whether the adjusted friction curve should be used or not.

RFLCR1 OUTPUT

Output of the RFLCR1 program includes:

(1) beta values (restraint coefficient),

(2) slope of the friction curve,

(3) stresses in the existing pavement, and

(4) overlay strains.

The horizontal tensile strain and the vertical shear strain in the overlay are used in designing against reflection cracking.

Limiting Value for Horizontal Tensile Strain

Since the horizontal tensile strain is caused by thermal movement and the more critical conditions prevail at a low temperature, it is suggested that the tensile strain at failure be determined using the indirect tensile test described by Anagnos and Kennedy (Ref 23). A value of 70 percent of the tensile strain for a strength test is suggested for use as a maximum allowable tensile strain.

Limiting Value for Vertical Shear Strain

The vertical shear strain is a repetitive strain which is load associated. For that reason, the dynamic modulus of the overlay is used in determining this strain. Since loading is independent of temperature, a suitable temperature for design against the vertical shear strain is the mean annual air temperature, of $70^{\circ}F$ ($21^{\circ}C$), as used in the fatigue cracking analysis. A relationship between this shear strain and allowable strain repetitions presented in Fig III-4.12 was developed as outlined in Appendix 6.

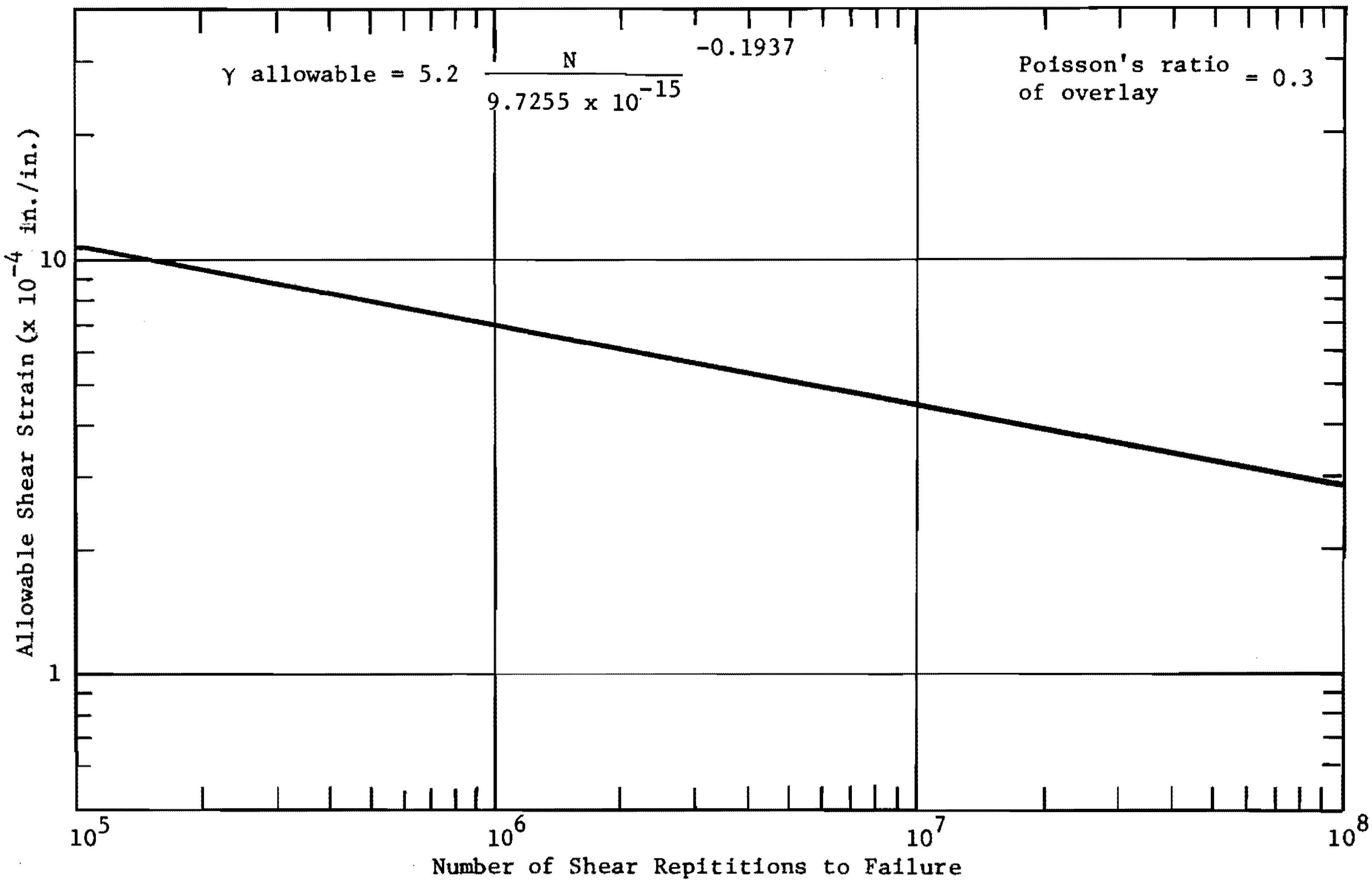


Fig III-4.12. Relation between allowable shear strain and repititions to failure.

It is suggested that a standard 18-kip single axle load be used, as for RPOD2; the maximum allowable shear strain is to be determined from Fig III-4.12 using the number of equivalent 18-kip (80-kN) single axle load applications in the design lane during the design period as the required number of strain repetitions.

CHAPTER III-5. SUMMARY AND IMPLEMENTATION

This part of the report presents a user's manual for the Texas SDHPT rigid pavement overlay design procedure. The selection of design sections, fatigue cracking analysis and reflection cracking analysis is dealt with in detail.

The final selection of the design thickness is made so as to satisfy the fatigue failure criteria as well as the reflection cracking criteria. It should be noted that for economical reasons, it might be decided to maintain reflection cracks rather than to eliminate them. Such a decision is up to the designer.

In general, structural overlays of less than 3 inches (75-mm) would seldom be economical and are generally not recommended.

Finally, this user's manual will be instrumental in the implementation of this procedure in Texas. It will allow the design engineers to become familiar with this rigid pavement overlay design procedure.

REFERENCES

1. Treybig, Harvey J., B. F. McCullough, Phil Smith, and Harold von Quintus, "Overlay Design and Reflection Cracking Analysis for Rigid Pavements, Volume 1 - Development of New Design Criteria," Research Report No. FHWA-RD-77-66, Federal Highway Administration, Washington, D.C., August 1977.

2. McComb, Richard A., and John J. Labra, "A Review of Structural Evaluation and Overlay Design for Highway Pavements," Pavement Rehabilitation Proceedings of a Workshop - Report No. FHWA-RD-74-60, Federal Highway Administration, Washington, D.C., June 1974.

3. Texas State Department of Highways and Public Transportation, Highway Deisgn Division: "Operations and Procedures Manual Part IV and Appendices A-H" Texas State Department of Highways and Public Transportation, Texas 1976.

4. Kher, Ramesh K., W. Ronald Hudson, and B. Frank McCullough, "A Systems Analysis of Rigid Pavement Design," Research Report Number 123-5, Center for Highway Research, The University of Texas at Austin, January 1971.

5. McCullough, B. F., "A Pavement Overlay Design System Considering Wheelloads, Temperature Changes, and Performance," Institute of Transportation and Traffic Engineering Graduate Report, University of California, Berkeley, July 1969.

6. Treybig, Harvey J., B. F. McCullough, Phil Smith, and Harold von Quintus, "Overlay Design and Reflection Cracking Analysis for Rigid Pavements, Volume 2 - Design Procedures," Research Report No. FHWA-RD-77-67, Federal Highway Administration, Washington, D.C., August 1977.

7. Nayak, B. C., W. R. Hudson, and B. Frank McCullough, "A Sensitivity Analysis of Rigid Pavement Overlay Design Procedure," Research Report Number 177-11, Center for Highway Research, The University of Texas at Austin, June 1977.

8. "The AASHO Road Test, Report 5, Pavement Research," Special Report 61E, Highway Research Board, 1962.

9. "AASHTO Interim Guide for Design of Pavement Structures 1972," Association of State Highway and Transportation Officials, 1974.

10. Smith, Phil, Harvey J. Treybig, and B. F. McCullough, "Concepts for Rigid Pavement Overlay Design," Proceedings, Internation Conference on Concrete Pavement Design, Purdue University, Indiana, 1977.

11. Hugo, F., and P. Raath, "The Prevention of Reflective Cracking in Asphalt Overlays," Proceedings, Conference on Asphalt Pavements in Southern Africa, Durban, South Africa, 1974.

12. Kennedy, Thomas W., W. Ronald Hudson, and B. F. McCullough, "Variability of Material Properties for Airport Pavement Systems," Report CE-5 Austin Research Engineers, Inc., Austin, Texas, December 1974.

13. Ferguson, P. M., Reinforced Concrete Fundamentals, 3rd Edition, John Wiley and Sons, New York, 1973.

14. Claessen, A. I. M., and R. Ditmarsch, "Pavement Evaluation and Overlay Design - The Shell Method," Proceedings, Fourth International Conference Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1977.

15. Austin Research Engineers, Inc., "Asphalt Concrete Overlays of Flexible Pavements - Volume 1, Development of New Design Criteria, Research Report No. FHWA-RD-75-75, Federal Highway Administration, Washington, D.C., June 1975.

16. Zaniewski, John P., "Design Procedure for Asphalt Concrete Overlays of Flexible Pavements," Dissertation, The University of Texas at Austin, December 1977.

17. Townsend, Frank C. and Ed. E. Chisolm, "Plastic and Resilient Properties of Heavy Clay under Repetitive Loadings," Technical Report S-76-1, Office Chief of Engineers, U.S. Army, Washington, D.C., November 1976.

18. Chisolm, Ed E., and Frank C. Townsend, "Behavioral Characteristics of Gravelly Sand and Crushed Limestone for Pavement Design," Technical Reports S-76-17, Office Chief of Engineers, U.S. Army, Washington, D.C., September 1976.

19. Westergaard, H. M. "Stresses in Concrete Pavements Computed by Theoretical Analysis," Public Roads, Vol 7, No. 2, April 1926.

20. Yoder, E. J., and M. W. Witczak, Principles of Pavement Design, 2nd edition, John Wiley and Sons, New York, 1975.

21. Sutaria, T. C. and W. Ronald Hudson, "A Sensitivity Analysis of Flexible Pavement System FPS-11," Masters of Science, Thesis, The University of Texas at Austin, February 1973.

22. Alder, Henry L. and Edward B. Roessler, Introduction to Probability and Statistics, Fifth Edition, W. H. Freeman and Company, San Francisco, California, 1972.

23. Anagnos, James N., and Thomas W. Kennedy, "Practical Method of Conducting the Indirect Tensile Test," Research Report 98-10, Center for Highway Research, The University of Texas at Austin, August 1972.

24. Grieb, W. E., and G. Werner, "Comparison of Splitting Tensile Strength of Concrete with Flexural and Compressive Strengths," Proceedings, Vol. 62, American Society for Testing Materials, pp. 972-995, 1962.

25. Ma, James, "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavements," Master of Science Thesis, The University of Texas at Austin, August 1977.

26. Strauss, Pieter, James Long, and B. Frank McCullough, "Development of Photographic Techniques for Performance Condition Surveys," Research Report 177-10, Center for Highway Research, The University of Texas at Austin, May 1977.

27. Packard, Robert G., "Design of Airport Pavements," Portland Cement Association, 1973.

28. Scrivner, F. H., G. Swift, and W. M. Moore, "A New Research Tool for Measuring Pavement Deflection," Highway Research Record No. 129, Highway Research Board, National Academy of Sciences, 1966.

29. Haas, Ralph, and W. Ronald Hudson, Pavement Management Systems, McGraw-Hill Company, New York, 1978.

30. Claessen, A.I.M., C. P. Valkering, and R. Ditmarsch, "Pavement Evaluation with the Falling Weight Deflectometer," Proceedings, Association of Asphalt Paving Technologists Vol 45, February 1976.

31. Kerbs, R. D., and R. D. Walker, Highway Materials, McGraw-Hill Company, New York, 1971.

32. McCullough, B. Frank, Adnan Abou-Ayyash, W. Ronald Hudson, and Jack P. Randall, "Design of Continuously Reinforced Concrete Pavements for Highways," Research Project NCHRP 1-15, Center for Highway Research, The University of Texas at Austin, August 1975.

33. Navarro, Domingo, and Thomas W. Kennedy, "Fatigue and Repeated-Load Elastic Characteristics of Inservice Asphalt-Treated Materials," Research Report 183-2, Center for Highway Research, The University of Texas at Austin, January 1975.

34. Merrit, F. S., Standard Handbook for Civil Engineers, 2nd edition, McGraw-Hill, New York, 1976.

35. Hudson, W. Ronald, and Thomas W. Kennedy, "An Indirect Tensile Test for Stabilized Materials," Research Report 98-1, Center for Highway Research, The University of Texas at Austin, January 1968.

36. Kennedy, Thomas W., "Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test, " Proceedings, Association of Asphalt Paving Technologists, Vol 46, February 1977.

37. Austin Research Engineers, Inc., "Asphalt Concrete Overlays of Flexible Pavements, Volume 2 - Design Procedures," Research Report No. FHWA-RD-75-75, Federal Highway Administration, Washington, D.C., June 1976.

APPENDIX 1

RPOD2 AND RFLCR1 COMPUTER OUTPUT FOR ILLUSTRATIVE OVERLAY DESIGN PROBLEM

```
RRRR    PPPP     OOO    DDDD    2222
R   R   P   P   O   O   D   D       2
RRRR    PPPP    O   O   D   D     222
R  R    P       O   O   D   D   2
R   R   P        OOO    DDDD    22222
```

NOTICE:

THIS COMPUTER PROGRAM IS A MODIFICATION OF THE ORIGINAL RIGID PAVEMENT OVERLAY DESIGN COMPUTER PROGRAM, RPOD1, DEVELOPED BY AUSTIN RESEARCH ENGINEERS INC, AUSTIN, TEXAS AND DOCUMENTED IN FEDERAL HIGHWAY ADMINISTRATION REPORT NOS. FHWA-RD-77-66 AND -67.

THE PROGRAM WAS MODIFIED TO CREATE RPOD2 BY THE CENTER FOR HIGHWAY RESEARCH (AT THE UNIVERSITY OF TEXAS AT AUSTIN) TO ADAPT THE PROCEDURE FOR USE BY THE TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION. DOCUMENTATION FOR THESE MODIFICATIONS AND USE OF THE PROGRAM IS PRESENTED IN CFHR RESEARCH REPORT NO. 177-13 BY OTTO SCHNITTER, W R HUDSON, AND B F MCCULLOUGH.

THIS PROGRAM WAS DEVELOPED FOR POSSIBLE USE BY THE TEXAS SDHPT BUT DOES NOT IMPLY ACCEPTANCE AS POLICY OR STANDARD OF THE DEPARTMENT. ANY USER SHOULD ACCEPT RESPONSIBILITY FOR THE ACCURACY OF THE INPUTS AND THE VALIDITY OF THE RESULTS.

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    1    ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 1.


          I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                    TYPE 1 AND 2 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                            570.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE             4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC      TYPE OF
   NO.     (IN.)      RATIO    MODULUS      MATERIAL
                                (PSI)
   1         8.0       .150   4200000.        CRCP
   2         6.0       .200    500000.   STABILIZED BASE
   3  SEMI-INFINITE    .450      5000.       SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES         .0006230
  LOAD MAGNITUDE, POUNDS                        500.0
  TIRE PRESSURE, PSI                            167.0

                        X,Y COORDINATES, INCHES
  LOAD  1 LOCATION      ( -10.00  ,    0.00  )
  LOAD  2 LOCATION      (  10.00  ,    0.00  )
  DEFLECTION LOCATION   (   0.00  ,    0.00  )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

       DEVIATOR      ELASTIC
        STRESS       MODULUS
        (PSI)         (PSI)
         1.00        22867.
         2.00        22400.
         5.00        16530.
         8.00        14442.
```

```
OVERLAY CHARACTERISTICS
************************

  OVERLAY TYPE                BONDED CRCP
  ELASTIC MODULUS, PSI          4500000.
  POISSON/S RATIO                    .15
  CONCRETE FLEX. STRENGTH, PSI  640.0

DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1           7000000.
          2          10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    1    ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 1.


            S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER   THICKNESS   POISSON/S  ELASTIC        TYPE OF
   NO.      (IN.)       RATIO    MODULUS        MATERIAL
                                  (PSI)
    1       VARIES      .150    4500000.          CRCP
    2        8.00       .150    4200000.          CRCP
    3        6.00       .200     500000.   STABILIZED BASE
    4   SEMI-INFINITE   .450       8617.        SUBGRADE


  PREDICTED LIFE OF ORIGINAL PAVEMENT
  (EQUIVALENT 18 KIP SINGLE AXLE LOADS)            5095202.
  REMAINING LIFE OF ORIGINAL PAVEMENT, PERCENT         21.5


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                    OVERLAY       CALCULATED
                   THICKNESS     FATIGUE LIFE
                     (IN.)       (EQUIVALENT
                                 18 KIP SAWL)
                      3.0           3375000
                      6.0           8833000
                      9.0          20752000
                     12.0          45005000
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    1    ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 1.
```

PLOT:

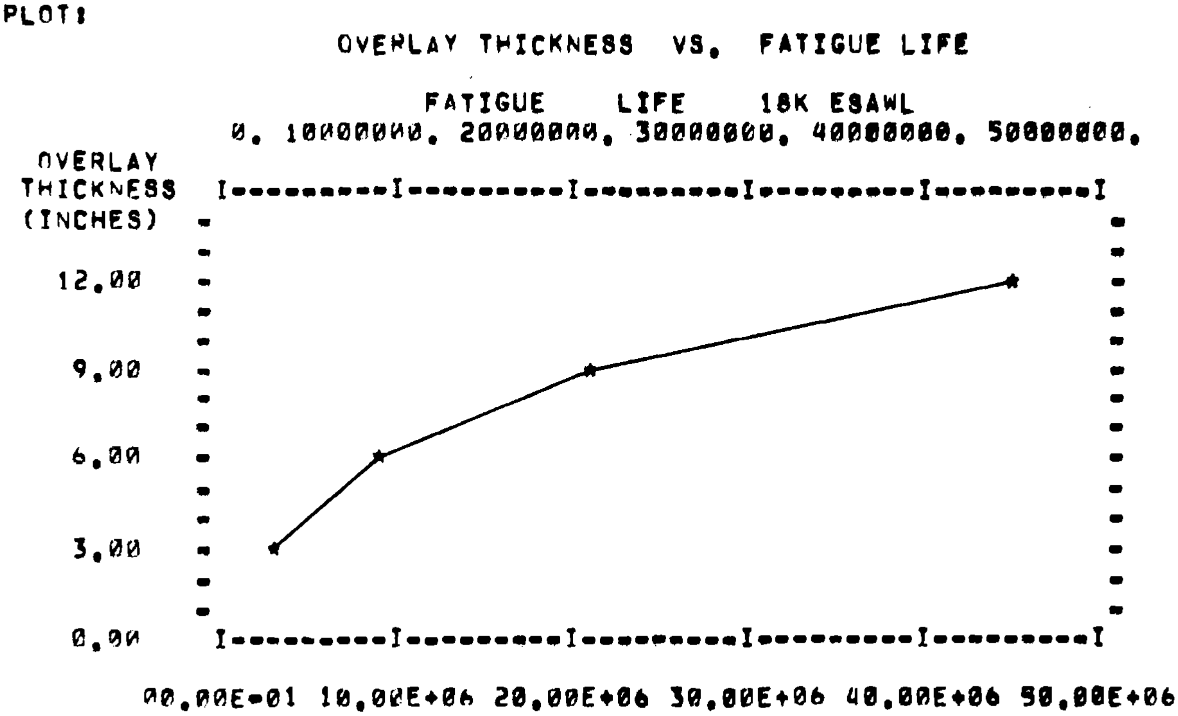


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	5.2
10000000	6.4

```
*******************************************************
*                                                     *
*  SINCE THIS EXISTING PAVEMENT HAS LESS THAN 25.0    *
*  PERCENT REMAINING LIFE, THE POSSIBILITY EXISTS     *
*  THAT A DESIGN NOT TAKING REMAINING LIFE INTO       *
*  CONSIDERATION MAY BE MORE ECONOMICAL.  THIS        *
*  ALTERNATIVE DESIGN IS THEREFORE PROVIDED ON THE    *
*  FOLLOWING PAGES.  IT IS RECOMMENDED THEN, THAT     *
*  THE THINNER OF THE TWO OVERLAY THICKNESSES BE      *
*  USED FOR DESIGN.                                   *
*                                                     *
*******************************************************
```

RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM 1 ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 1.

S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
****

PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE PREDICTIONS WERE MADE.

LAYER NO.	THICKNESS (IN.)	POISSON/S RATIO	ELASTIC MODULUS (PSI)	TYPE OF MATERIAL
1	VARIES	.150	4500000.	CRCP
2	1.00	.300	100000.	BOND BREAKER
3	8.00	.150	500000.	CRCP
4	6.00	.200	500000.	STABILIZED BASE
5	SEMI-INFINITE	.450	8617.	SUBGRADE

PREDICTED LIFE OF ORIGINAL PAVEMENT
(EQUIVALENT 18 KIP SINGLE AXLE LOADS) 5095202.
REMAINING LIFE OF ORIGINAL PAVEMENT, PERCENT 21.5

NOTE EXISTING PAVEMENT NOT CONSIDERED TO HAVE REMAINING LIFE, PROCEDURE REQUIRES A BOND BREAKER BETWEEN THE OVERLAY AND THE ORIGINAL PAVEMENT.

TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN PLOT ON NEXT PAGE.

OVERLAY THICKNESS (IN.)	CALCULATED FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)
3.0	1815000
6.0	6976000
9.0	23425000
12.0	64150000

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    1    ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 1.
```

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

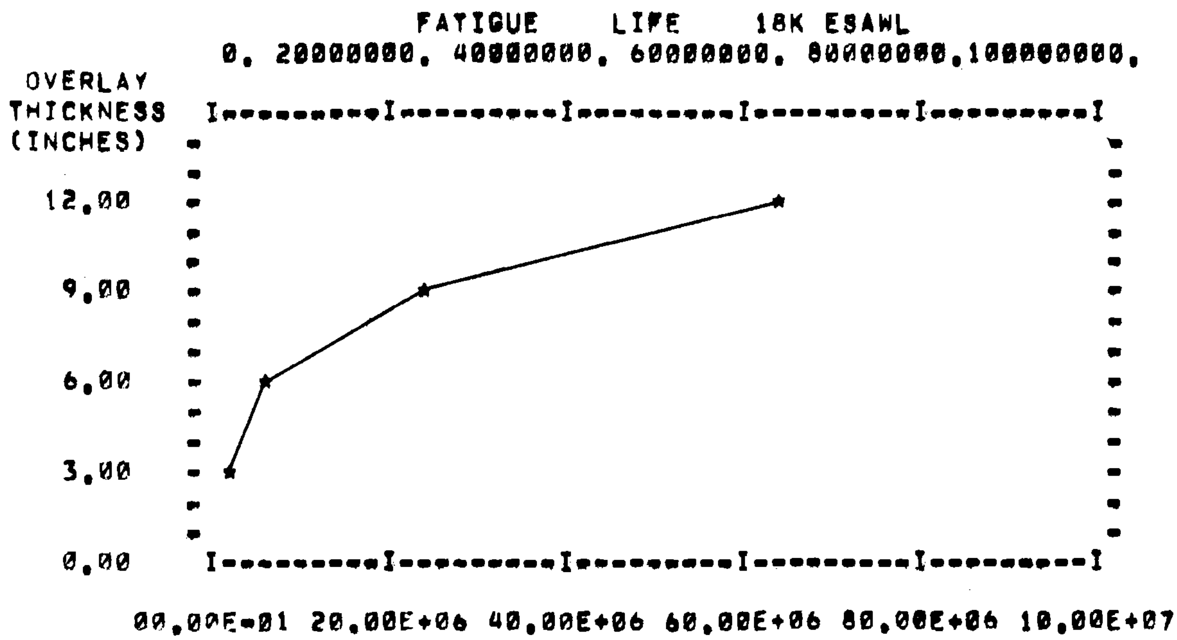


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	6.0
10000000	6.9

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   2   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 1.


          I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                     TYPE 1 AND 2 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                             570.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE              4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
    1       8.0       .150   4200000.          CRCP
    2       6.0       .200    500000.    STABILIZED BASE
    3  SEMI-INFINITE  .450      8617.        SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES        .0006230
  LOAD MAGNITUDE, POUNDS                       500.0
  TIRE PRESSURE, PSI                           167.0

                           X,Y COORDINATES, INCHES
  LOAD  1 LOCATION       ( -10.00  ,   0.00  )
  LOAD  2 LOCATION       (  10.00  ,   0.00  )
  DEFLECTION LOCATION    (   0.00  ,   0.00  )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

        DEVIATOR       ELASTIC
         STRESS        MODULUS
         (PSI)          (PSI)
          1.00         22867.
          2.00         22400.
          5.00         16530.
          8.00         14442.
```

```
OVERLAY CHARACTERISTICS
***********************

  OVERLAY TYPE                          AC
  ELASTIC MODULUS, PSI             400000.
  POISSON/S RATIO                      .30

DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1          7000000.
          2         10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   2   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 1.


          S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER   THICKNESS   POISSON/S  ELASTIC        TYPE OF
   NO.      (IN.)       RATIO    MODULUS        MATERIAL
                                  (PSI)
    1       VARIES      .300    400000.           AC
    2        8.00       .150   4200000.          CRCP
    3        6.00       .200    500000.   STABILIZED BASE
    4   SEMI-INFINITE   .450      8617.       SUBGRADE


  PREDICTED LIFE OF ORIGINAL PAVEMENT
  (EQUIVALENT 18 KIP SINGLE AXLE LOADS)          5095202.
  REMAINING LIFE OF ORIGINAL PAVEMENT, PERCENT       21.5


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                  OVERLAY      CALCULATED
                 THICKNESS    FATIGUE LIFE
                   (IN.)      (EQUIVALENT
                              18 KIP SAWL)
                    3.0          1831000
                    6.0          3310000
                    9.0          5999000
                   12.0         10616000
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   2   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 1.
```

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

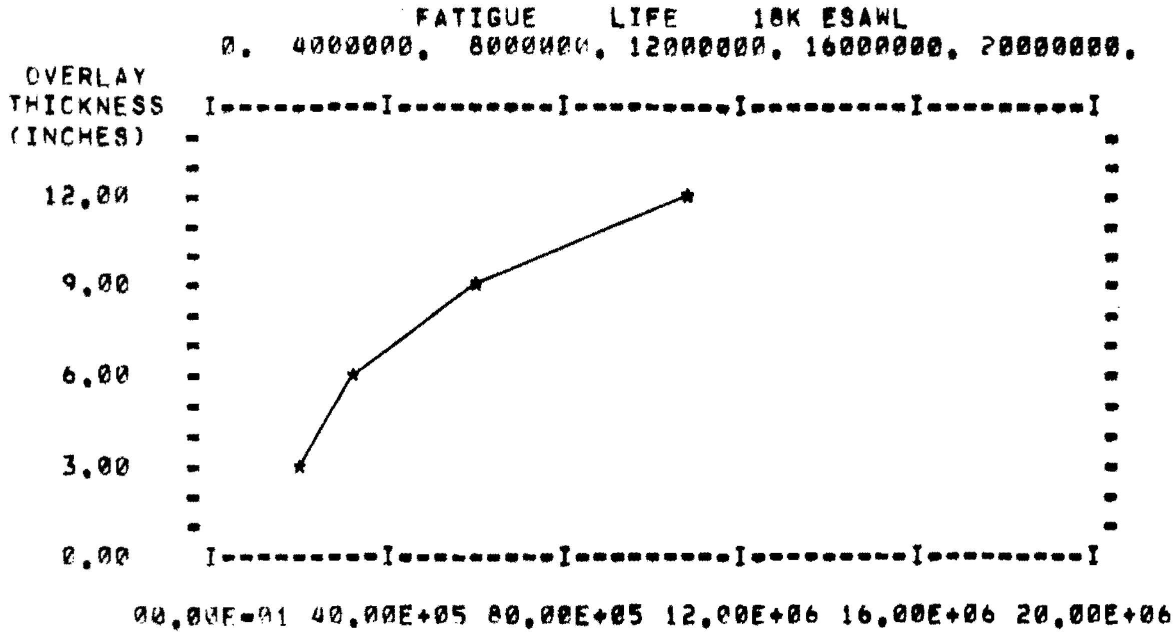


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	9.8
10000000	11.7

```
*******************************************************
*                                                     *
*  SINCE THIS EXISTING PAVEMENT HAS LESS THAN 25.0    *
*  PERCENT REMAINING LIFE, THE POSSIBILITY EXISTS     *
*  THAT A DESIGN NOT TAKING REMAINING LIFE INTO       *
*  CONSIDERATION MAY BE MORE ECONOMICAL.  THIS        *
*  ALTERNATIVE DESIGN IS THEREFORE PROVIDED ON THE    *
*  FOLLOWING PAGES.  IT IS RECOMMENDED THEN, THAT     *
*  THE THINNER OF THE TWO OVERLAY THICKNESSES BE      *
*  USED FOR DESIGN.                                   *
*                                                     *
*******************************************************
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   2   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 1.


            S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
    1      VARIES      .300    400000.          AC
    2  SEMI-INFINITE   .400     53030.  EQUIVALENT SUBGRADE


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                    OVERLAY     CALCULATED
                   THICKNESS   FATIGUE LIFE
                     (IN.)     (EQUIVALENT
                               18 KIP SAWL)
                      3.0           91000
                      6.0          929000
                      9.0         9783000
                     12.0        71954000
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   2   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 1.
```

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

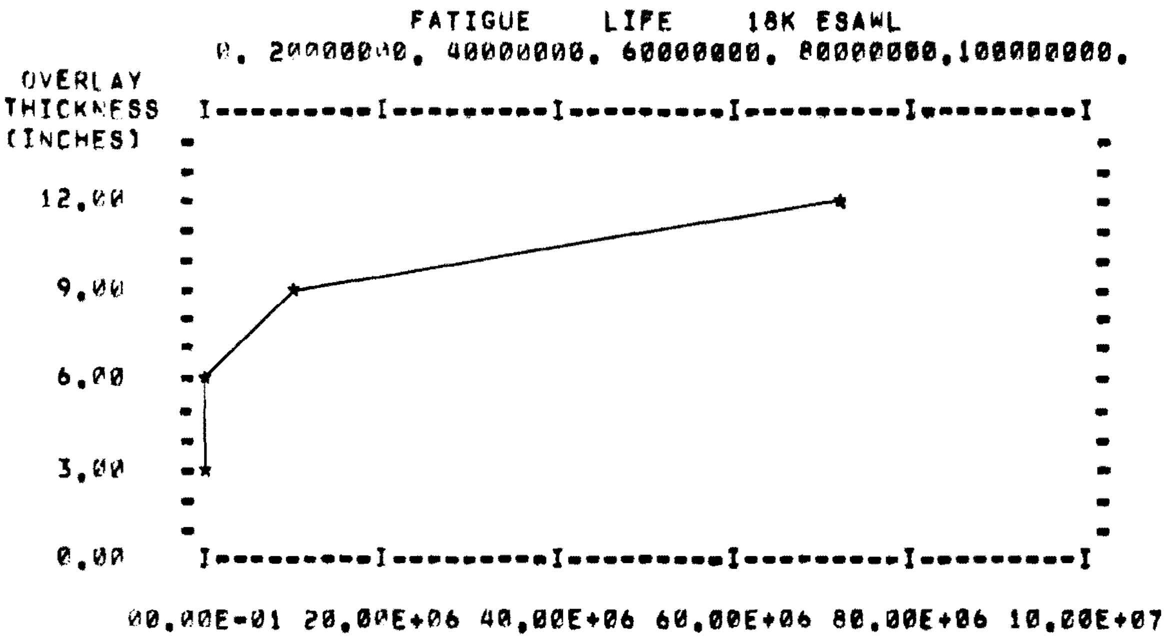


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	8.5
10000000	9.0

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    3   ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 2.


           I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                      TYPE 3 AND 4 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                               670.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE               4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
    1        8.0       .150    500000.         CRCP
    2        6.0       .200    500000.   STABILIZED BASE
    3   SEMI-INFINITE  .450      8617.       SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES        .0003870
  LOAD MAGNITUDE, POUNDS                       500.0
  TIRE PRESSURE, PSI                           167.0

                      X,Y COORDINATES, INCHES
  LOAD  1 LOCATION     ( -10.00  ,    0.00  )
  LOAD  2 LOCATION     (  10.00  ,    0.00  )
  DEFLECTION LOCATION  (   0.00  ,    0.00  )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

        DEVIATOR     ELASTIC
         STRESS      MODULUS
         (PSI)        (PSI)
          1.00       44642.
          2.00       29673.
          5.00       15686.
          8.00        5859.
```

```
OVERLAY CHARACTERISTICS
***********************

  OVERLAY TYPE                  BONDED CRCP
  ELASTIC MODULUS, PSI            4500000.
  POISSON/S RATIO                      .15
  CONCRETE FLEX. STRENGTH, PSI  640.0


DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1          7000000.
          2         10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   3   ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 2.


            S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER  THICKNESS  POISSON/S  ELASTIC      TYPE OF
   NO.     (IN.)      RATIO    MODULUS      MATERIAL
                                (PSI)
    1      VARIES      .150   4500000.        CRCP
    2       1.00       .300    100000.    BOND BREAKER
    3       8.00       .150    500000.        CRCP
    4       6.00       .200    500000.  STABILIZED BASE
    5   SEMI-INFINITE  .450     10680.      SUBGRADE

*NOTE*   EXISTING PAVEMENT NOT CONSIDERED TO HAVE REMAINING LIFE.


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                     OVERLAY     CALCULATED
                    THICKNESS    FATIGUE LIFE
                      (IN.)      (EQUIVALENT
                                 18 KIP SAWL)
                       3.0          1783000
                       6.0          7124000
                       9.0         24591000
                      12.0         68554000
```

RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM 3 ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 2.

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

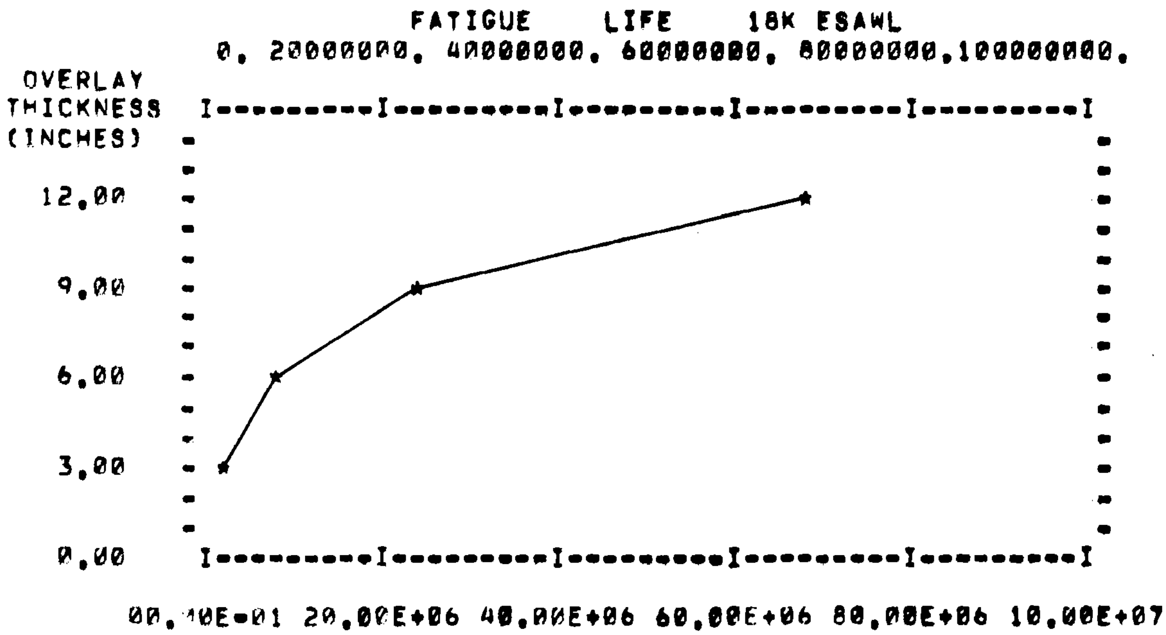


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	6.0
10000000	6.8

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   4   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 2.


           I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                      TYPE 3 AND 4 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                              670.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE              4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
   1        8.0        .150    500000.         CRCP
   2        6.0        .200    500000.   STABILIZED BASE
   3   SEMI-INFINITE   .450     10680.       SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES          .0003870
  LOAD MAGNITUDE, POUNDS                         500.0
  TIRE PRESSURE, PSI                             167.0

                         X,Y COORDINATES, INCHES
  LOAD  1 LOCATION       ( -10.00  ,    0.00  )
  LOAD  2 LOCATION       (  10.00  ,    0.00  )
  DEFLECTION LOCATION    (   0.00  ,    0.00  )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

        DEVIATOR      ELASTIC
         STRESS       MODULUS
         (PSI)         (PSI)
          1.00        44642.
          2.00        29673.
          5.00        15686.
          8.00         5859.
```

```
OVERLAY CHARACTERISTICS
***********************

  OVERLAY TYPE                        AC
  ELASTIC MODULUS, PSI           400000.
  POISSON/S RATIO                    .30

DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1          7000000.
          2         10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   4   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 2.


           S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER  THICKNESS  POISSON/S  ELASTIC      TYPE OF
   NO.     (IN.)      RATIO    MODULUS      MATERIAL
                                (PSI)
    1      VARIES      .300    400000.         AC
    2  SEMI-INFINITE   .400     61252.  EQUIVALENT SUBGRADE


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                   OVERLAY     CALCULATED
                  THICKNESS    FATIGUE LIFE
                    (IN.)      (EQUIVALENT
                               18 KIP SAWL)
                     3.0          154000
                     6.0         1390000
                     9.0        14086000
                    12.0       101381000
```

RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM 4 ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 2.

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

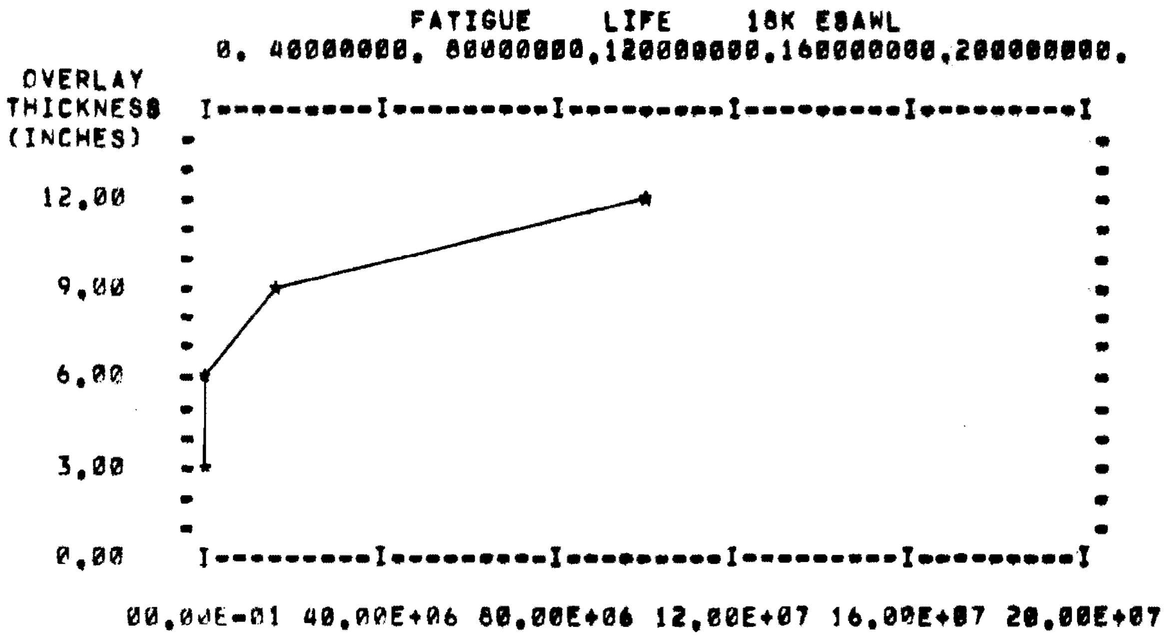


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	8.0
10000000	8.5

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   5   ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 3.


            I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                  TYPE 1 AND 2 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                          610.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE          4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC      TYPE OF
   NO.     (IN.)      RATIO    MODULUS      MATERIAL
                                (PSI)
    1        8.0        .150   3200000.        CRCP
    2        4.0        .200    250000.  STABILIZED BASE
    3  SEMI-INFINITE    .450     10680.      SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES         .0007720
  LOAD MAGNITUDE, POUNDS                        500.0
  TIRE PRESSURE, PSI                            167.0

                         X,Y COORDINATES, INCHES
  LOAD  1 LOCATION       ( -10.00  ,    0.00  )
  LOAD  2 LOCATION       (  10.00  ,    0.00  )
  DEFLECTION LOCATION    (   0.00  ,    0.00  )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

        DEVIATOR      ELASTIC
         STRESS       MODULUS
         (PSI)         (PSI)
          1.00        34300.
          2.00        30489.
          5.00        28583.
          8.00        22866.
```

```
OVERLAY CHARACTERISTICS
************************

  OVERLAY TYPE                BONDED CRCP
  ELASTIC MODULUS, PSI          4500000.
  POISSON/S RATIO                    .15
  CONCRETE FLEX. STRENGTH, PSI     640.0


DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1          7000000.
          2         10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    5   ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 3.


           S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER   THICKNESS   POISSON/S  ELASTIC       TYPE OF
   NO.      (IN.)       RATIO    MODULUS       MATERIAL
                                  (PSI)
    1       VARIES       .150   4500000.         CRCP
    2        1.00        .300    100000.     BOND BREAKER
    3        8.00        .150    500000.         CRCP
    4        4.00        .200    250000.   STABILIZED BASE
    5  SEMI-INFINITE     .450      9302.       SUBGRADE


  PREDICTED LIFE OF ORIGINAL PAVEMENT
  (EQUIVALENT 18 KIP SINGLE AXLE LOADS)          3140490.
  REMAINING LIFE OF ORIGINAL PAVEMENT, PERCENT      -27.4

*NOTE*   EXISTING PAVEMENT NOT CONSIDERED TO HAVE REMAINING LIFE,
         PROCEDURE REQUIRES A BOND BREAKER BETWEEN THE OVERLAY AND
         THE ORIGINAL PAVEMENT.


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                         OVERLAY      CALCULATED
                        THICKNESS    FATIGUE LIFE
                          (IN.)      (EQUIVALENT
                                     18 KIP SAWL)
                            3.0         1307000
                            6.0         4535000
                            9.0        15398000
                           12.0        43616000
```

RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM 5 ILLUSTRATIVE DESIGN PROBLEM,CRCP OVERLAY,SECTION 3.

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

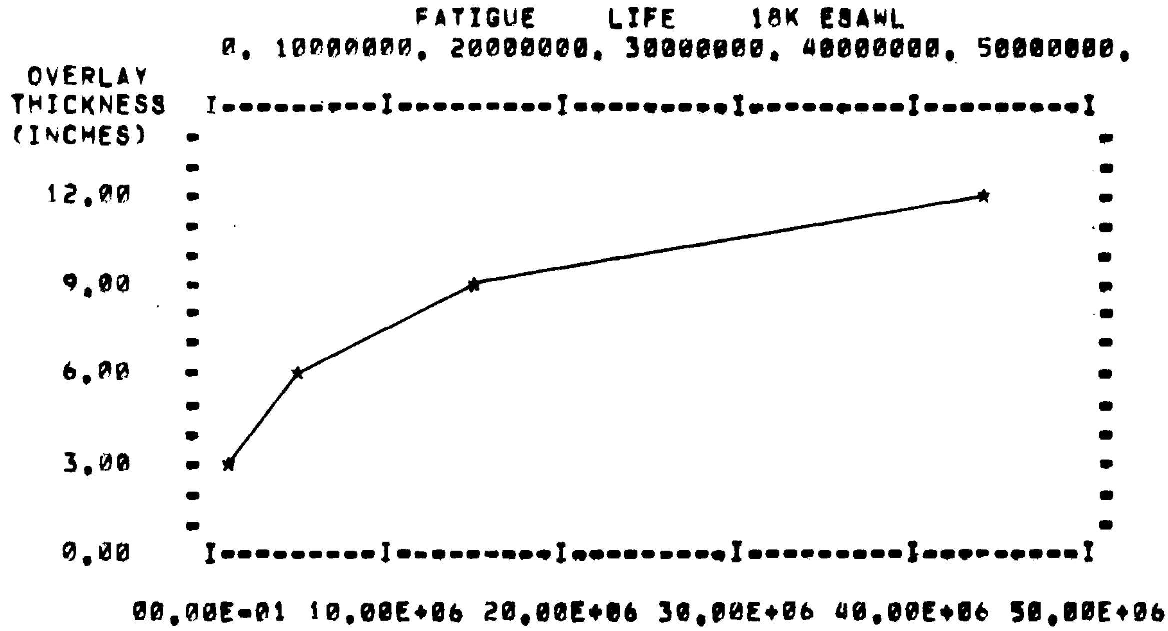


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	7.1
10000000	7.9

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM   6   ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 3.


            I N P U T   V A R I A B L E S

EXISTING PAVEMENT
*****************

  CONDITION                TYPE 1 AND 2 CRACKING WITH NO VOIDS
  CONCRETE FLEXURAL STRENGTH, PSI                         610.0
  EQUIVALENT 18 KIP SINGLE AXLE LOADS TO DATE          4000000.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
    1        8.0       .150   3200000.         CRCP
    2        4.0       .200    250000.   STABILIZED BASE
    3  SEMI-INFINITE   .450      9302.       SUBGRADE


DEFLECTION DATA
***************

  INTERIOR DESIGN DEFLECTION, INCHES         .0007720
  LOAD MAGNITUDE, POUNDS                        500.0
  TIRE PRESSURE, PSI                            167.0

                            X,Y COORDINATES, INCHES
  LOAD  1 LOCATION      ( -10.00  ,    0.00   )
  LOAD  2 LOCATION      (  10.00  ,    0.00   )
  DEFLECTION LOCATION   (   0.00  ,    0.00   )


LABORATORY TESTS OF SUBGRADE SAMPLES
************************************


  DATA DETERMINED FROM REPETITIVE LOAD TRIAXIAL TESTING
  MEAN SUBGRADE MODULUS FOR EACH DEVIATOR STRESS.

        DEVIATOR      ELASTIC
         STRESS       MODULUS
         (PSI)         (PSI)
          1.00        34300.
          2.00        30489.
          5.00        28583.
          8.00        22866.
```

```
OVERLAY CHARACTERISTICS
***********************

  OVERLAY TYPE                          AC
  ELASTIC MODULUS, PSI             400000.
  POISSON/S RATIO                      .30

DESIGN TRAFFIC
**************

  EQUIVALENT 18 KIP SINGLE AXLE LOADS ANTICIPATED ON OVERLAY.
  (TO BE USED IN CALCULATING CORRESPONDING REQUIRED OVERLAY
  THICKNESSES.)

          1          7000000.
          2         10000000.
```

```
RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM    6    ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 3.


            S Y S T E M   R E S U L T S

OVERLAY LIFE PREDICTIONS
*************************

  PAVEMENT SYSTEM DESCRIPTION FOR WHICH OVERLAY LIFE
  PREDICTIONS WERE MADE.

  LAYER  THICKNESS  POISSON/S  ELASTIC       TYPE OF
   NO.     (IN.)      RATIO    MODULUS       MATERIAL
                                (PSI)
    1      VARIES      .300    400000.          AC
    2  SEMI-INFINITE   .400     46334.  EQUIVALENT SUBGRADE


  TABLE OF OVERLAY THICKNESS VS. FATIGUE LIFE USED IN
  PLOT ON NEXT PAGE.

                   OVERLAY      CALCULATED
                  THICKNESS    FATIGUE LIFE
                    (IN.)      (EQUIVALENT
                               18 KIP SAWL)
                     3.0            56000
                     6.0           648000
                     9.0          7065000
                    12.0         53073000
```

RPOD2 - RIGID PAVEMENT OVERLAY DESIGN PROGRAM - VERSION 2.0
LATEST REVISION - APRIL 1978
CENTER FOR HIGHWAY RESEARCH, UNIV. OF TEXAS AT AUSTIN

PROBLEM 6 ILLUSTRATIVE DESIGN PROBLEM,AC OVERLAY,SECTION 3.

PLOT:

OVERLAY THICKNESS VS. FATIGUE LIFE

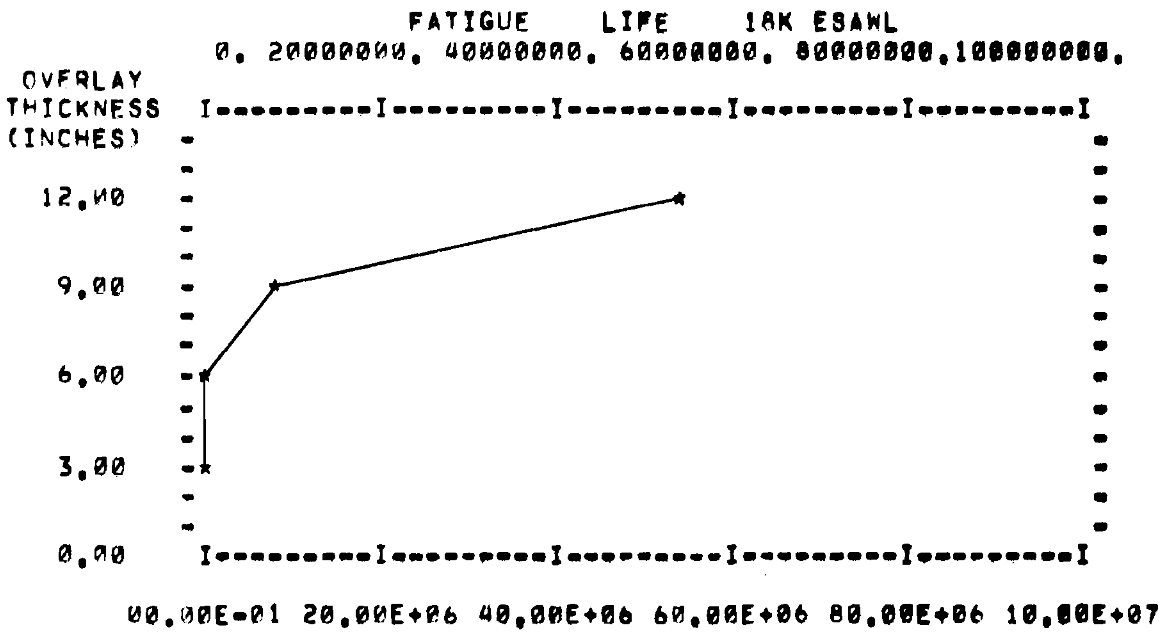


TABLE OF INTERPOLATED OVERLAY THICKNESSES FOR
REQUESTED DESIGN FATIGUE LIVES.

FATIGUE LIFE (EQUIVALENT 18 KIP SAWL)	INTERPOLATED THICKNESS (IN.)
7000000	9.0
10000000	9.5

```
**********************************************************************
     REFLECTION CRACKING INPUT VARIABLES
**********************************************************************


           **********************
           * EXISTING PAVEMENT *
           **********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM.
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 1.

PAVEMENT TYPE                                  CRCP
   CONDITION                                CRACKED
   CRACK SPACING, FT                           8.00

PAVEMENT PROPERTIES
   MODULUS, PSI                            4200000.
   THICKNESS, INCHES                           8.00
   DENSITY, PCF                               140.0
   THERMAL COEFFICIENT, PER DEGREE F      .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                           29000000.
   AREA, SQUARE INCHES                          .48
   PERIMETER OF STEEL, INCHES                  3.19
   BONDING STRESS, PSI                        295.0
   THERMAL COEFFICIENT, PER DEGREE F      .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES          .0200


           *********************
           * CHARACTERIZATION *
           *********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                 80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES .02100
   LOW TEMPERATURE, DEGREES F                  70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES  .02500
   MIN. TEMPERATURE OBSERVED, DEGREES F         13.

VERTICAL
   JOINT WIDTH, INCHES                         .023
   LOAD TRANSFER, PERCENT                      92.0
```

```
             ************
             * OVERLAY *
             ************

OVERLAY TYPE                                        AC

   CREEP MODULUS, PSI                          250000.
   DYNAMIC MODULUS, PSI                        400000.
   THICKNESS, INCHES                              8.50
   DENSITY, PCF                                  136.0
   POISSONS RATIO                                 .300
   THERMAL COEFFICIENT, PER DEGREE F          .0000120
   BONDING STRESS, PSI                           500.0

BOND BREAKER WIDTH, FEET                           2.0


          ***********************
          * OTHER DESIGN INPUTS *
          ***********************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                               85.
   OVERLAY                                         97.

DESIGN LOAD
   WEIGHT, POUNDS                              18000.0
   WIDTH, INCHES                                  28.0
```

```
*****************************************************************************
     REFLECTION CRACKING PROGRAM OUTPUT
*****************************************************************************


            ***************
            * BETA VALUES *
            ***************

   BEFORE OVERLAY                                      .7635
   AFTER OVERLAY
     BONDED                                            .7791
     UNBONDED                                          .7374


       ***************************
       * SLOPE OF FRICTION CURVE *
       ***************************

   BEFORE OVERLAY                                  1.319E+05
   AFTER OVERLAY                                   1.319E+05


        ************************
        *   MAXIMUM STRESSES   *
        * IN EXISTING PAVEMENT *
        *        (PSI)         *
        ************************

   CONCRETE, BEFORE OVERLAY                            594.4

   STEEL, BEFORE OVERLAY                             39860.7
          AFTER OVERLAY                              22262.2


          *********************
          *********************
          ** OVERLAY STRAINS **
          **      IN/IN      **
          *********************
          *********************

   SHEAR                                           3.933E-05
   TENSILE                                         1.810E-03
```

```
***********************************************************************
     REFLECTION CRACKING INPUT VARIABLES
***********************************************************************


          *********************
          * EXISTING PAVEMENT *
          *********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM,
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 1.

PAVEMENT TYPE                                    CRCP
   CONDITION                                  CRACKED
   CRACK SPACING, FT                             8.00

PAVEMENT PROPERTIES
   MODULUS, PSI                              4200000.
   THICKNESS, INCHES                             8.00
   DENSITY, PCF                                 140.0
   THERMAL COEFFICIENT, PER DEGREE F  .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                             29000000.
   AREA, SQUARE INCHES                            .48
   PERIMETER OF STEEL, INCHES                    3.19
   BONDING STRESS, PSI                          295.0
   THERMAL COEFFICIENT, PER DEGREE F  .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES      .0200


          ********************
          * CHARACTERIZATION *
          ********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                   80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES .02100
   LOW TEMPERATURE, DEGREES F                    70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES  .02500
   MIN. TEMPERATURE OBSERVED, DEGREES F    13.

VERTICAL
   JOINT WIDTH, INCHES                           .023
   LOAD TRANSFER, PERCENT                        92.0
```

```
                 ************
                 * OVERLAY *
                 ************

OVERLAY TYPE                                        AC

   CREEP MODULUS, PSI                          250000.
   DYNAMIC MODULUS, PSI                        400000.
   THICKNESS, INCHES                              9.00
   DENSITY, PCF                                  136.0
   POISSONS RATIO                                 .300
   THERMAL COEFFICIENT, PER DEGREE F          .0000120
   BONDING STRESS, PSI                           500.0

BOND BREAKER WIDTH, FEET                           2.0


           ***************************
           * OTHER DESIGN INPUTS *
           ***************************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                               85.
   OVERLAY                                         97.

DESIGN LOAD
   WEIGHT, POUNDS                              18000.0
   WIDTH, INCHES                                  28.0
```

```
*********************************************************************
    REFLECTION CRACKING PROGRAM OUTPUT
*********************************************************************


            ***************
            * BETA VALUES *
            ***************

   BEFORE OVERLAY                                    .7635
   AFTER OVERLAY
     BONDED                                          .7811
     UNBONDED                                        .7374


        ***************************
        * SLOPE OF FRICTION CURVE *
        ***************************

   BEFORE OVERLAY                                1.319E+05
   AFTER OVERLAY                                 1.319E+05


         ************************
         *   MAXIMUM STRESSES   *
         * IN EXISTING PAVEMENT *
         *        (PSI)         *
         ************************

   CONCRETE, BEFORE OVERLAY                          594.4

   STEEL, BEFORE OVERLAY                           39860.7
          AFTER OVERLAY                            22242.5


           *********************
           *********************
           ** OVERLAY STRAINS **
           **      IN/IN      **
           *********************
           *********************

   SHEAR                                         3.714E-05
   TENSILE                                       1.796E-03
```

```
**********************************************************************
     REFLECTION CRACKING INPUT VARIABLES
**********************************************************************


          *********************
          * EXISTING PAVEMENT *
          *********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM.
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 2.

PAVEMENT TYPE                                        CRCP
   CONDITION                                      CRACKED
   CRACK SPACING, FT                                 8.20

PAVEMENT PROPERTIES
   MODULUS, PSI                                  3800000.
   THICKNESS, INCHES                                 8.00
   DENSITY, PCF                                     140.0
   THERMAL COEFFICIENT, PER DEGREE F             .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                                 29000000.
   AREA, SQUARE INCHES                                .48
   PERIMETER OF STEEL, INCHES                        3.19
   BONDING STRESS, PSI                              295.0
   THERMAL COEFFICIENT, PER DEGREE F             .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES                .0200


          ********************
          * CHARACTERIZATION *
          ********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                       80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES            .02700
   LOW TEMPERATURE, DEGREES F                        70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES             .03100
   MIN. TEMPERATURE OBSERVED, DEGREES F               13.

VERTICAL
   JOINT WIDTH, INCHES                               .029
   LOAD TRANSFER, PERCENT                            83.0
```

```
                ***********
                * OVERLAY *
                ***********

OVERLAY TYPE                                              AC

   CREEP MODULUS, PSI                                250000.
   DYNAMIC MODULUS, PSI                              400000.
   THICKNESS, INCHES                                    8.00
   DENSITY, PCF                                        136.0
   POISSONS RATIO                                       .300
   THERMAL COEFFICIENT, PER DEGREE F                .0000120
   BONDING STRESS, PSI                                 500.0

BOND BREAKER WIDTH, FEET                                 2.0


          ***********************
          * OTHER DESIGN INPUTS *
          ***********************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                                     85.
   OVERLAY                                               97.

DESIGN LOAD
   WEIGHT, POUNDS                                    18000.0
   WIDTH, INCHES                                        28.0
```

```
****************************************************************************
     REFLECTION CRACKING PROGRAM OUTPUT
****************************************************************************


            ***************
            * BETA VALUES *
            ***************

   BEFORE OVERLAY                                   .7750
   AFTER OVERLAY
     BONDED                                         .7883
     UNBONDED                                       .7466


       ***************************
       * SLOPE OF FRICTION CURVE *
       ***************************

   BEFORE OVERLAY                               1.234E+05
   AFTER OVERLAY                                1.234E+05


        ************************
        *    MAXIMUM STRESSES  *
        * IN EXISTING PAVEMENT *
        *         (PSI)        *
        ************************

   CONCRETE, BEFORE OVERLAY                         587.7

   STEEL, BEFORE OVERLAY                          42392.5
          AFTER OVERLAY                           23169.0


          *********************
          *********************
          ** OVERLAY STRAINS **
          **      IN/IN      **
          *********************
          *********************

   SHEAR                                        8.879E-05
   TENSILE                                      1.833E-03
```

```
*****************************************************************
     REFLECTION CRACKING INPUT VARIABLES
*****************************************************************


          *********************
          * EXISTING PAVEMENT *
          *********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM.
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 2.

PAVEMENT TYPE                                      CRCP
   CONDITION                                    CRACKED
   CRACK SPACING, FT                               8.20

PAVEMENT PROPERTIES
   MODULUS, PSI                                3800000.
   THICKNESS, INCHES                               8.00
   DENSITY, PCF                                   140.0
   THERMAL COEFFICIENT, PER DEGREE F           .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                               29000000.
   AREA, SQUARE INCHES                              .48
   PERIMETER OF STEEL, INCHES                      3.19
   BONDING STRESS, PSI                            295.0
   THERMAL COEFFICIENT, PER DEGREE F           .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES              .0200


          ********************
          * CHARACTERIZATION *
          ********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                     80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES          .02700
   LOW TEMPERATURE, DEGREES F                      70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES           .03100
   MIN. TEMPERATURE OBSERVED, DEGREES F             13.

VERTICAL
   JOINT WIDTH, INCHES                             .029
   LOAD TRANSFER, PERCENT                          83.0
```

```
           ************
           * OVERLAY *
           ************

OVERLAY TYPE                                        AC

   CREEP MODULUS, PSI                           250000.
   DYNAMIC MODULUS, PSI                         400000.
   THICKNESS, INCHES                              8.50
   DENSITY, PCF                                  136.0
   POISSONS RATIO                                 .300
   THERMAL COEFFICIENT, PER DEGREE F          .0000120
   BONDING STRESS, PSI                           500.0

BOND BREAKER WIDTH, FEET                           2.0


     ************************
     * OTHER DESIGN INPUTS *
     ************************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                               85.
   OVERLAY                                         97.

DESIGN LOAD
   WEIGHT, POUNDS                              18000.0
   WIDTH, INCHES                                  28.0
```

```
*****************************************************************
    REFLECTION CRACKING PROGRAM OUTPUT
*****************************************************************


          ***************
          * BETA VALUES *
          ***************

  BEFORE OVERLAY                                    .7750
  AFTER OVERLAY
    BONDED                                          .7911
    UNBONDED                                        .7466


      ***************************
      * SLOPE OF FRICTION CURVE *
      ***************************

  BEFORE OVERLAY                                1.234E+05
  AFTER OVERLAY                                 1.234E+05


       ************************
       *   MAXIMUM STRESSES   *
       * IN EXISTING PAVEMENT *
       *        (PSI)         *
       ************************

  CONCRETE, BEFORE OVERLAY                          587.7

  STEEL, BEFORE OVERLAY                           42392.5
         AFTER OVERLAY                            23141.2


         *********************
         *********************
         ** OVERLAY STRAINS **
         **      IN/IN      **
         *********************
         *********************

  SHEAR                                         8.357E-05
  TENSILE                                       1.814E-03
```

```
*****************************************************************
     REFLECTION CRACKING INPUT VARIABLES
*****************************************************************


          *********************
          * EXISTING PAVEMENT *
          *********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM,
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 3.

PAVEMENT TYPE                                       CRCP
   CONDITION                                     CRACKED
   CRACK SPACING, FT                               17.80

PAVEMENT PROPERTIES
   MODULUS, PSI                                 4500000.
   THICKNESS, INCHES                                8.00
   DENSITY, PCF                                    140.0
   THERMAL COEFFICIENT, PER DEGREE F            .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                                29000000.
   AREA, SQUARE INCHES                               .48
   PERIMETER OF STEEL, INCHES                       3.19
   BONDING STRESS, PSI                             295.0
   THERMAL COEFFICIENT, PER DEGREE F            .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES               .0200


          ********************
          * CHARACTERIZATION *
          ********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                      80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES           .01600
   LOW TEMPERATURE, DEGREES F                       70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES            .02600
   MIN. TEMPERATURE OBSERVED, DEGREES F              13.

VERTICAL
   JOINT WIDTH, INCHES                              .021
   LOAD TRANSFER, PERCENT                           87.0
```

```
            ************
            * OVERLAY *
            ************

OVERLAY TYPE                                          AC

   CREEP MODULUS, PSI                            250000.
   DYNAMIC MODULUS, PSI                          400000.
   THICKNESS, INCHES                                9.00
   DENSITY, PCF                                    136.0
   POISSONS RATIO                                   .300
   THERMAL COEFFICIENT, PER DEGREE F            .0000120
   BONDING STRESS, PSI                             500.0

BOND BREAKER WIDTH, FEET                             4.0


         ************************
         * OTHER DESIGN INPUTS *
         ************************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                                 85.
   OVERLAY                                           97.

DESIGN LOAD
   WEIGHT, POUNDS                                18000.0
   WIDTH, INCHES                                    28.0
```

```
************************************************************************
     REFLECTION CRACKING PROGRAM OUTPUT
************************************************************************


            ***************
            * BETA VALUES *
            ***************

   BEFORE OVERLAY                                      .7400
   AFTER OVERLAY
     BONDED                                            .7669
     UNBONDED                                          .6510


       ***************************
       * SLOPE OF FRICTION CURVE *
       ***************************

   BEFORE OVERLAY                                  1.394E+04
   AFTER OVERLAY                                   1.394E+04


        *************************
        *   MAXIMUM STRESSES    *
        * IN EXISTING PAVEMENT  *
        *         (PSI)         *
        *************************

   CONCRETE, BEFORE OVERLAY                            442.9

   STEEL, BEFORE OVERLAY                             50561.5
          AFTER OVERLAY                              20680.0


          *********************
          *********************
          ** OVERLAY STRAINS **
          **      IN/IN      **
          *********************
          *********************

   SHEAR                                           6.036E-05
   TENSILE                                         2.111E-03
```

```
**********************************************************************
     REFLECTION CRACKING INPUT VARIABLES
**********************************************************************


          *********************
          * EXISTING PAVEMENT *
          *********************

LOCATION
ILLUSTRATIVE OVERLAY DESIGN PROBLEM.
ASPHALTIC CONCRETE OVERLAY ON CRCP, SECTION 3.

PAVEMENT TYPE                                      CRCP
   CONDITION                                    CRACKED
   CRACK SPACING, FT                              17.80

PAVEMENT PROPERTIES
   MODULUS, PSI                                4500000.
   THICKNESS, INCHES                               8.00
   DENSITY, PCF                                   140.0
   THERMAL COEFFICIENT, PER DEGREE F           .0000060

REINFORCEMENT PROPERTIES
   MODULUS, PSI                               29000000.
   AREA, SQUARE INCHES                              .48
   PERIMETER OF STEEL, INCHES                      3.19
   BONDING STRESS, PSI                            295.0
   THERMAL COEFFICIENT, PER DEGREE F           .0000060

PAVEMENT MOVEMENT AT SLIDING, INCHES              .0200


          ********************
          * CHARACTERIZATION *
          ********************

HORIZONTAL
   HIGH TEMPERATURE, DEGREES F                     80.0
   HIGH TEMPERATURE JOINT WIDTH, INCHES          .01600
   LOW TEMPERATURE, DEGREES F                      70.0
   LOW TEMPERATURE JOINT WIDTH, INCHES           .02600
   MIN. TEMPERATURE OBSERVED, DEGREES F             13.

VERTICAL
   JOINT WIDTH, INCHES                             .021
   LOAD TRANSFER, PERCENT                          87.0
```

```
************
* OVERLAY *
************

OVERLAY TYPE                                            AC

   CREEP MODULUS, PSI                              250000.
   DYNAMIC MODULUS, PSI                            400000.
   THICKNESS, INCHES                                 9.50
   DENSITY, PCF                                      136.0
   POISSONS RATIO                                     .300
   THERMAL COEFFICIENT, PER DEGREE F              .0000120
   BONDING STRESS, PSI                               500.0

BOND BREAKER WIDTH, FEET                               4.0


************************
* OTHER DESIGN INPUTS *
************************

DESIGN TEMPERATURE CHANGES, DEGREES F
   EXISTING PAVEMENT                                   85.
   OVERLAY                                             97.

DESIGN LOAD
   WEIGHT, POUNDS                                  18000.0
   WIDTH, INCHES                                      28.0
```

```
**********************************************************************
     REFLECTION CRACKING PROGRAM OUTPUT
**********************************************************************


            ***************
            * BETA VALUES *
            ***************

   BEFORE OVERLAY                                     .7400
   AFTER OVERLAY
     BONDED                                           .7705
     UNBONDED                                         .6510


       ***************************
       * SLOPE OF FRICTION CURVE *
       ***************************

   BEFORE OVERLAY                                 1.394E+04
   AFTER OVERLAY                                  1.394E+04


        ************************
        *   MAXIMUM STRESSES   *
        * IN EXISTING PAVEMENT *
        *        (PSI)         *
        ************************

   CONCRETE, BEFORE OVERLAY                           442.9

   STEEL, BEFORE OVERLAY                            50561.5
          AFTER OVERLAY                             20643.7


          *********************
          *********************
          ** OVERLAY STRAINS **
          **      IN/IN      **
          *********************
          *********************

   SHEAR                                          5.718E-05
   TENSILE                                        2.093E-03
```

APPENDIX 2

SELECTION OF PRACTICAL RANGE FOR THE SLOPE OF THE LOG RESILIENT MODULUS VERSUS LOG DEVIATOR STRESS LINE (S_{SG}) FOR TYPICAL SUBGRADE SOILS

APPENDIX 2. SELECTION OF A PRACTICAL RANGE FOR THE SLOPE OF THE LOG RESILIENT MODULUS VERSUS THE LOG DEVIATOR STRESS LINE (S_{SG}) FOR TYPICAL SUBGRADE SOILS

In order to determine a practical range for S_{SG}, laboratory data made available by Austin Research Engineers, Inc., as well as data obtained from reports of the Corps of Engineers (Refs 17 and 18), have been used.

Results have been analyzed using linear regression to obtain S_{SG} with the following equation

$$\log M_R = a + S_{SG} \log (\sigma_{dev}) \qquad \text{A2.1}$$

where

M_R = Resilient Modulus, psi,

σ_{dev} = deviator stress, psi,

a = intercept on $\log M_R$-axis,

S_{SG} = slope of $\log M_R$ versus $\log (\sigma_{dev})$ line.

S_{SG} has been determined for each test considered in this analysis. Means and standard deviations of S_{SG} have been determined for each type of soil on each project.

Since no correlation could be found between S_{SG} and material type with the information available, it was decided to group all cohesive materials together to determine a range for S_{SG}. A summary of results of S_{SG} can be seen on Table A2.1.

It was assumed that values of S_{SG} would be normally distributed and the range has been obtained by calculating the overall mean and standard deviation as follows.

<u>Weighted Mean (Ref 7)</u>:

$$\bar{\bar{x}} = \Sigma \frac{f\bar{x}_i}{\Sigma f} \qquad \text{A2.2}$$

TABLE A2.1. SUMMARY OF RESULTS FOR COHESIVE MATERIALS

Project	S_{SG} Mean	S_{SG} Standard Deviation	S_{SG} Number of Tests
Tulsa International Airport	-.8643	.3000	10
Randolph AFB, Texas	-.2768	.1434	30
Corps of Engineers	-.3301	.0709	6
Adamsfield, Little Rock	-.4276	.3492	6
Memorial Field, Hot Springs	-.6147	.2924	6
Houdaille Plant, Pearland, TX	-.4715	.2229	2
Houdaille Plant, Pearland, TX	-.3394	.0557	2
AASHO Road Test	-1.0764	.2344	14

where

$\bar{\bar{x}}$ = weighted mean of all projects,

f = frequency of occurence in each project,

$\bar{x}_i$ = mean value for each project.

Total Standard Deviation (Ref 21):

$$\sigma = \sqrt{(\sigma_W)^2 + (\sigma_B)^2} \qquad \text{A2.3}$$

where

σ = total standard deviation,

σ_W = within-project standard deviation,

σ_B = between-project standard deviation.

$$\sigma_B^{\ 2} = \frac{1}{N-1} \sum^{L} n_h(\bar{x}_h - \bar{\bar{x}})^2 \qquad \text{A2.4}$$

where

n_h = number of tests conducted on project h,

N = total number of tests conducted on all projects,

$\bar{x}_h$ = mean of project h, and

all other variables are as previously defined.

and

$$\sigma_W^{\ 2} = \frac{1}{N-1} \sum^{L} \sigma_n^{\ 2} (n_h - 1) \qquad \text{A2.5}$$

where

σ_h = standard deviation of project h and

all other variables are as previously defined.

Using these equations (A2.2 to A2.5) and analyzing the data in Table A2.1 an overall mean S_{SG} of -0.55 and a total standard deviation of 0.38 has been found. For a 90 percent confidence level the confidence interval of $-1.17 \leq S_{SG} \leq +0.07$ has been determined.

APPENDIX 3.

EQUATIONS USED IN THE COMPARISON STUDY BETWEEN RPOD1 AND A SIMPLIFIED METHOD, USING WESTERGAARD EQUATIONS FOR CALCULATION OF STRESS AND DEFLECTIONS

APPENDIX 3. EQUATIONS USED IN THE COMPARISON STUDY BETWEEN RPOD1 AND A SIMPLIFIED METHOD, USING WESTERGAARD EQUATIONS FOR CALCULATION OF STRESS AND DEFLECTIONS

Equations used in the simplified method to predict overlay thicknesses, outlined in Chapter II - 2, are given here.

WESTERGAARD EQUATIONS

The Westergaard equations used in this analysis are those given by Westergaard (Ref 19) and in class notes in the CE391 P.1 course: Pavement Systems - Theory.

Corner Condition

Stress.

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{L}\right)^{0.6}\right] \qquad \text{A3.1}$$

where

σ_c = corner stress in psi,

P = wheel load in pounds,

h = pavement thickness in inches,

a_1 = $\sqrt{2} \times a$

a = load radius in inches,

$$L = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)\,k}}$$

where

E = modulus of elasticity of concrete in psi,

μ = Poisson's ratio,

k = modulus of subgrade reaction psi/in.

Deflection.

$$W_c = (1.1 - 0.88 \frac{a_1}{L}) \frac{P}{kL^2} \qquad \text{A3.2}$$

where

W_c = deflection at the corner (inches) and

all other variables are as defined above.

Interior Conditions

Stress.

$$\sigma_i = 0.31625 \frac{P}{h^2} \left[4 \log_{10}(\frac{L}{b}) + 1.0693\right] \qquad \text{A3.3}$$

where

σ_i = interior stress in psi,

$b = \sqrt{1.6a^2 + h^2} - 0.675h$

if $a < 1.724\ h$

and $a = b$

if $a > 1.724\ h$.

All other variables are as defined above.

Deflection.

$$W_i = \frac{P}{8kL^2} \qquad \text{A3.4}$$

where

W_i = interior deflection in inches and

all other variables are as previously defined.

Edge Condition

Stress.

$$\sigma_e = 0.57185 \frac{P}{h^2} \left[4 \log_{10}(\frac{L}{b}) + 0.3593\right] \qquad \text{A3.5}$$

where

> σ_e = edge stress in psi and
>
> all other variables are as defined above except for the fact that the load radius, a, is determined by considering the load to act on a half circular area with radius, a, on the edge of the pavement so that

$$\frac{\pi a^2}{2} = \frac{P}{p}$$

or

$$a = \sqrt{\frac{2P}{\pi p}}$$

where

> p = the contact pressure in psi.

<u>Deflection</u>.

$$W_e = 0.441 \frac{P}{kL^2} \qquad \text{for } \mu = 0.2 \qquad \text{A3.6}$$

where

> W_e = deflection at the edge in inches and
>
> all other variables are as previously defined.

EQUATIONS FOR DETERMINATION OF COMPOSITE k-VALUES

Kher et al. (Ref 4) developed statistical equations using layered theory to predict the composite k-value on top of a subbase.

The following were the equations developed by them.

For subbase with thickness of 0-6 inches:

$$K_T = 385.76 + 69.7\tau_1 + 8.59\tau_2 + 27.06\varepsilon_1 + 3.98\varepsilon_2 + 5.55\varepsilon_3$$

$$+ 66.48M_1 - 1.6M_2 + 0.43M_3 + 31.07\tau_1\varepsilon_1 + 4.41\tau_1\varepsilon_2 + 5.06\tau_1\varepsilon_3$$

$$+ 7.08\tau_1 M_1 - 2.35\tau_1 M_2 + 0.25\tau_1 M_3 + 4.01\tau_2\varepsilon_1 + 0.42\tau_2\varepsilon_2$$

$$+ 1.13\tau_2 M_1 + 3.56\epsilon_1 M_1 + 0.36\epsilon_2 M_1 - 0.20\epsilon_2 M_2 + 1.06\epsilon_3 M_1$$

$$+ 4.22\tau_1\epsilon_1 M_1 - 0.46\tau_1\epsilon_1 M_2 + 0.47\tau_1\epsilon_2 M_1 - 0.18\tau_1\epsilon_2 M_2$$

$$+ 0.66\tau_2\epsilon_1 M_1 + 0.11\tau_2\epsilon_2 M_1 + 0.13\epsilon_1 M_3 + 0.14\tau_1\epsilon_1 M_3 \qquad \text{(A3.7)}$$

for subbase with thickness of 6-12 inches:

$$K_T = 578.62 + 115.16\tau_1 + 0.59\tau_2 + 108.03\epsilon_1 + 13.39\epsilon_2 + 13.09\epsilon_3$$

$$+ 88.40M_1 - 7.09M_2 + 1.35M_3 + 45.94\tau_1\epsilon_1 + 4.57\tau_1\epsilon_2 + 2.92\tau_1\epsilon_3$$

$$+ 13.81\tau_1 M_1 - 3.00\tau_1 M_2 + 0.58\tau_1 M_3 + 15.36\epsilon_1 M_1 - 1.46\epsilon_1 M_2$$

$$+ 0.40\epsilon_1 M_3 + 1.55\epsilon_2 M_1 - 0.45\epsilon_2 M_2 + 0.07\epsilon_2 M_3 + 2.36\epsilon_3 M_1$$

$$+ 6.93\tau_1\epsilon_1 M_1 - 0.56\tau_1\epsilon_1 M_2 + 0.13\tau_1\epsilon_1 M_3 + 0.61\tau_1\epsilon_2 M_1$$

$$- 0.10\tau_1\epsilon_2 M_2 \qquad \text{(A3.8)}$$

for subbase with thickness of 12-18 inches

$$K_T = 810.62 + 115.99\tau_1 + 200.53\epsilon_1 + 23.21\epsilon_2 + 18.75\epsilon_3 + 116.50M_1$$

$$- 13.39M_2 + 2.66M_3 + 46.54\tau_1\epsilon_1 + 5.35\tau_1\epsilon_2 + 2.75\tau_1\epsilon_3 + 14.19\tau_1 M_1$$

$$- 3.30\tau_1 M_2 + 0.71\tau_1 M_3 + 29.35\epsilon_1 M_1 - 2.94\epsilon_1 M_2 + 0.74\epsilon_1 M_3$$

$$+ 3.00\epsilon_2 M_1 - 0.72\epsilon_2 M_2 + 0.17\epsilon_2 M_3 + 3.19\epsilon_3 M_1 - 0.54\epsilon_3 M_2$$

$$+ 7.08\tau_1\epsilon_1M_1 - 0.92\tau_1\epsilon_1M_2 + 0.20\tau_1\epsilon_1M_3 + 0.88\tau_1\epsilon_2M_1$$

$$- 0.17\tau_1\epsilon_2M_2$$

Transformations are defined as

$$\epsilon_1 = \frac{\mathrm{Log}_{10}E_3 - 5.05}{0.35} \tag{A3.10a}$$

$$\epsilon_2 = \epsilon_1^{\,2} - 4 \tag{A3.10b}$$

$$\epsilon_3 = \frac{\epsilon_1^{\,3} - 7\epsilon_1}{6} \tag{A3.10c}$$

$$M_1 = \frac{E_4 - 8100}{1500} \tag{A3.10d}$$

$$M_2 = \frac{3M_1^{\,2} - 35}{8} \tag{A3.10e}$$

$$M_3 = \frac{5M_1^{\,3} - 101M_1}{24} \tag{A3.10f}$$

τ_1 and τ_2 are different for the three equations.
For 0-6 inches,

$$\tau_1 = \frac{D_3 - 3}{3} \tag{A3.11a}$$

and

$$\tau_2 = 3\tau_1^{\,2} - 2 \tag{A3.11b}$$

For 6-12 inches,

$$\tau_1 = \frac{D_3 - 9}{3} \tag{A3.12a}$$

and

$$\tau_2 = 3\tau_1^2 - 2 \tag{A3.12b}$$

For 12-18 inches:

$$\tau_1 = \frac{D_3 - 15}{3} \tag{A3.13a}$$

and

$$\tau_2 = 3\tau_1^2 - 2 \tag{A3.13b}$$

where

D_3 = thickness of subbase, inches,

E_3 = modulus of subbase, psi,

E_4 = modulus of subgrade material, psi.

For each of these equations the values of correlation coefficient R^2 and the standard error of residuals are given below:

	Standard Error	R^2
Equation, 0-6 inches	3.752	.9998
Equation, 6-12 inches	3.797	.9999
Equation, 12-18 inches	7.178	.9998

EQUATIONS FOR DETERMINATION OF OVERLAY THICKNESSES

For pavements with remaining life the overlay thickness has been calculated using an "effective" thickness and the governing stress considered to be at the bottom of the existing pavement layer.

The relationship is

$$h_r = (h - h_e) \qquad \text{A3.14}$$

where

h_r = thickness of overlay in inches

h = "effective thickness" or thickness of a new pavement from portland cement concrete pavement design (taking remaining life into consideration) in inches

h_e = thickness of the existing pavement in inches.

This is the equation used for bonded overlays (Ref 27).

FATIGUE EQUATION

The concrete fatigue equation used in the simplified method is the same as Equation I - 3.3.

REMAINING LIFE EQUATION

The remaining life equation used in the simplified method is given in Part I (Eq I-3.5).

APPENDIX 4.

AN APPROXIMATE METHOD FOR OBTAINING A VALUE FOR S_{SG}, USING DEFLECTION MEASUREMENTS

APPENDIX 4. AN APPROXIMATE METHOD FOR OBTAINING A VALUE FOR S_{SG}, USING DEFLECTION MEASUREMENTS

In the Texas SDHPT rigid pavement overlay design method, the subgrade material is characterized using deflection measurements on the existing pavement. In the case where the deflection load is not equal to the design load, the stress dependency of the subgrade material is taken into account by using a laboratory determined relationship between resilient modulus and deviator stress. As pointed out in Chapter II-2, this relationship is a straight line when plotted on a log-log scale (see Eq I-2.2), and S_{SG} has been defined as the slope of this log-log relationship.

It has also been pointed out in Chapter II - 2 that the overlay thickness is not very sensitive to changes in S_{SG} if the existing pavement has no remaining life. In this event, it would be adequate to get a reasonably good estimate of the value of S_{SG}. Such an approximate method is described here.

DEFLECTION MEASUREMENT

Various devices for measuring deflections due to applied loads on pavement surfaces are being used. It would be beyond the scope of this report to evaluate all these in detail. A limited discussion on some ways to measure deflection follows:

Dynaflect

The Dynaflect is the most widely used deflection measuring device in the State of Texas. The Dynaflect was developed in Texas in 1964 and it measures pavement surface deflections under a cyclic vertical force of 1000 pounds (Ref 28). A set of five geophones measures deflections at a series of five points on the surface of the pavement. Figure A4.1 shows the loads and deflection measuring arrangement.

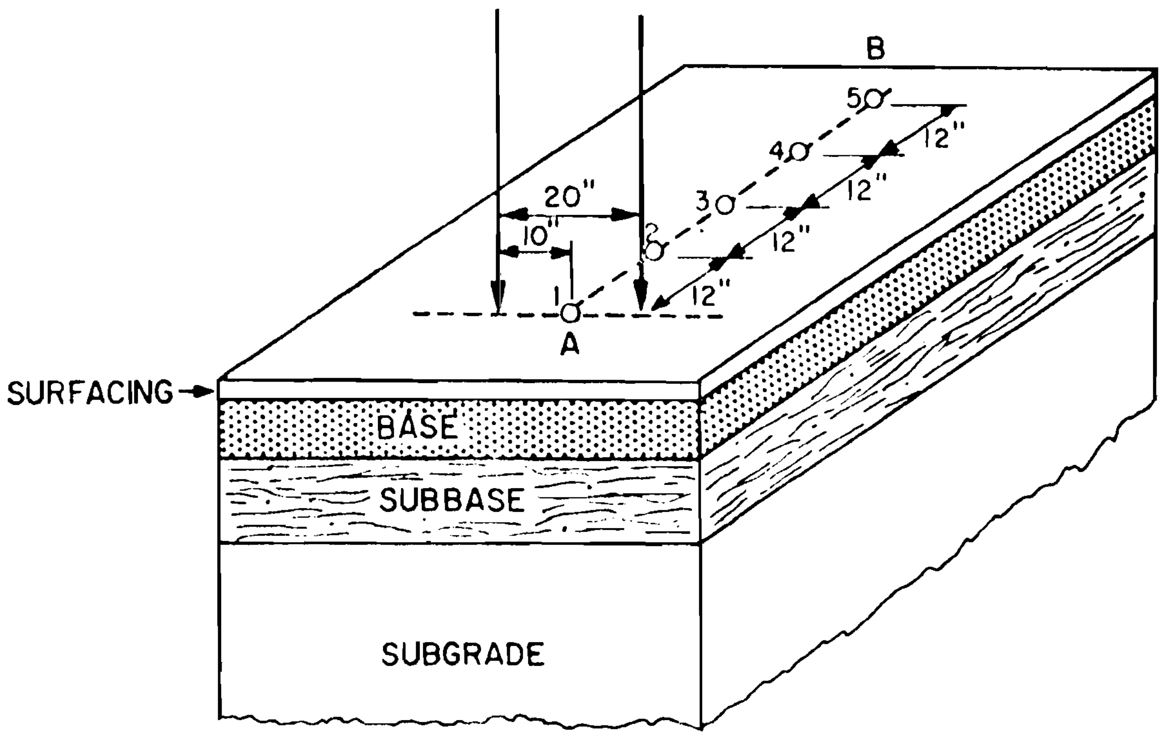


Fig A-4.1. Position of Dynaflect sensors during test. Vertical arrows represent load wheels. Points numbered 1 through 5 indicate location of sensors.

The Dynaflect has certain very desirable properties such as being simple and economical to operate, being reliable, and measuring deflections under a dynamic load. One of the drawbacks of this device is the relatively low stress level at which deflection measurements are made.

Benkelman Beam

The Benkelman beam is used to measure deflections, generally under a standard 18-kip (80 kN) single axle load, although any other load can be used in conjunction with this apparatus. This is a simple and widely used apparatus. The greatest disadvantage is the fact that the deflection is measured at such a low speed that it can be considered as a static load. Another problem with the Benkelman beam is that the beam supports can be resting in the deflection bowl if it is fairly large.

Some variations on the Benkelman beam, using essentially the same principles but more sophisticated and automated, are the Travelling Deflectometer and the LaCroix Deflectograph (Ref 29).

Falling Weight Deflectometer (Ref 30)

The Falling Weight Deflectometer is mounted on a small trailer and can be towed behind a car. It consists of a mass that slides down a shaft and falls on a system of springs on a circular plate. The maximum force it is able to apply to the pavement is 13.5 kips (60 kN). By varying the drop height of the mass, different dynamic loads can be applied to the surface. The deflection of the pavement is measured using velocity transducers. By using one transducer at the center of the loaded area and one some distance away, both maximum deflection and the shape of the deflection bowl can be determined. Some of the advantages of this device are that the pavement is subjected to a dynamic load and that the stress level is comparable to stress levels under the design load.

APPROXIMATE METHOD TO DETERMINE S_{SG}

In cases where the deflection load is not equal to the design load, as is the case with Dynaflect deflections, a relationship between resilient modulus and deviator stress for the subgrade material must be provided. An approximate method for determining a value for S_{SG} is suggested here, using deflection measurements at two stress levels, as follows:

(1) Determine the deflection profile and select design sections in the normal manner as prescribed for the Texas procedure (see Parts I and II). If these are Dynaflect measurements, deflections will be at much lower stress levels than under the design load and a value for S_{SG} must be determined.

(2) On each design section, perform at least 3 deflection measurements at a higher stress level, say using a Benkelman beam, at the location where the Dynaflect measurements were taken. For long design sections, it is recommended that 10 Benkelman Beam measurements be taken per mile.

(3) Determine the mean of the Dynaflect deflection measurements ($\bar{\Delta}_D$) as well as the mean of the Benkelman beam deflections ($\bar{\Delta}_B$) for each design section.

(4) Using the existing pavement structure and varying the resilient modulus of the subgrade, under Dynaflect load, determine the relationship between subgrade resilient modulus and surface deflection and subgrade resilient modulus and deviator stress at the top of the subgrade. A linear elastic layered program, such as ELSYM5, is suggested for use to develop these relationships. These relationships are conceptually plotted (Fig A.4-2) for Dynaflect loading conditions. $\bar{\Delta}_D$ is entered on the surface deflection axis and the resilient modulus (M_{RD}) and corresponding deviator stress (σ_{DD}) at the top of the subgrade are determined under Dynaflect loading.

(5) Repeat the procedure in (4) for Benkelman beam deflections and loading conditions, and obtain M_{RB} and σ_{DB}.

(6) Calculate S_{SG} as follows:

$$S_{SG} = \frac{\log M_{RB} - \log M_{RD}}{\log \sigma_{DB} - \log \sigma_{DD}} \qquad \text{A4.1}$$

where

S_{SG} = slope of the log resilient modulus versus log deviator stress relationship for subgrade material,

M_{RB} = subgrade resilient modulus resulting under Benkelman beam load (psi),

M_{RD} = subgrade resilient modulus resulting under Dynaflect load (psi),

σ_{DB} = deviator stress at top of subgrade under Benkelman beam load (psi),

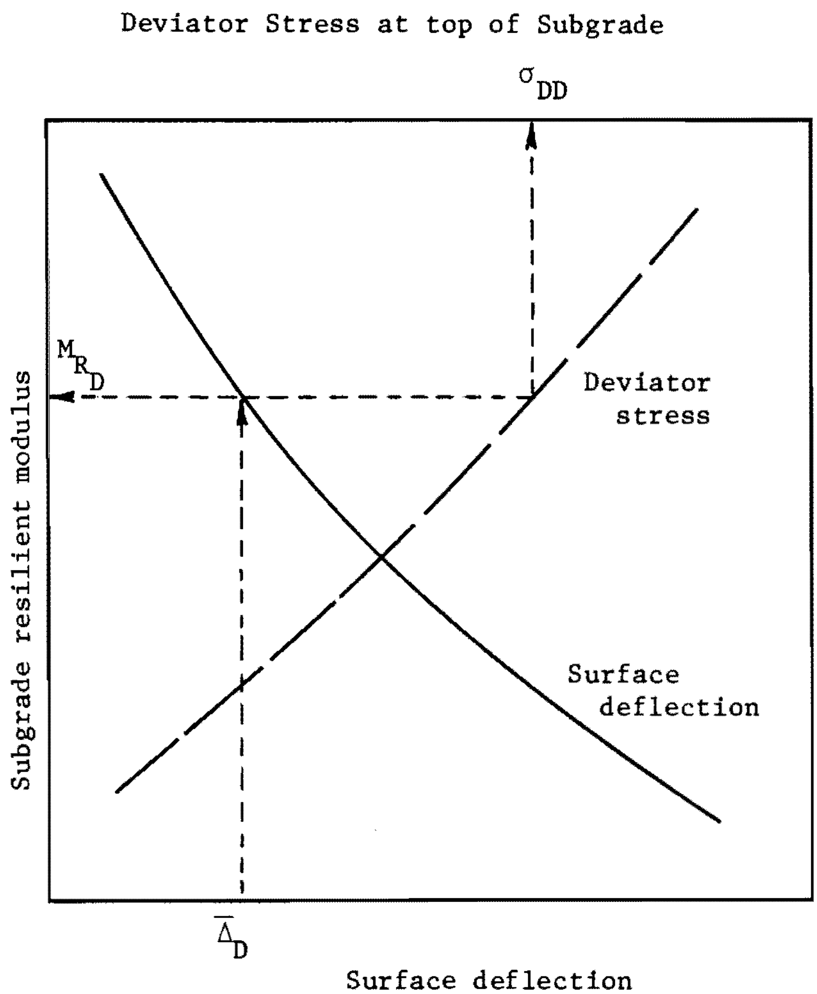


Fig A4.2. Conceptual determination of subgrade resilient modulus and deviator stress resulting under Dynaflect load at the top of the subgrade.

σ_{DD} = deviator stress at top of subgrade under Dynaflect load (psi).

This value of S_{SG} can be used directly in the RPOD2 program.

APPENDIX 5.

ESTIMATION OF MEAN VALUES AND STANDARD DEVIATIONS OF INPUT VARIABLES FOR THE RFLCR1 COMPUTER PROGRAM

APPENDIX 5. ESTIMATION OF MEAN VALUES AND STANDARD DEVIATIONS OF INPUT VARIABLES FOR THE RFLCR1 COMPUTER PROGRAM

In the single factorial sensitivity analysis reported in Chapter II-3, the medium, low, and high level values of the independent variables have been used. These values were determined using Eqs. II-3.3 and II-3.4, and, therefore, it was necessary to estimate mean values and standard deviations for all input variables. The sensitivity of the response is dependent on these values, and it is necessary to determine them as accurately as possible from field test conditions.

In this study, values have been determined using information from various existing reports. Some of the variables for RFLCR1 are the same as for RPOD1 and for these variables the values determined by Nayak et al. (Ref 7) for their sensitivity anlaysis as indicated on Table A5.1 have been used. In some instances no useful information was available and engineering judgement had to be used. In the case of the CRCP existing pavement, it was found that crack spacing, existing pavement thermal coefficient, and change in crack width with change in temperature have been so interrelated that it was decided to vary these three variables together. A large crack spacing together with a high thermal coefficient would result in very high concrete stresses before overlay, which would cause further cracking and a decrease in crack spacing. On the other hand, a larger crack movement than could have been caused by thermal movement would be an unrealistic situation one which RFLCR1 cannot handle. Factors like those mentioned above have been taken into consideration in determining the means and standard deviations for the different input variables.

ELASTIC MODULUS OF CONCRETE

Values for elastic modulus of concrete, as determined by Nayak et al., shown on Table A5.1, have been used in this sensitivity analysis:

TABLE A5.1. INPUTS FOR FACTORIAL DESIGN (Ref 7)

SL Number	Layer	Variable	Mean Value $\bar{X}_i$	Standard Deviation, σ_i (Total)	Lower Value of Variable, $\bar{X}_{iL}$ $(\bar{X}_i - 2\sigma)$	Higher Value of Variable, $\bar{X}_{iH}$ $(X_i + 2\sigma)$
1	Overlay	Modulus of elasticity (psi)	4.60×10^6	0.40×10^6	3.80×10^6	5.40×10^6
2		Poisson's ratio	0.20	0.05	0.10	0.30
3	Bond Breaker	Modulus of elasticity (psi)	10×10^4	2.5×10^4	5×10^4	15×10^4
4		Thickness (inch)	2.00	0.80	0.40	3.60
5		Poisson's ratio	0.40	0.05	0.30	0.50
6	Surface Course	Modulus of elasticity (psi)	4.60×10^6	0.40×10^6	3.80×10^6	5.40×10^6
7		Thickness (inch)	8.00	0.50	7.00	9.00
8		Poisson's ratio	0.20	0.05	0.10	0.30
9	Base Course	Modulus of elasticity (psi)	5×10^5	1.00×10^5	3.00×10^5	7.00×10^5
10		Thickness (inch)	8.00	0.80	6.40	9.60
11		Poisson's ratio	0.20	0.05	0.10	0.30
12	Subgrade	Resilient Moduli (psi) Deviator Stress				
		2 psi	19×10^3	7.50×10^3	4.00×10^3	34.00×10^3
		5 psi	16×10^3	7.00×10^3	2.00×10^3	30.00×10^3
		8 psi	12×10^3	5.50×10^3	1.00×10^3	23.00×10^3
13		Poisson's ratio	0.40	0.10	0.20	0.60
14		Deflection (inch)				
		CRCP	0.0090	0.0030	0.0030	0.0150
		JCP-Class 1 and 2	0.0100	0.0036	0.0028	0.0172
		JCP-Class 3 and 4	0.0140	0.0044	0.0052	0.0228
15		Ratio of corner to interior deflection (JCP)	2.800	0.20	2.40	3.20
16		Flexural strength of concrete (psi)	600	50.0	500.00	700.00
17		Traffic prior to overlay	4×10^6	0.5×10^6	3.0×10^6	5.0×10^6

1 psi = 6.8948 kPa

1 inch = 25.4 mm

Mean elastic modulus = 4.6×10^6 psi (2.75×10^3 MPa)

Total standard deviation = 0.4×10^6 psi (2.75×10^3 MPa)

THERMAL COEFFICIENT OF CONCRETE

Reference 6 suggests values for concrete thermal coefficient to be used between 3.8×10^{-6} and 6.6×10^{-6} in./in./°F (6.8×10^{-6} and 1.9×10^{-5} mm/mm/°C). Kerbs and Walker (Ref 31) suggests a range of 3.6 to 6.8×10^{-6} in./in./°F (6.48×10^{-6} to 1.2×10^{-5} mm/mm/°C).

With this information it was decided to use the following values:

Mean thermal coefficient = 5.2×10^{-6} in./in./°F (9.4×10^{-6} mm/mm/°C)

Total standard deviation = 1.4×10^{-6} in./in./°F (2.5×10^{-6} mm/mm/°C)

JOINT SPACING FOR JCP

Using engineering judgement, the following values for joint spacing have been selected:

Mean joint spacing = 13.5 feet (4.1 m)

Total standard deviation = 1.5 feet (457 mm)

CRACK SPACING FOR CRCP

Table A5.2 shows information obtained from Ref 32.

A mean crack spacing of 6.075 feet (1.85 m) and a standard deviation of 2.5179 feet (767 mm) have been determined from this information. As previously mentioned, however, some amount of engineering judgement had to be applied in determining values for crack spacing, horizontal joint

TABLE A5.2. MEAN VALUES OF CRACK SPACING FOR DIFFERENT TEST SECTIONS IN TEXAS (Ref 32)

Section	Crack Spacing (ft.)	Section	Crack Spacing (ft.)
1	8.8	15	5.7
2	9.2	16	7.8
3	8.5	17	6.6
4	5.8	22	5.3
5	7.7	24	3.3
6	11.1	25	3.1
7	8.6	26	3.5
8	9.3	27	1.9
9	7.4	28	2.7
10	6.3	29	4.3
11	7.6	30	4.6
12	6.6	31	1.9
13	8.8	32	2.7
14	6.1	33	4.9

(1 foot = .3048 m)

movement and thermal coefficient, and finally the following values have been selected:

Mean crack spacing = 6 feet (1.828 m)

Standard deviation = 2 feet (610 mm)

CHANGE IN JOINT WIDTH FOR TEMPERATURE CHANGE FROM 80°F (26.7°C) TO 70°F (21.1°C) FOR JCP

No field measurements were available for this variable, and engineering judgment was used. Using the following equation (Ref 6),

$$Y_c(X) = \alpha_c(\Delta T_c)\ (X - \beta X^{\beta}) \qquad \text{(A5.1)}$$

where

$Y_c(X)$ = actual concrete movement at a distance X from the slab's center due to a temperature change of ΔT_c, inches;

X = distance from slab's center to point of observation, inches; and

β = restraint coefficinet.

The β term is a restraint coefficient that represents any force which restricts free concrete movement. By selecting realistic values for the variables in this equation, the following values for change in joint width for temperature change from 80°F (26.7°C) to 70°F (21.1°C) have been selected:

Mean value = 3.5×10^{-3} inch (.089-mm)

Standard deviation = 1.5×10^{-3} inch (.038-mm)

Care has been taken to avoid a situation where this movement would be greater than that which could be caused by the thermal expansion or contraction.

CHANGE IN CRACK WIDTH FOR TEMPERATURE CHANGE FROM 80°F (26.7°C) TO 70°F (21.1°C) FOR CRCP

For this variable, it was found that it is so interrelated with crack spacing and thermal coefficient that unrealistic combinations of these three variables would easily predict a too high tensile stress in the concrete (which would result in further cracking and reduced crack spacing), or more movement would be predicted in the crack than the thermal volume change could cause -- a situation the model cannot handle.

Realistic values for this variable have been determined through trial and error and engineering judgement, as follows:

$$\text{Mean value} = 3.2 \times 10^{-3} \text{ inch } (.081\text{-mm})$$

$$\text{Standard deviation} = 1.95 \times 10^{-3} \text{ inch } (.050\text{-mm})$$

CONCRETE THICKNESS

Values used by Nayak et al. (Ref 7) have been used in this analysis (see Table A5.1).

Mean concrete thickness = 8 inches (203.2 mm)

Standard deviation = 0.5 inch (12.7-mm)

CONCRETE DENSITY

Table A5.3 shows information on concrete densities (Ref 12).

TABLE A5.3. CONCRETE DENSITIES FOR VARIOUS PROJECTS (Ref 12)

Project	Number of Tests	Density, pcf		
		Mean	CV%	Variance
17-B	141	142.4	2.0	8.11
	29	144.3	0.8	1.33
	122	141.3	1.4	3.91
17-M	21	141.2	1.2	2.87
	24	141.3	1.1	2.42
	63	133.1	1.9	6.40
19-B	25	134.5	1.6	4.63
	17	135.0	1.6	4.67

1 pcf = 16.01 kgm^{-3}

$$\text{Weighted mean} = 140.05 \text{ pcf } (2242 \text{ kgm}^{-3})$$

$$\text{Within-project variance} = \sigma_W^2 = 2.304 \text{ pcf } (37 \text{ kgm}^{-3})$$

$$\text{Between-project variance} = \sigma_B^2 = 13.162 \text{ pcf } (211 \text{ kgm}^{-3})$$

$$\text{Total variance} = \sigma_T^2 = \sigma_W^2 + \sigma_B^2 \qquad \text{(A5.1)}$$

Therefore,

$$\sigma_T = 3.933 \text{ pcf } (63 \text{ kgm}^{-3})$$

Values selected for this variable are as follows:

$$\text{Mean concrete density} = 140 \text{ pcf } (2241 \text{ kgm}^{-3})$$

$$\text{Standard deviation} = 4 \text{ pcf } (64 \text{ kgm}^{-3})$$

MOVEMENT AT SLIDING

Since very little information on friction curves exists, the values suggested for use by Treybig et al. (Ref 6) have been used in determining values for this variable:

Mean value for movement at sliding = 0.135 inch (3.4-mm)

Standard deviation = 0.115 inch (2.9-mm)

MINIMUM TEMPERATURE OBSERVED

Weather records at the Department of Meterology at The University of Texas have been studied for the purpose of determining a value for the minimum temperature observed. Table 5.4 indicates minimum temperatures

TABLE A5.4. MINIMUM TEMPERATURES OBSERVED AT LOCATIONS IN TEXAS

	Minimum Temperature at (°F)	
Year	Wichita Falls	Amarillo
1960	17	-4
1964	7	5
1968	8	3
1970	12	1
1972	7	1
1976	8	1

°C = (°F - 32)5/9

selected at random for six different years at Wichita Falls and Amarillo, Texas. Using the data in Table A5.4 the following values have been selected for the minimum temperature observed.

Mean minimum temperture = 5.5°F (-14.7°C)

Standard deviation = 5.5°F (3°C)

MEAN CRACK WIDTH FOR CRCP

Information concerning joint width in CRCP pavements has been listed in Table A5.5 (Ref 32).

Using the information in Table A5.5, the following values have been selected for use in the sensitivity analysis:

Mean value for mean crack width = .018 inch (.45 mm)

Standard deviation = .01 inch (.25 mm)

MEAN JOINT WIDTH FOR JCP

Information on joint width in JCP has not been available, but with engineering judgement, the following values have been selected:

Mean value for mean joint width = .04 inch (1mm)

Standard deviation = .01 inch (.25 mm)

LOAD TRANSFER

Using engineering judgement, values for load transfer have been selected as follows:

Mean value for load transfer = 80 percent

Standard deviation = 15 percent

TABLE A5.5. VALUES OF MEAN CRACK WIDTH IN CRCP PAVEMENTS ON VARIOUS TEST SECTIONS (Ref 32)

Sections	Crack Width (inches)	Section	Crack Width (inches)
1	.03	26	.004
6	.028	27	.004
10	.031	28	.018
13	.029	29	.004
14	.024	30	.026
17	.029	31	.016
24	.006	32	.017
25	.004	33	.019

1 inch = 25.4 mm

DESIGN TEMPERATURE CHANGE

In order to determine the design temperatures for the different layers, it is first necessary to determine the minimum surface or air temperature expected to occur in the design period from historical records as a 5-day mean air temperature (Ref 6). In order to estimate reasonable values for design temperatures, weather information at the Department of Meterology at The University of Texas has been studied. Table A5.6 gives daily mean air temperatures for the months of January 1970 and January 1976 at Wichita Falls and Amarillo, Texas.

Five-day mean air temperatures have been determined from Table A5.6, and, for each month at each location, the five lowest 5-day mean temperatures have been considered to determine a mean and standard deviation for minimum air temperature:

Mean for minimum air temperature = 26°F (-3°C)

Standard deviation = 5°F (2.8°C)

Using the information that the minimum 5-day air temperature could range from 21°F to 31°F (-6°C to -0.5°C) and that minimum surface temperature could be between 0°F and 11°F (-18°C and -11.7°C) and assuming that the maximum temperature of the slab after placement of the overlay will be 110°F, the following values for design temperature change have been determined.

For overlay,

Mean design temperature change = 105°F (58°C)

Standard deviation = 5°F (2.8°C)

For existing pavement,

Mean design temperature change = 94°F (52°C)

Standard deviation = 4°F (2.2°C)

TABLE A5.6. DAILY MEAN AIR TEMPERATURE IN °F $\left[°C = .556(°F - 32)\right]$

Day	Wichita Falls January 1970	Wichita Falls January 1976	Amarillo January 1970	Amarillo January 1976
1	28	48	21	30
2	33	30	22	21
3	37	29	20	21
4	35	28	24	27
5	30	38	18	35
6	23	44	12	32
7	28	19	18	10
8	18	18	14	25
9	23	35	21	42
10	40	52	43	48
11	39	42	42	36
12	30	49	32	47
13	35	49	34	40
14	38	40	40	31
15	39	48	44	47
16	42	46	34	43
17	39	50	28	52
18	16	56	20	51
19	25	48	35	36
20	24	39	34	35
21	24	45	22	43
22	34	51	45	44
23	42	60	45	52
24	51	51	57	41
25	56	39	52	30
26	52	34	48	24
27	55	37	52	34
28	64	47	47	45
29	40	50	31	47
30	42	54	33	46
31	42	49	36	39
Average	36.1	42.9	33.2	36.9

OVERLAY CREEP MODULUS

The creep modulus used in this procedure is to be determined at a loading time of between 20,000 seconds and 12 hours at the minimum temperature expected to occur in the asphalt concrete layer (as mentioned above). It is suggested this be done using Figs III - 4.7 to III - 4.9. With this information and some engineering judgement, values for creep modulus have been selected:

Mean value for creep modulus = 320,000 psi (2206 MPa)

Standard deviation = 180,000 psi (1241 MPa)

OVERLAY THICKNESS

A mean value for overlay thickness of 8 inches (203.2-mm) has been selected, and it has been decided to use the same standard deviation as for the existing pavement (0.5 inch or 12.7-mm).

DENSITY OF ASPHALT CONCRETE

Density information on asphalt concrete is given in Table A5.7 (Ref 33).

With the data in Table A5.7 an overall mean of 136.01 pcf (2177 kgm^{-3}), a within-project variance of 9.11, a between-project variance of 46.25, and a total standard deviation of 7.44 pcf (119 kgm^{-3}) have been determined.

For the purpose of the sensitivity analysis, the following values have been selected:

Mean asphalt concrete density = 136 pcf (2177 kgm^{-3})

Standard deviation = 7.5 pcf (120 kgm^{-3})

POISSON'S RATIO

For Poisson's ratio the value used as a default in RPOD1 (Ref 6) has been considered as a reasonable mean value, and the standard deviation

TABLE A5.7. ASPHALT CONCRETE DENSITY INFORMATION OBTAINED FROM SEVERAL PROJECTS (Ref 33)

Project	Number of Tests	Density pcf	
		Mean	CV%
2	23	129.1	1.6
5	20	126.2	1.8
8B	20	135.7	2.4
8C	14	137.6	3.0
17B(1)	15	137.4	1.7
17B(2)	15	136.5	1.7
25-97(1)	15	130.4	1.4
25-97(2)	15	135.5	1.6
25-97(3)	15	150.2	1.4
25-97(5)	15	141.7	3.3
25-100(1,2)	16	133.8	2.0
25-100(3)	12	149.9	1.9
25-100(5)	12	133.8	4.6

1 pcf = 16.01 kgm^{-3}

for Poisson's ratio used by Nayak et al. (Ref 7) for asphalt concrete bondbreaker has been accepted resulting in the selection of the following values:

Mean Poisson's ratio = 0.3

Standard deviation = 0.05

DYNAMIC MODULUS

Ref 6 suggests that dynamic modulus for the overlay should be determined in the same way as for RPOD1. In studying the illustrative examples, it will be noted that the dynamic modulus is determined at the same low temperature as the creep modulus. Since no data have been available on dynamic modulus at those low temperatures, the approach followed to determine reasonable values for dynamic modulus is that described for creep modulus, with the exception that loading times considered were 0.4 seconds.

The following values have been selected:

Mean dynamic modulus = 6.75×10^6 psi (46.5×10^3 MPa)

Standard deviation = 2.25×10^6 psi (15.5×10^3 MPa)

OVERLAY TO EXISTING SURFACE BONDING STRESS

Since no field data were available for this value, engineering judgement has been used, together with guidelines given in Ref 6 to select the following values:

Mean value for bonding stress = 850 psi (5.85 MPa)

Standard deviation = 350 psi (2.41 MPa)

WIDTH OF BONDBREAKER

Arbitrary values have been selected for this variable.

Mean width of bondbreaker = 1 foot (304.8 mm)

Standard deviation = 0.5 feet (152.4 mm)

ELASTIC MODULUS OF STEEL

Since the most general value for steel modulus of elasticity suggested in the literature seems to be 29×10^6 psi (199.9×10^3 MPa) (Refs 6 and 34) it has been decided to use this value and not to vary this variable.

THERMAL COEFFICIENT OF STEEL

Reference 6 suggests a range for steel thermal coefficient of 5.0×10^{-6} to 6.5×10^{-6} inch/inch/°F (9×10^{-6} to 1.17×10^{-5} mm/mm/°C). Merrit (Ref 35) suggests a value of 6.5×10^{-6} inch/inch/°F (1.17×10^{-5} mm/mm/°C). Bearing this information in mind, a mean thermal coefficient of 5.75×10^{-6} inch/inch/°F (1.0×10^{-5} mm/mm/°C) and a standard deviation of 0.75×10^{-6} inch/inch/°F (1.35×10^{-6} mm/mm/°C) have been selected for use.

AREA OF STEEL IN CRCP

Table A5.8 gives steel percentages in CRCP on different sections of highway in Texas as reported by McCullough et al. (Ref 32).

These data give a mean percentage steel of 0.527 percent and a standard deviation of .043 percent. Using this information together with the variation in slab thickness and with some judgement, the following values have been used for the sensitivity analysis:

Mean area of steel per foot width = 0.508 in^2/ft (1075 mm^2/m)

Standard deviation = 0.073 in^2/ft (155 mm^2/m)

TABLE A5.8. STEEL PERCENTAGE IN CRCP FOR VARIOUS HIGHWAY SECTIONS IN TEXAS (Ref 32)

Section	Steel (%)	Section	Steel (%)
1	0.5	15	0.5
2	0.5	16	0.5
3	0.5	17	0.5
4	0.5	22	0.5
5	0.5	24	0.6
6	0.5	25	0.6
7	0.5	26	0.6
8	0.5	27	0.6
9	0.5	28	0.5
10	0.5	29	0.596
11	0.5	30	0.596
12	0.5	31	0.596
13	0.5	32	0.553
14	0.5	33	0.5

PERIMETER OF STEEL

With the abovementioned information and taking in consideration that reinforcing bar size in the longitudinal direction might be between 0.5 inch (127 mm) and 0.75 inch (19 mm) in diameter, values for the perimeter of steel have been selected.

Mean value for perimeter of steel = 3.49 in/ft (291 mm/m)

Standard deviation = 1.17 in/ft (98 mm/m)

STEEL TO CONCRETE BONDING STRESS

With guidelines from Ref 6 and engineering judgement, a mean value for steel to concrete bonding stress of 260 psi (1.79 MPa) and a standard deviation of 90 psi (0.62 MPa) have been selected.

DESIGN LOAD WEIGHT

Arbitrary values for this variable have been selected.

Mean design load weight = 18,000 pounds (80 kN)

Standard deviation = 2,000 pounds (8.9 kN)

WIDTH OF DESIGN LOAD

Here, also, arbitrary values have been selected:

Mean width of design load = 24 inches (609.6 mm)

Standard deviation = 4 inches (101.6 mm)

The values selected have been used in the sensitivity analysis described in Chapter II-3. Table II-3.1 is a summary of the values used for input variables for the JC existing pavement, as well as for the CRC existing pavement.

APPENDIX 6.

A TENTATIVE METHOD FOR DETERMINATION OF A MAXIMUM ALLOWABLE VALUE FOR REPEATED SHEAR STRAIN DUE TO TRAFFIC LOADS

APPENDIX 6. A TENTATIVE METHOD FOR DETERMINATION OF A MAXIMUM ALLOWABLE VALUE FOR REPEATED SHEAR STRAIN DUE TO TRAFFIC LOADS

Because of differential vertical deflections at joints or cracks in existing pavements, overlays are subjected to repeated shear strains. The RFLCR1 program calculates this shear strain. The object of this section is to relate this shear strain to fatigue life. With this relationship it will then be possible to determine a maximum allowable shear strain value for a specific design traffic.

RELATIONSHIP BETWEEN SHEAR STRAIN AND SHEAR STRESS

Hudson and Kennedy (Ref 35) and Kennedy (Ref 36) indicate that for an element at the center of a specimen during the indirect tensile test, the relationship between the vertical compression stress and horizontal tensile stress is as follows:

$$\sigma_c = 3\sigma_t \qquad \text{A6.1}$$

where

σ_c = vertical compressive stress, psi

σ_t = horizontal tensile stress, psi.

This relationship is indicated on Fig A.6-1. These are principal stresses and from a Mohr circle plot, as in Fig A.6-2, an equation to relate shear stress with tensile stress can be obtained:

$$\tau = 2\sigma_t \qquad \text{A6.2}$$

where

τ = maximum shear stress, psi,

and

$$G = \frac{\tau}{\gamma} \qquad \text{A6.3}$$

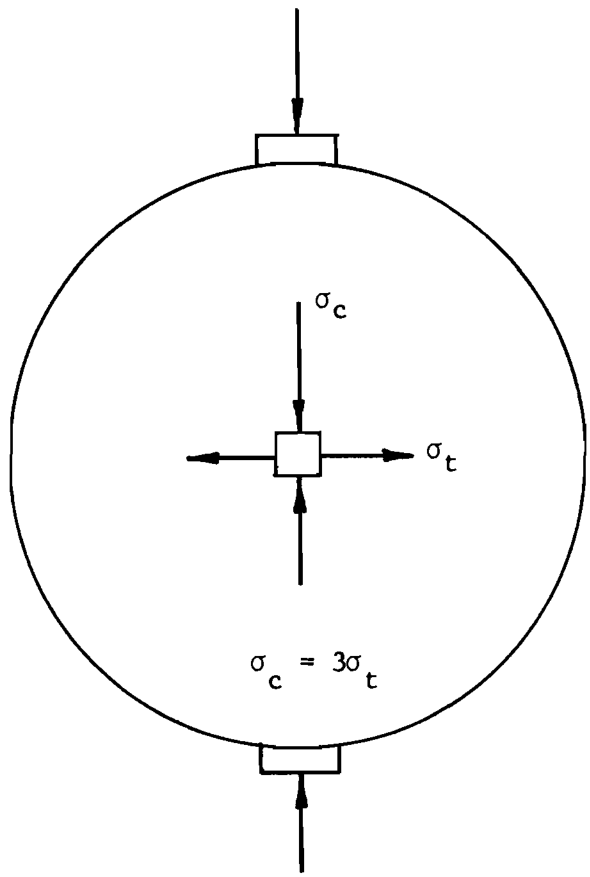


Fig A6.1. Element showing biaxial state of stress for the indirect tensile test.

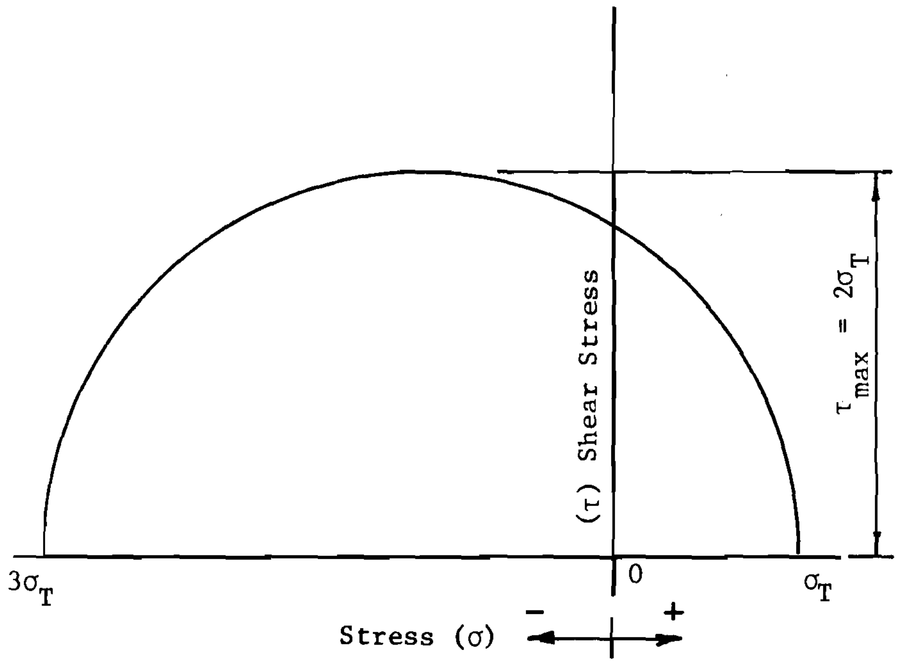


Fig A6.2. Mohr diagram for stresses on an element at the center of a specimen in indirect tensile test.

where

G = shear modulus, psi,

γ = maximum shear strain, in.

Also

$$G = \frac{E}{2(1 + \mu)} \qquad A6.4$$

where

E = elastic modulus,

μ = Poisson's ratio.

From Eqs A6.2 and A6.3 and A6.4,

$$\gamma = 4(1 + \mu)\varepsilon \qquad A6.5$$

RELATIONSHIP BETWEEN SHEAR STRAIN AND STRAIN REPETITIONS TO FAILURE

The general form of the fatigue equation is as follows:

$$N = A\left(\frac{1}{\varepsilon}\right)^{B} \qquad A6.6$$

where

N = number of strain repetitions to failure,

ε = horizontal tensile strain (or critical tensile strain),

A, B = constants.

Therefore,

$$\varepsilon_{max} = \left(\frac{N}{A}\right)^{\frac{1}{-B}} \qquad A6.6$$

From A6.5 and A6.6,

$$\gamma_{allowable} = 4(1 + \mu)\left[\frac{N}{A}\right]^{\frac{1}{-B}} \qquad A6.7$$

For the Texas method the relationship could be developed as follows:

From equation A2.4,

$$A = 9.7255 \times 10^{-15}$$

and

$$B = 5.163$$

For asphaltic concrete assume Poisson's ratio = 0.3. Then

$$\gamma_{allowable} = 5.2 \left(\frac{N}{9.7255 \times 10^{-15}} \right)^{-0.1937} . \qquad A6.8$$

Figure III - 4.10 is a plot of allowable shear strain versus repetitions to failure.

It is suggested that the vertical shear strain determined by RFLCR1 be limited to a value equal to N less strain $\gamma_{allowable}$ determined by equation A6.8 or from Fig III - 4.10.

This method is suggested for use until a better method for determination of the fatigue properties of asphaltic concrete due to repeated shear strain has been developed.

APPENDIX 7

PLOT 2 INPUT GUIDE

APPENDIX 7. PLOT2 INPUT GUIDE

Card Type 1: Problem Title Card

1.1 Title for this problem (any combination of letters and/or numbers)

11	80

Card Type 2: Plot Label Card

2.1 General label for deflection axis plot (any combination of letters and/or numbers)
Default value: "DYNAFLECT READING, SENSOR 1"

1	30

2.2 Label for list of deflection locations-first row (any combinations of letter and/or numbers)
Default value: "STATIONING"

31	32	33	34	35	36	37	38	39	40

2.3 Label for list of deflection locations-second row (any combination of letters and/or numbers)
Default value: "(FEET)"

41	42	43	44	45	46	47	48	49	50

2.4 Label for list of deflection values-first row (any combination of letters and/or numbers)
Default value: "DEFLECTION"

51	52	53	54	55	56	57	58	59	60

2.5 Label for deflection values-second row (any combination of letters and/or numbers)
Default value: "(MILS)"

61	62	63	64	65	66	67	68	69	70

Card Type 3: Deflection Format Card

3.1 Format for reading in deflection data

1	39

Default value: "(2A4, A2, F10.0, F5.0)"
Where the first 10 columns (2A4, A2) are for reading in deflection location or station, the next 10 columns (F10.0) are for reading in the actual deflection and the last 5 columns, (F5.0) are for reading in the deflection multiplier. This card is not required if the default format is acceptable to the user.

Card Type 4: Deflection Data Card

This card is used to read in deflection data. A maximum of 300 deflections can be read and plotted. The format of input can be specified through card number 3; however, the order in which deflection and multiplier are to be read in cannot be changed.

If the user does not specify an input format, data will be read as follows.

4.1 Location of deflection measurement (any combination of letters and/or numbers)

1	2	3	4	5	6	7	8	9	10

4.2 Deflection value

•									
11	12	13	14	15	16	17	18	19	20

4.3 Deflection multiplication factor. This value is useful if Dynaflect readings are to be used. Default value: 1.0 (Position of decimal point can be changed; however, a decimal point is required)

21	22	23	24	25

Card Type 5: Termination Card

5.1 Termination of problem."FINISH" must appear in the first six columns of this card.

F	I	N	I	S	H
1	2	3	4	5	6

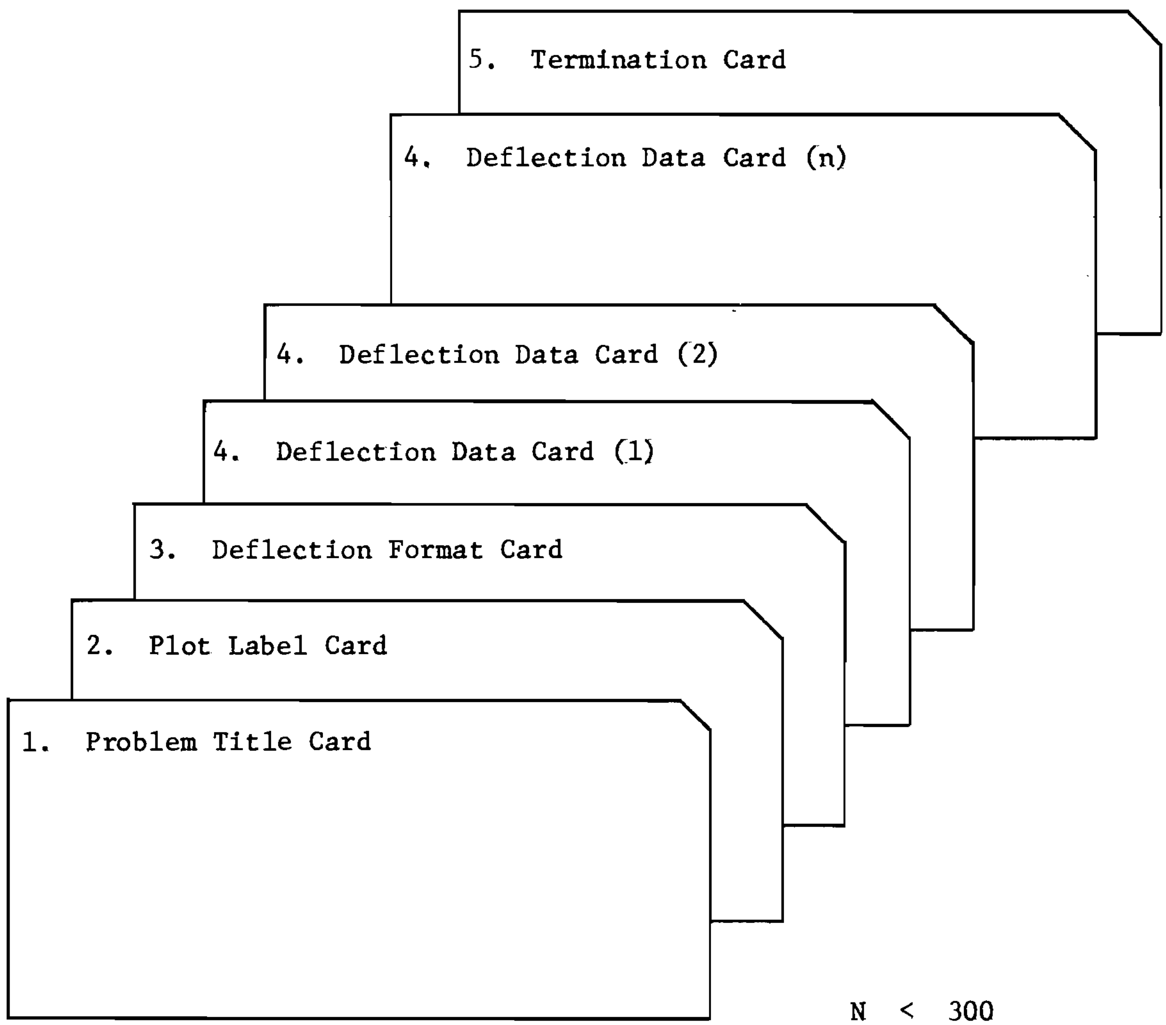


Fig A7.1. Assembly order of PLOT2 data.

APPENDIX 8.

TVAL2 INPUT GUIDE

APPENDIX 8. TVAL2 INPUT GUIDE

Card Type 1: Problem Descritpion Card

1.1 Total number of deflections to be read (right justify)

1	2	3	4	5

1.2 Number of sections into which deflections are divided (right justify)

6	7	8	9	10

1.3 Problem title: it is useful to specify the deflection units here also (any combination of letters and or numbers)

11		80

Card Type 2: Section Specification Card

2.1 Number of deflections in section 1 (right justify)

1	2	3	4	5

2.2 Number of deflections in section 2 (right justify)

6	7	8	9	10

.
.
.

2.i Number of deflections in section i (right justify)

5i-4	5i-3	5i-2	5i-1	5i

.
.
.

2.16 Number of deflections in section 16 (right justify)

76	77	78	79	80

Card Type 3: Confidence Level and Deflection Format Card

3.1 Confidence level for student's t analysis (right justify)
Default value: 95 percent
Legal values: 90,95 and 99 percent

1	2	3	4	5

3.2 Confidence level for computing interior design deflection
Legal values: 99, 97.5, 95, 90, 75 and 50 percent
Leave blank if this option is not desired

			•	
6	7	8	9	10

3.3 Format for reading deflection data (any combination of letters and/or numbers)
Note that this format must include open and close parenthesis.
Default is (10X, F10.0, F5.0) where 10X designates the first 10 columns of the card to be shipped, the F10.0 field is used for reading the deflection and the F5.0 field is for reading in the multiplier. If no format is specified, the default will be used.

11		50

Card Type 4: Deflection Data Card

Through the use of 3.3 above, it is possible for the user to specify the format in which he wishes to read in his data. The order in which the deflection and its multiples are read in cannot be changed. If the default value for 3.3 is used, data will be read in as follows.

4.1 Deflection value. These values are the same values as those read in for the PLOT2 program. For this purpose the position of deflection is not required. It should, however, be read in in the correct order to ensure that the appropriate deflections are used for each specific section considered.

•									
11	12	13	14	15	16	17	18	19	20

4.2 Deflection multiplication factor
Default value: 1.0
A maximum of 1440 deflection values (16 sections at 90 values per section) is allowed

	•			
21	22	23	24	25

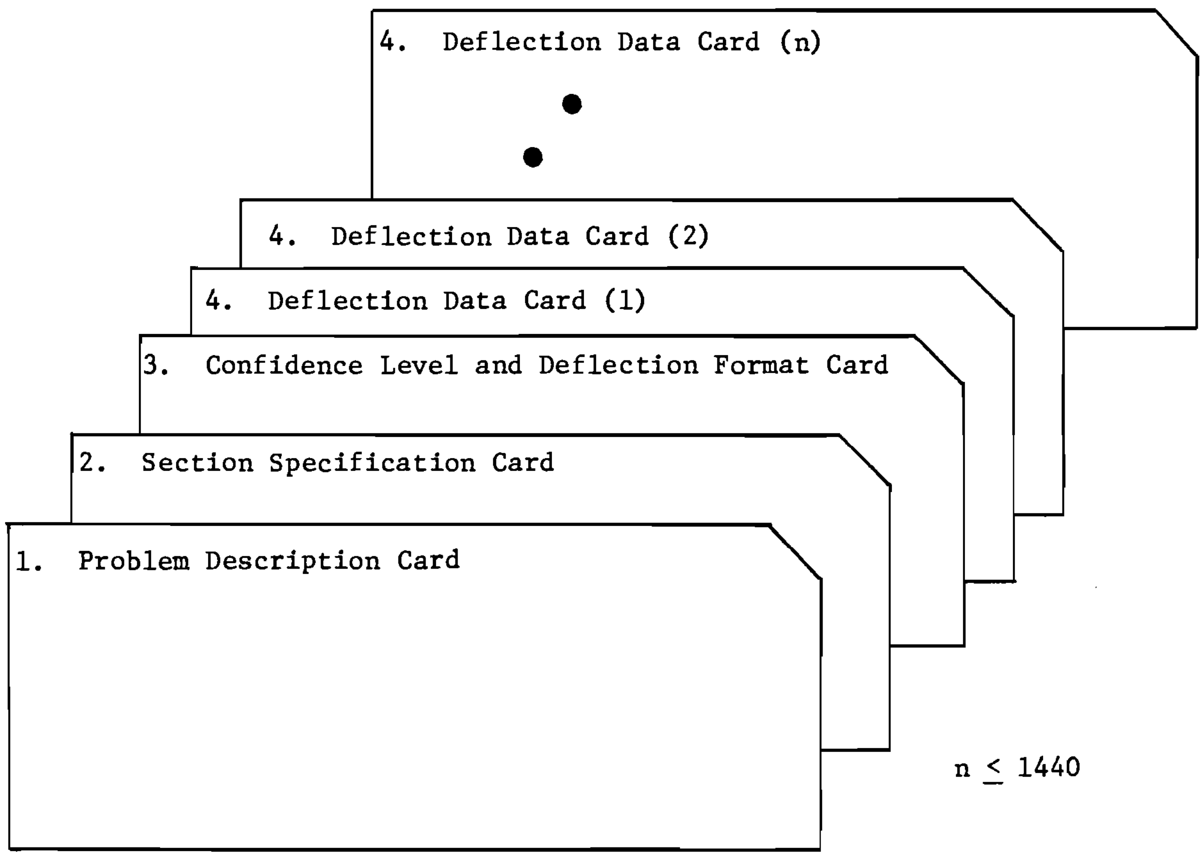


Fig A-8.1. Assembly of TVAL2 data.

APPENDIX 9

RPOD2 INPUT GUIDE

APPENDIX 9. RPOD2 INPUT GUIDE

Card Type 1: New Problem Card

1.1 Directive

P	R	O	B	L	E	M	
1	2	3	4	5	6	7	8

1.2 Problem Number

9	10

1.3 Title Card Switch
If this value is greater than zero, the entire 80 columns of the following card will be read as a title card.

									•
11	12	13	14	15	16	17	18	19	20

Card Type 2: Title Card

1	80

(Any combination of letters and/or numbers).
Note: present this card only if 1.3 is greater than zero.

Card Type 3: Existing Pavement Card

3.1 Directive

P	A	V	E	M	E	N	T
1	2	3	4	5	6	7	8

3.2 Number of layers in existing pavement structure.
This must include the subgrade.
At least one and not more than four layers may be specified.
If a bondbreaker is used only three layers may be specified here. This value also designates how many of Card Type 4 (Layer Cards) are to be expected.

9	10

3.3 Number of 18-kip equivalent single axle wheel loads applied to date.
This value must be non-zero; therefore, default value = 1.

									•
11	12	13	14	15	16	17	18	19	20

3.4 Existing pavement concrete flexural strength, psi
Default value = 690 psi

				•
21	22	23	24	25

3.5 Existing pavement condition (any combination of letters and/ or numbers).

31	32	33	34	35	36	37	38

Blank - No cracking or voids present
"VOID" - Voids present, but no cracking
"TYPE 1,2" - Type 1 or 2 cracking present
"VOID 1,2" - Type 1 or 2 cracking with voids present
"TYPE 3,4" - Type 3 or 4 cracking present
"MECH BKN" - Pavement will be mechanically broken prior to overlay.

Card Type 4: Layer Card

This card defines the properties of the existing pavement and is required for each layer, down to and including the subgrade. The layers are numbered from the top down and a maximum of four layers is permitted unless a bondbreaker is specified, in which case only three layers are permitted. If the thickness of the subgrade is zero, the program will assume it semi-infinite. If the thickness of the subgrade is not zero, the program will assume the presence of bedrock beneath the subgrade layer when performing deflection calculations. The variable definitions are;

4.1 Directive

L	A	Y	E	R			
1	2	3	4	5	6	7	8

4.2 Layer Number (right justify)

9	10

4.3 Elastic modulus of layer in 4.2, psi. Note: If card type 7 is provided, the subgrade requires only an approximate value to start iteration.

									•
11	12	13	14	15	16	17	18	19	20

4.4 Thickness of layer in 4.2, inches

			•	
21	22	23	24	25

4.5 Material type of layer in 4.2 (any combination of letters and/ or numbers)

31	32	33	34

"AC" - asphaltic concrete,
"CRCP" - continuously reinforced concrete pavement,
"GRAN" - granular base material,
"JCP" - jointed concrete pavement,
"STAB" - stabilized base material,
"SUBG" - subgrade layer.
(top layer must be either JCP or CRCP)

4.6 Rigid base interface type
(required if rigid base is required)
"FF" - full friction interface
"NF" - no friction interface
(no default value)
Note: A fixed value for Poisson's ratio for a specific material type is being used. For more information on the values being used as well as how to use other values, see the supplement to this guide.

35	36	37	38

Card Type 5: Lab Data Designation Card

This card is required if the load under which the deflection measurements are taken differs significantly from the 18 kip equivalent axle load. Laboratory tests must be made to determine elastic modulus as a function of deviator stress for the subgrade.

As an alternative this function can be expressed as the slope of the log resilient modulus versus log deviator stress relationship, which might be determined by approximate ways discussed in Appendix 4.

5.1 Directive

L	A	B		D	A	T	A
1	2	3	4	5	6	7	8

5.2 Number of pairs of lab data points
(right justify)
Lab data required are elastic modulus versus corresponding deviator stress. A minimum of two points and a maximum of ten may be specified.
If this value is provided, card Type 6 must follow this card. If 1 is entered in this field, 5.3 must be provided.

9	10

5.3 Slope of the log resilient modulus versus log deviator stress line.
This program can handle only negative values for this slope. Zero slopes must be input as a slight negative value, say -0.0001. In this case, the number of pairs of lab data points (5.2) must be 1.

					•				
11	12	13	14	15	16	17	18	19	20

Card Type 6: Lab Data Card

If 5.2 is not zero or one, this card type must be provided to designate the value of elastic modulus versus deviator stress for each lab data point (read in consecutive 10-column fields, four pairs of values per card). A minimum of ten sets of data are to be provided.

6.1 Elastic modulus for data point 1, psi

									•
1	2	3	4	5	6	7	8	9	10

6.2 Deviator stress for data point 1, psi
etc.

						•			
11	12	13	14	15	16	17	18	19	20

Card Type 7: Design Deflection Card

This card designates the magnitude of the design deflection. The deflection load is assumed to be the Dynaflect load. If deflections other than Dynaflect deflections are to be used, see the supplement to this input guide.

7.1 Directive

D	E	F	L	E	C	T	
1	2	3	4	5	6	7	8

7.2 Design deflection, inches
This value should be representative of the more distressed portion of the particular pavement section, hence a minimum confidence level of 90 percent is recommended. Interior deflections are to be used in this procedure. If Card Type 7 is not provided, the value of the subgrade modulus (read from a Card Type 4) will be used in the calculations.

	•								
11	12	13	14	15	16	17	18	19	20

Card Type 8: Corner to Interior Stress Ratio Card

This card is not required. It is used with JC existing pavements, and provides a measured ratio of corner deflection to interior deflection for a given pavement section. This ratio is used to obtain the stress adjustment factor for the determination of remaining life and, for JCP overlays, of estimated overlay life. The default value of the stress adjustment factor is 1.5.

8.1 Directive

C	O	R	N	E	R		
1	2	3	4	5	6	7	8

8.2 Ratio of the deflection measured at a corner (of a JC existing pavement) to that measured at an interior point

					•				
11	12	13	14	15	16	17	18	19	20

Card Type 9: Overlay Card

This card defines the type of overlay to be used. With it, the designer specifies the material type and properties of the overlay and also the presence or absence of a bondbreaker layer.

9.1 Directive

O	V	E	R	L	A	Y	
1	2	3	4	5	6	7	8

9.2 Modulus of overlay, psi

									•
11	12	13	14	15	16	17	18	19	20

9.3 Overlay concrete flexural strength, psi
Default value 690 psi
Leave blank if AC overlay.

				•
26	27	28	29	30

9.4 Overlay material type as follows:
"AC" - asphaltic concrete overlay
"CRCP" - continuously reinforced concrete overlay
"JCP" - jointed concrete overlay

31	32	33	34

9.5 Bonding condition as follows:
Blank - AC overlay
"BOND" - bonded PCC overlay
"UNBD" - unbonded PCC overlay
(If bondbreaker will be used, reduce the maximum allowable number of layers in existing pavement from four to three.)
Note: A fixed value for Poisson's ratio for a specific material type is being used. For more information on the values being used as well as how to use other values, see the supplement to this guide.

35	36	37	38

Card Type 10: Bondbreaker Card

This card is never required. If it does not appear, default values for the bondbreaker layer will be used. Default values will be supplied for any field on the directive which is left blank.

A bondbreaker will be used only if specified through 11.5 or for PCC overlays on pavements without remaining life.

10.1 Directive

B	O	N	D		B	K	R
1	2	3	4	5	6	7	8

10.2 Modulus of bondbreaker, psi
Default value: 100,000 psi

									•
11	12	13	14	15	16	17	18	19	20

10.3 Thickness of bondbreaker, inches
Default value: 1.0 inch
A default value of 0.3 is being used for Poisson's ratio of bondbreaker. For information on how to use other values see the supplement to this guide.

		•		
21	22	23	24	25

Card Type 11: Traffic Designation Card

This directive is never required. It provides up to five design traffic values, for which overlay thicknesses are obtained by interpolation from the overlay thickness versus pavement life curve calculated by the program.

This card designates the number of design traffic values to be read and used for interpolation.

If this card is used, it must be followed by Card Type 14.

11.1 Directive

T	R	A	F	F	I	C	
1	2	3	4	5	6	7	8

11.2 Number of design traffic values
(right justify)

9	10

Card Type 12: Traffic Card

This card designates the magnitudes of design traffic values specified in 13.2.

12.1 Traffic i

									•
10(i-1)+ 1	10(i-1)+ 2	10(i-1)+ 3	10(i-1)+ 4	10(i-1)+ 5	10(i-1)+ 6	10(i-1)+ 7	10(i-1)+ 8	10(i-1)+ 9	10(i-1)+10

Card Type 13: End

This card informs the program that no more problems are to be executed in this run. Every input deck must contain one of this type of cards at the end of the data, even if only one problem is to be analyzed.

13.1 Directive

E	N	D					
1	2	3	4	5	6	7	8

Note: More than one problem may be solved in a simple execution of the program. Each problem is prefaced with a "PROBLEM" directive. All relevant information must be supplied for the first problem of a run as explained above. Subsequent problems in the same run need only specify directives which are changed. All other values will be retained from the preceding problem, with the exception of the corner directive, which applies only to the current problem. All data on a single directive must be supplied, however, even if only one number is being changed.

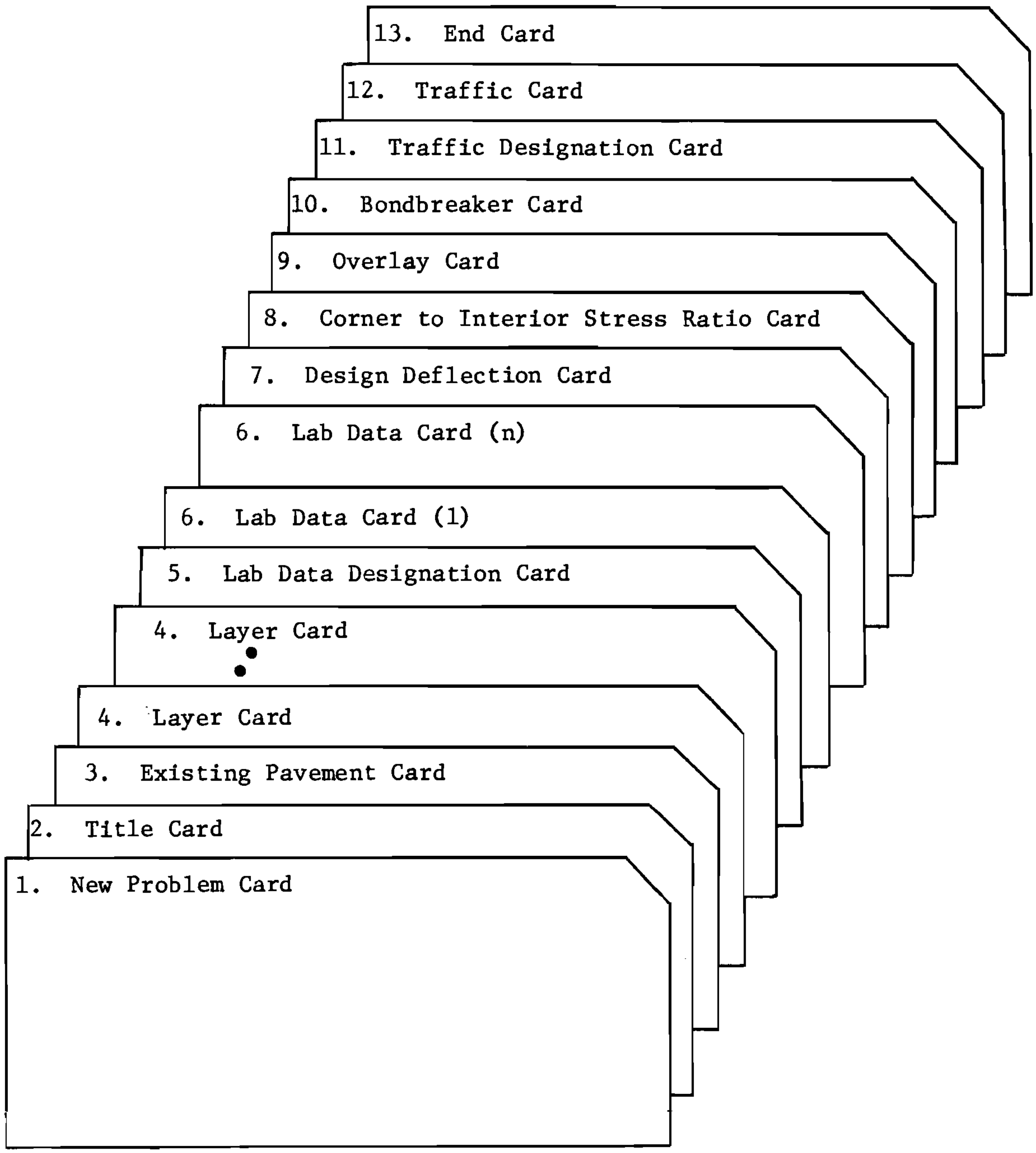


Fig A-9.1. Assembly of RPOD2 data
General Input Guide

SUPPLEMENT TO RPOD2 GENERAL INPUT GUIDE

The purpose of this supplement is to enable the user to change the following "fixed" variables:

(1) Poisson's ratio of existing pavement layers,
(2) Poisson's ratio of overlay,
(3) Poisson's ratio of bondbreaker layer, and
(4) Deflection loads to other than Dynaflect loads.

The following values are used for Poisson's ratio in the RPOD2 program if the general input guide is used:

portland cement concrete	0.15
asphaltic concrete	0.30
stabilized subbases	0.20
granular subbases	0.40
subgrade	0.45

Poisson's Ratio of Existing Pavement Layers

Poisson's ratio values of existing pavement layers can be specified on Card Type 4 if values other than the fixed values are desired, as follows:

Poisson's ratio for layer in 4.2 ______

	•			
26	27	28	29	30

Poisson's Ratio of Overlay

The value of Poisson's ratio of overlay can be specified on Card Type 9 if another value than the fixed value is desired, as follows:

Poisson's ratio for overlay ______

	•			
21	22	23	24	25

Poisson's Ratio of Overlay

The value of Poisson's ratio of the bondbreaker can be specified on Card Type 10 if another value than the fixed value is desired as follows:

Poisson's ratio for bondbreaker layer ______

	•			
26	27	28	29	30

Deflection Loads

Dynaflect load magnitude, pressure, and load geometry are automatically fixed in RPOD2 if the general input guide is used.

It is, however, possible to use any other deflection measuring device and to input the load magnitudes, load pressure, and load geometry.

Card Type 7a: Deflection Load Magnitude Card

This card describes the load magnitude of the deflection measuring device. If this card is not provided, Dynaflect loads will be assumed. From one to four circular loads of equal magnitude may be specified.

7a.1 Directive

L	O	A	D	S			
1	2	3	4	5	6	7	8

7a.2 Load magnitude, pounds

									•
11	12	13	14	15	16	17	18	19	20

7a.3 Load pressure, psi

				•
21	22	23	24	25

Card Type 7b: Deflection Load Geometry Card

If card 8 is provided it must be followed by this card type. To describe the load geometry, it is necessary to select a cartesian coordinate system, in such a way that the locations of the deflection measurements are centered at the origin. The load geometry is described by determining x and y coordinates for each load.

7b.1 x - coordinate for load 1

								•	
1	2	3	4	5	6	7	8	9	10

7b.2 y - coordinate for load 1

								•	
11	12	13	14	15	16	17	18	19	20

.

.

.

7b.7 x - coordinate for load 4 ______

								•	
61	62	63	64	65	66	67	68	69	70

7b.8 y - coordinate for load 4 ______

								•	
71	72	73	74	75	76	77	78	79	80

Figure 9.2 indicates the assembly of the RPOD2 input guide if other loads than Dynaflect loads are used.

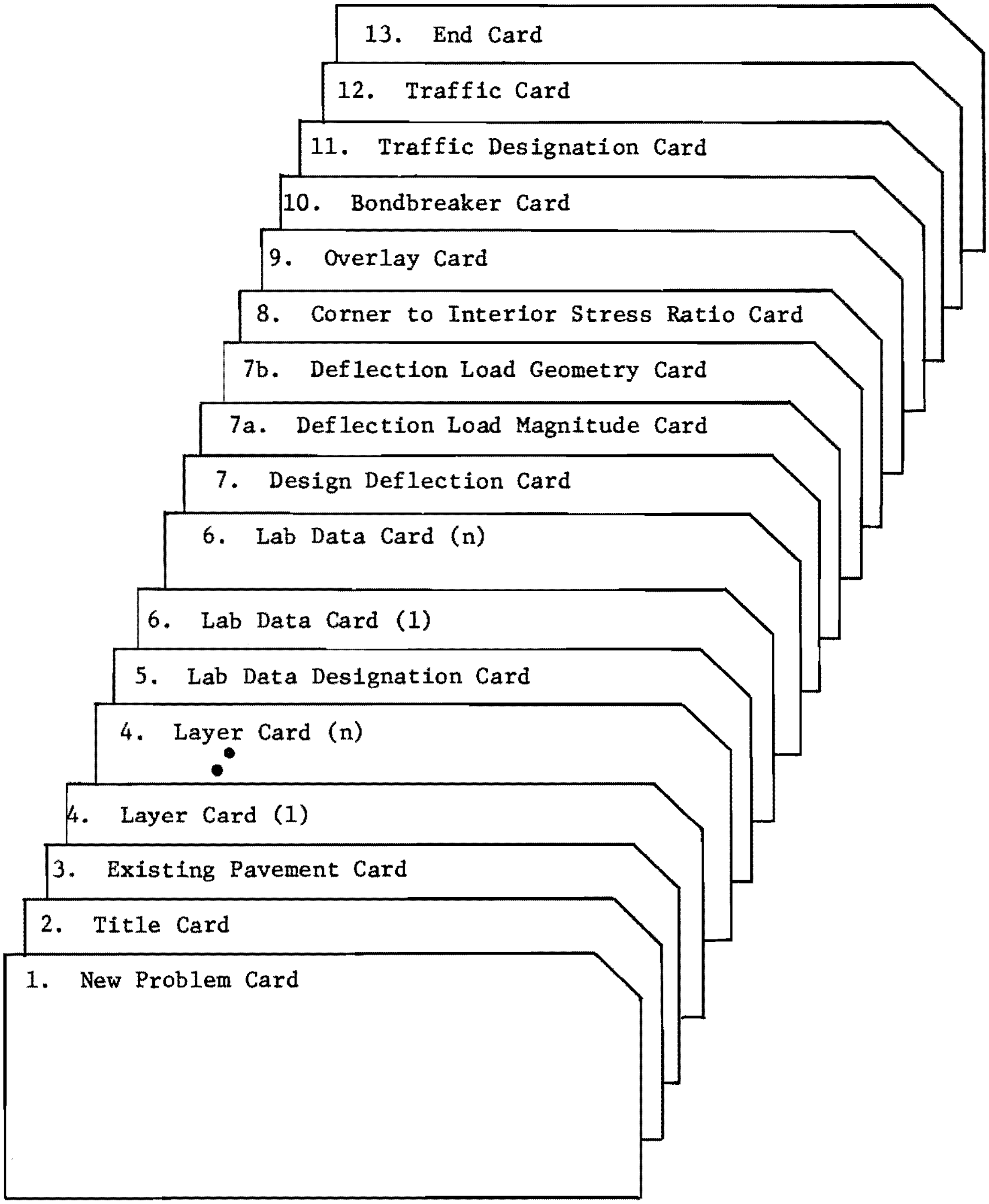


Fig A-9.2. Assembly of RPOD2 data
Special Input Guide

APPENDIX 10

RPOD2 RANDOM ORDER INPUT GUIDE

I N P U T   G U I D E

INSTRUCTIONS TO THE PROGRAM ARE SUPPLIED IN THE FORM OF DIRECTIVES. A DIRECTIVE OCCUPIES EITHER THE FIRST OR SECOND HALF OF A CARD (COLUMNS 1-40 OR 41-80). THE FIRST EIGHT CHARACTERS OF EACH DIRECTIVE CONTAIN A KEYWORD IDENTIFYING THE TYPE OF INFORMATION BEING ENTERED. ALL KEYWORDS MAY BE ABBREVIATED TO THEIR FIRST FOUR CHARACTERS, THE REST OF THE IDENTIFIER IS IGNORED. IF THE FIRST FOUR CHARACTERS OF A DIRECTIVE ARE BLANK, THEN THE WHOLE DIRECTIVE IS SKIPPED, AND READING CONTINUES WITH THE NEXT DIRECTIVE. THIS MEANS THAT ALL DIRECTIVES MAY BEGIN IN COLUMN ONE AT THE OPTION OF THE USER.

MORE THAN ONE PROBLEM MAY BE SOLVED IN A SINGLE EXECUTION OF THE PROGRAM. EACH PROBLEM IS PREFACED WITH A ≠PROBLEM≠ DIRECTIVE AND THE LAST PROBLEM OF A RUN IS TERMINATED BY AN ≠END≠ DIRECTIVE. ALL RELEVANT INFORMATION MUST BE SUPPLIED FOR THE FIRST PROBLEM OF A RUN VIA THE VARIOUS DIRECTIVES WHICH ARE EXPLAINED BELOW. SUBSEQUENT PROBLEMS IN THE SAME RUN NEED ONLY SPECIFY DIRECTIVES WHICH ARE TO BE CHANGED, ALL OTHER VALUES WILL BE RETAINED FROM THE PRECEDING PROBLEM, WITH THE EXCEPTION OF THE CORNER DIRECTIVE, WHICH APPLIES ONLY TO THE CURRENT PROBLEM. ALL DATA ON A SINGLE DIRECTIVE MUST BE SUPPLIED, HOWEVER, EVEN IF ONLY ONE NUMBER IS BEING CHANGED.

ALL DIRECTIVES SHARE A COMMON FORMAT, BUT THE MEANINGS OF THE FIELDS DIFFER DEPENDING ON THE KEYWORD IDENTIFIER. THESE SPECIFIC MEANINGS ARE DESCRIBED BELOW UNDER THE HEADINGS OF THE APPROPRIATE KEYWORDS. THE GENERAL FORMAT IS AS FOLLOWS:

FIELD NAME	COLUMN NUMBERS	TYPE OF VALUE	FORMAT USED
KEYWORD	1-8	CHARACTER	2A4
IVL	9-10	INTEGER	I2
VAL(1)	11-20	REAL	F10.0
VAL(2)	21-25	REAL	F5.0
VAL(3)	26-30	REAL	F5.0
ITYPE(1)	31-34	CHARACTER	A4
ITYPE(2)	35-38	CHARACTER	A4

ADDING 40 TO THE COLUMNS LISTED ABOVE GIVES THE CORRESPONDING COLUMN NUMBER FOR A DIRECTIVE WHICH IS PUNCHED IN THE SECOND HALF OF THE CARD.

SOME DIRECTIVES REQUIRE FURTHER VALUES FROM CARDS WHICH ARE PLACED IMMEDIATELY AFTER THE CARD ON WHICH THE DIRECTIVE APPEARS. THESE CARDS WILL BE READ IN 8F10.0 FORMAT. AS MANY CARDS AS ARE NEEDED TO HOLD THE NUMBER OF VALUES TO BE INPUT SHOULD BE SUPPLIED. IF TWO SUCH DIRECTIVES ARE PUNCHED ON A SINGLE CARD, THE EXTRA CARDS FOR THE DIRECTIVE IN COLUMNS 1 THROUGH 40 SHOULD PRECEDE THOSE REQUIRED FOR THE ONE IN COLUMNS 41 THROUGH 80.

K E Y W O R D   D I C T I O N A R Y

BOND BKR

THIS DIRECTIVE IS NEVER REQUIRED. IF IT DOES NOT APPEAR, THEN THE DEFAULT VALUES FOR THE BOND BREAKER LAYER WILL BE USED. DEFAULT VALUES WILL ALSO BE SUPPLIED FOR ANY FIELD ON THE DIRECTIVE WHICH IS LEFT BLANK.

NOTE THAT A BOND BREAKER LAYER IS ONLY USED IF THE ≠UNBD≠ OPTION IS SELECTED ON THE OVERLAY DIRECTIVE, INDICATING THAT AN UNBONDED OVERLAY IS TO BE BUILT (SEE COMMENTS FOR OVERLAY DIRECTIVE BELOW). IF THIS OPTION IS NOT SPECIFIED, THEN THE BOND BREAKER DESCRIPTION WILL BE IGNORED, ALTHOUGH THE VALUES SUPPLIED WILL STILL BE AVAILABLE TO SUBSEQUENT PROBLEMS.

FIELD DEFINITIONS:

```
VAL(1) = MODULUS OF BOND BREAKER LAYER IN PSI.
         (DEFAULT IS 100000.0)
VAL(2) = THICKNESS OF BOND BREAKER LAYER IN INCHES.
         (DEFAULT IS 1.0)
VAL(3) = POISSON/S RATIO FOR BOND BREAKER LAYER
         (DEFAULT IS 0.3)
```

CORNER

THIS DIRECTIVE IS NEVER REQUIRED. IT IS USED ONLY WITH JCP EXISTING PAVEMENT, AND PROVIDES A MEASURED RATIO OF CORNER DEFLECTION TO INTERIOR DEFLECTION FOR A GIVEN PAVEMENT SECTION. THIS RATIO IS USED TO OBTAIN THE LOAD LOCATION (STRESS ADJUSTMENT) FACTOR FOR THE DETERMINATION OF REMAINING LIFE AND, FOR JCP OVERLAYS, OF ESTIMATED OVERLAY LIFE. THE LOAD LOCATION FACTOR IS DETERMINED USING INTERPOLATION IN A CURVE OF STRESS RATIO VS. DEFLECTION RATIO. THIS DIRECTIVE APPLIES ONLY TO THE PROBLEM WITH WHICH IT WAS READ. DEFAULT VALUE OF THE LOAD LOCATION FACTOR FOR JCP EXISTING PAVEMENT AND JCP/JCP OVERLAYS IS 1.5.

FIELD DEFINITIONS:

```
VAL(1) = RATIO OF DEFLECTION MEASURED AT A CORNER (JCP)
         TO THAT MEASURED AT AN INTERIOR POINT.
```

DEFLECT

THIS DIRECTIVE IS REQUIRED TO DESIGNATE THE MAGNITUDE OF THE DESIGN DEFLECTION. ITS LOCATION (IN CARTESIAN COORDINATES) WITH RESPECT TO THE LOAD WHEELS OF THE DEFLECTION MEASURING DEVICE IS X = 0.0, Y = 0.0.

IF THE LOADS DIRECTIVE IS LEFT OUT, THEN THE DYNAFLECT IS ASSUMED TO BE THE DEFLECTION MEASURING DEVICE AND ONLY THE DESIGN DEFLECTION (DETERMINED FROM MEASUREMENTS BETWEEN THE DYNAFLECT LOAD WHEELS) IS REQUIRED.

IF THIS DIRECTIVE AND THE LOADS DIRECTIVE BOTH ARE LEFT OUT, THEN THE MODULUS READ ON THE SUBGRADE LAYER DIRECTIVE WILL BE USED FOR BOTH EXISTING PAVEMENT AND OVERLAY LIFE CALCULATIONS.

FIELD DEFINITIONS:

VAL(1) = DESIGN DEFLECTION IN INCHES. THIS DEFLECTION SHOULD BE REPRESENTATIVE OF THE MORE DISTRESSED PORTIONS OF THE PAVEMENT, HENCE A MINIMUM CONFIDENCE LEVEL OF 90 PERCENT IS RECOMMENDED.

END

THIS DIRECTIVE INFORMS THE PROGRAM THAT NO MORE PROBLEMS ARE TO BE EXECUTED IN THIS RUN. EVERY INPUT DECK MUST CONTAIN AN END DIRECTIVE, EVEN IF ONLY ONE PROBLEM IS TO BE ANALYZED. THIS DIRECTIVE HAS NO PARAMETERS.

LAB DATA

THIS DIRECTIVE IS REQUIRED IF THE LOAD UNDER WHICH THE DEFLECTION MEASUREMENTS WERE TAKEN DIFFERS SIGNIFICANTLY FROM THE 18-KIP SINGLE AXLE DESIGN LOAD. THIS DATA IS USED TO DETERMINE THE SLOPE OF THE SUBGRADE RESILIENT MODULUS VS. DEVIATOR STRESS CURVE. TWO OPTIONS ARE AVAILABLE FOR INPUTTING THIS DATA.

OPTION 1. THE USER CAN INPUT THE ACTUAL DATA POINTS (FROM LAB TESTS OF SUBGRADE SAMPLES DETERMINING RESILIENT MODULUS AS A FUNCTION OF DEVIATOR STRESS) AND THE PROGRAM WILL COMPUTE THE SLOPE OF THE CURVE. THE NUMBER OF DATA POINTS TO BE READ IS SPECIFIED ON THE DIRECTIVE CARD. PAIRED VALUES OF RESILIENT MODULUS AND CORRESPONDING DEVIATOR STRESS ARE READ FROM CARDS IMMEDIATELY FOLLOWING THIS DIRECTIVE IN 8F10.0 FORMAT. A MINIMUM OF TWO POINTS AND A MAXIMUM OF 10 MAY BE SUPPLIED. NOTE THAT FOUR POINTS CAN BE PUNCHED ON A SINGLE CARD, THAT NO FIELDS CAN BE SKIPPED AND THAT AS MANY CARDS AS ARE NECESSARY TO READ THE DATA MUST BE PROVIDED.

OPTION 2. THIS OPTION ALLOWS THE USER TO INPUT THE SLOPE OF THE LAB DATA CURVE. IT IS IMPORTANT TO NOTE THAT THE SLOPE REPRESENTS A CHANGE IN THE LOG OF THE RESILIENT MODULUS OVER A CHANGE IN THE LOG OF THE DEVIATOR STRESS. SLOPES GREATER THAN OR EQUAL TO ZERO ARE NOT ALLOWED. TO INPUT THIS SLOPE, THE USER MUST SET IVL = 1 (UNDER THE FIELD DEFINITIONS BELOW) AND ENTER VAL(1) = SLOPE.

FIELD DEFINITIONS:

IVL = NUMBER OF LAB DATA POINTS TO BE READ.
VAL(1) = SLOPE OF LAB DATA CURVE (READ ONLY IF IVL = 1).

LAYER

THIS DIRECTIVE DEFINES THE PROPERTIES OF A SINGLE LAYER OF THE EXISTING PAVEMENT. A LAYER DIRECTIVE IS REQUIRED FOR EACH LAYER DOWN TO AND INCLUDING THE SUBGRADE. AFTER THE FIRST PROBLEM IT IS POSSIBLE TO CHANGE THE VALUES FOR A SINGLE LAYER WITHOUT ALTERING THE OTHERS BY INCLUDING A LAYER DIRECTIVE FOR THAT LAYER ONLY. A MAXIMUM OF FOUR LAYERS ARE PERMITTED, UNLESS A BOND BREAKER LAYER IS TO BE USED (SEE OVERLAY DIRECTIVE) IN WHICH CASE ONLY THREE EXISTING LAYERS ARE ALLOWED. IF THE THICKNESS OF THE SUBGRADE LAYER IS INPUT AS ZERO, THEN IT IS ASSUMED TO BE SEMI-INFINITE. OTHERWISE THE PROGRAM WILL SIMULATE THE PRESENCE OF BEDROCK AT THE INDICATED DEPTH BELOW THE TOP OF THE SUBGRADE WHEN PERFORMING DEFLECTION CALCULATIONS.

FIELD DEFINITIONS:

```
IVL = LAYER NUMBER. LAYERS ARE NUMBERED FROM THE TOP DOWN.
      0 < IVL < 5
      (NO DEFAULT VALUE)
VAL(1) = MODULUS OF ELASTICITY FOR LAYER MATERIAL IN PSI.
         (NO DEFAULT VALUE)
VAL(2) = LAYER THICKNESS IN INCHES (ZERO IF INFINITE).
         (NO DEFAULT VALUE UNLESS SUBGRADE)
VAL(3) = POISSON/S RATIO FOR LAYER MATERIAL.
         (DEFAULT VALUE BASED ON MATERIAL TYPE)
ITYPE(1) = MATERIAL TYPE AS FOLLOWS:
           ≠AC  ≠ - ASPHALTIC CONCRETE,
           ≠CRCP≠ - CONTINUOUSLY REINFORCED CONCRETE PAVEMENT,
           ≠GRAN≠ - GRANULAR BASE MATERIAL,
           ≠JCP ≠ - JOINTED CONCRETE PAVEMENT,
           ≠STAB≠ - STABALIZED BASE MATERIAL,
           ≠SUBG≠ - SUBGRADE LAYER.
           (MUST BE JCP OR CRCP IF TOP LAYER)
ITYPE(2) = RIGID BASE INTERFACE TYPE (REQUIRED IF RIGID BASE
           REQUESTED):
           ≠FF  ≠ - FULL FRICTION INTERFACE,
           ≠NF  ≠ - NO FRICTION INTERFACE.
           (NO DEFAULT VALUE)
```

LOADS

THIS DIRECTIVE DESCRIBES THE LOAD GEOMETRY OF THE DEFLECTION MEASURING DEVICE WITH RESPECT TO THE LOCATION OF THE DEFLECTION MEASUREMENTS, X = Ø.Ø, Y = Ø.Ø. IF THIS DIRECTIVE IS LEFT OUT, THEN THE DYNAFLECT IS ASSUMED TO BE THE DEFLECTION MEASURING DEVICE (SEE DEFLECT DIRECTIVE).

FROM ONE TO FOUR UNIFORM CIRCULAR LOADS MAY BE MODELED WITH THIS DIRECTIVE. A SINGLE LOAD FORCE AND PRESSURE ARE INPUT FOR ALL OF THE LOADS. AN EXTRA CARD MUST BE PROVIDED IMMEDIATELY AFTER THIS DIRECTIVE, SPECIFYING THE LOCATIONS OF EACH OF THE LOADS AS PAIRS OF X AND Y COORDINATES (IN 8F10.Ø FORMAT).

FIELD DEFINITIONS:

```
IVL    = NUMBER OF LOADS (Ø < IVL < 5).
VAL(1) = DEFLECTION LOAD FORCE IN POUNDS.
VAL(2) = DEFLECTION LOAD PRESSURE IN PSI.
```

OVERLAY

THIS DIRECTIVE DEFINES THE TYPE OF OVERLAY TO BE BUILT. WITH IT THE DESIGNER SPECIFIES THE MATERIAL TO BE USED, ITS PROPERTIES, AND THE PRESENCE OR ABSENCE OF A BOND BREAKER LAYER. IT IS IMPORTANT TO NOTE THAT THE INCLUSION OF A BOND BREAKER LAYER (VIA THE ≠UNBD≠ OPTION) REDUCES THE MAXIMUM NUMBER OF EXISTING PAVEMENT LAYERS FROM FOUR TO THREE. AN OVERLAY DIRECTIVE IS REQUIRED FOR THE FIRST PROBLEM OF EVERY RUN.

FIELD DEFINITIONS:

```
VAL(1) = MODULUS OF OVERLAY MATERIAL IN PSI.
         (NO DEFAULT VALUE)
VAL(2) = POISSON/S RATIO FOR OVERLAY MATERIAL.
         (DEFAULT VALUE BASED ON MATERIAL TYPE)
VAL(3) = CONCRETE FLEXURAL STRENGTH FOR PCC OVERLAY, IN PSI.
         (DEFAULT = 690.Ø)
ITYPE(1) = MATERIAL TYPE AS FOLLOWS:
           ≠AC≠ - ASPHALTIC CONCRETE OVERLAY,
           ≠CRCP≠ - CONTINUOUSLY REINFORCED CONCRETE PAVEMENT,
           ≠JCP≠ - JOINTED CONCRETE PAVEMENT.
ITYPE(2) = BOND BREAKER CONDITION AS FOLLOWS:
         = BLANK IF AC OVERLAY,
         = ≠BOND≠ IF BONDED PORTLAND CEMENT OVERLAY,
         = ≠UNBD≠ IF UNBONDED PCC OVERLAY.
           (BOND BREAKER LAYER WILL BE USED)
```

PAVEMENT

THIS DIRECTIVE DESCRIBES THE CONDITION OF THE EXISTING PAVEMENT. IT IS REQUIRED FOR THE FIRST PROBLEM OF EVERY RUN. NOTE THAT LAYER DIRECTIVES ARE ALSO REQUIRED FOR EACH LAYER INCLUDING THE TOP ONE.

FIELD DEFINITIONS:

```
IVL = NUMBER OF LAYERS IN EXISTING PAVEMENT DOWN TO AND
      INCLUDING THE SUBGRADE.  AT LEAST ONE AND NOT MORE
      THAN FOUR LAYERS MAY BE SPECIFIED (THREE IF BOND
      BREAKER LAYER SPECIFIED ON OVERLAY DIRECTIVE).
      (NO DEFAULT VALUE)
VAL(1) = NUMBER OF 18 KIP EQUIVALENT SINGLE AXLE WHEEL LOADS
         APPLIED TO DATE (PUNCHED WITH DECIMAL POINT).
         (DEFAULT IS 1.)
VAL(2) = CONCRETE FLEXURAL STRENGTH FOR EXISTING PAVEMENT,
         IN PSI.
         (DEFAULT IS 690.Ø)
ITYPE = 8-CHARACTER FIELD SPECIFYING PAVEMENT CONDITION:
        BLANK       - NO CRACKING OR VOIDS PRESENT.
        ≠VOID     ≠ - VOIDS PRESENT BUT NO CRACKING.
        ≠TYPE 1,2≠  - TYPE 1 OR 2 CRACKING PRESENT.
        ≠VOID 1,2≠  - TYPE 1 OR 2 CRACKING WITH VOIDS PRESENT.
        ≠TYPE 3,4≠  - TYPE 3 OR 4 CRACKING PRESENT.
        ≠MECH BKN≠  - PAVEMENT WILL BE MECHANICALLY BROKEN
                      PRIOR TO OVERLAY.
```

PROBLEM

THIS DIRECTIVE SIGNALS THE BEGINNING IF A GROUP OF DIRECTIVES THAT DESCRIBE A SINGLE PROBLEM FOR WHICH SOLUTIONS OF ALLOWABLE TRAFFIC AS A FUNCTION OF OVERLAY THICKNESS ARE DESIRED. IT PERMITS THE USER TO SPECIFY A TITLE AND A PROBLEM NUMBER WHICH WILL APPEAR IN THE PRINTED OUTPUT AND CAN BE USED TO IDENTIFY THE RESULTS. IF A NON-ZERO DIGIT APPEARS ANYWHERE BETWEEN COLUMNS 11 AND 2Ø OF THIS DIRECTIVE, THEN AN 8Ø-CHARACTER TITLE IS READ FROM AN EXTRA CARD WHICH IMMEDIATELY FOLLOWS THE PROBLEM DIRECTIVE. THIS TITLE WILL REMAIN IN EFFECT UNTIL ANOTHER IS PROVIDED.

FIELD DEFINITIONS:

```
IVL = PROBLEM NUMBER (IVL < 1ØØ).
      (DEFAULT IS 1 IF FIRST PROBLEM, PREVIOUS PROBLEM NUMBER
       PLUS ONE OTHERWISE)
VAL(1) = Ø IF NO TITLE CARD,
       > Ø IF TITLE CARD FOLLOWS.
```

TRAFFIC

THIS DIRECTIVE IS NEVER REQUIRED. IT PROVIDES UP TO 5 DESIGN TRAFFIC VALUES, FOR WHICH OVERLAY THICKNESSES ARE OBTAINED BY INTERPOLATION IN THICKNESS AS A FUNCTION OF LOG(PREDICTED APPLICATIONS TO FAILURE). CONSERVATIVE OVERLAY THICKNESSES ARE CALCULATED IF THE SPECIFIED FATIGUE LIFE IS LESS THAN THAT FOR THE RECOMMENDED MINIMUM OVERLAY THICKNESS.

AN EXTRA CARD MUST BE PROVIDED IMMEDIATELY AFTER THIS DIRECTIVE, SPECIFYING THE DESIGN TRAFFIC VALUES IN 5F10.0 FORMAT.

FIELD DEFINITIONS:

IVL = NUMBER OF DESIGN TRAFFIC VALUES (LESS THAN OR EQUAL TO 5) (DEFAULT: 0)

APPENDIX 11

RECOMMENDED TEST PROCEDURE DYNAMIC MODULUS
OF ELASTICITY OF ASPHALT CONCRETE

(Reprinted from Austin Research Engineers, Asphalt Concrete Overlays of Flexible Pavements, Vol. 2, Appendix B, pp. 68-76, Report No. FHWA 75-76, June, 1976) (Ref 37)

APPENDIX 11. TEST PROCEDURE FOR DYNAMIC MODULUS OF ASPHALT CONCRETE (Ref 37)

Apparatus

1. Testing machine--the two types of equipment capable of producing one or more of the load pulses required are electrohydraulic testing machines with function generators and pneumatic machines with fluidic timers. The former is readily available from well known sources and much more expensive, but has flexibility in shapes of pulses that may be generated. The latter is limited to square pulses, but is much less expensive, simple to operate and almost maintenance free. The pneumatic machine is basically of the type developed by Seed et al. (1967) for resilient modulus testing of soils, but a larger loading piston is used to produce the load required for asphaltic concrete specimens. A photograph of the testing machine is shown in Figure A-11.1.

2. Strain measurement system--both LVDT's (Linear Variable Differential Transformers) and strain gauges have been used successfully.

   The LVDT's are usually used in pairs on the opposite sides of the sample measuring vertical movement between two horizontal clamps firmly attached to the sample (See Figure C.1). The LVDT transducers are attached to one clamp, and rods that can be screwed in or out for zeroing to the other with the LVDT cores on the opposite end fitting into the transducers. Change in sample length between the clamps will result in an increase in voltage output through the transducer and a calibrated trace on a strip recorder's chart paper.

   Wire strain gauges are also used in pairs bonded at mid-height on opposite sides of the specimen. The gauges are wired in a Wheatstone Bridge circuit with two active gauges on the test specimen and two temperature compensating gauges similarly bonded on an unstressed specimen exposed to the same environment as the test specimen.

   The LVDT's or strain gauges should have the capability for operating across the range of strains occurring in the specimen and should, in combination with signal conditioning equipment and the recorder, produce traces on the chart

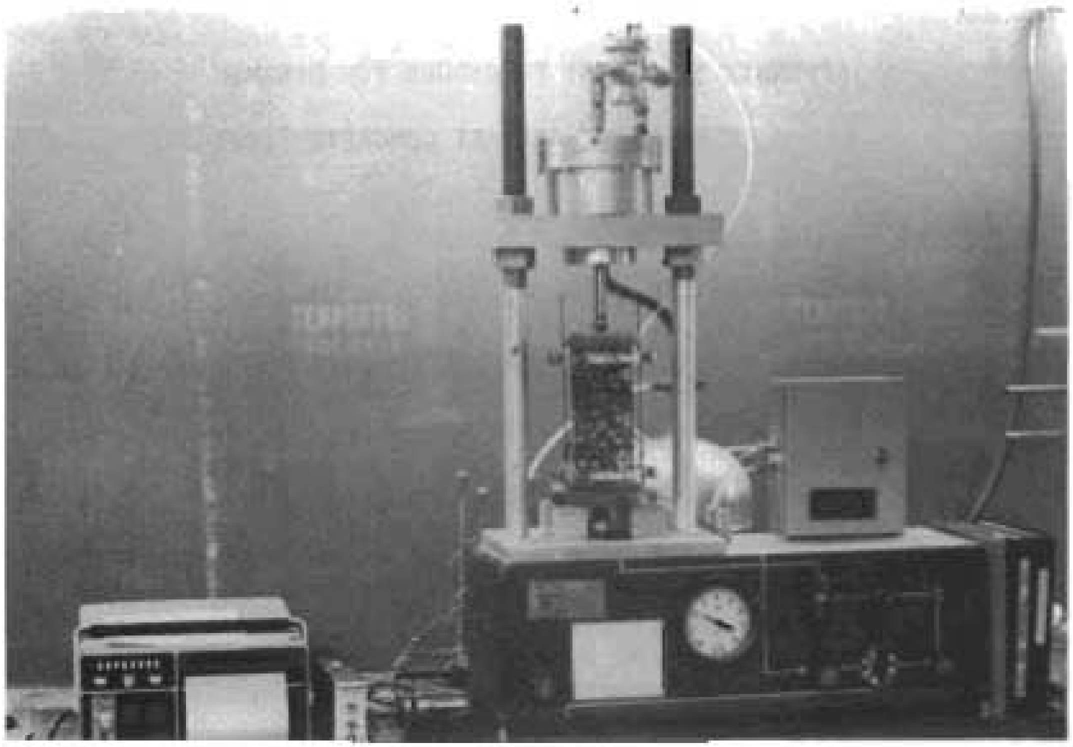


Fig A-11.1. Photographs of dynamic modulus testing equipment.

paper that may be easily and accurately measured for the smallest strains (those at a specimen temperature of 40°F that will be measured). The system should at its highest sensitivity setting display 4 micro strain units or less per mm on the recorded chart for strain gauges. As the LVDT's measure total strain across a gauge length (usually around 4 inches or more), and a calculation is made to obtain unit strain, sensitivity must be measured in total strain.

As an example, 56 micro inches of movement will occur over a 4-inch gauge length in a material having a very high dynamic modulus of 2,500,000 psi and subjected to 35 psi of vertical stress. Assuming .05 inches (12.7 mm) is the smallest trace that may be accurately measured, a sensitivity allowing measurement of 4.4 micro inches per mm of chart width should be required. A sensitivity of 3 micro inches per mm should be sufficient for all practical condition-.

The recorder should have sufficiently rapid response to swing almost full scale in .01 second. Recording oscillographs of good quality using light sensitive paper have proven satisfactorily responsive.

While both LVDT's and bonded strain gauges may be used successfully, more of the material is directly active in the test when measurements are made over a longer gauge length for LVDT's compared to the nominal length for strain gauges. In either case, special attention is warranted to assure firm attachment to the sample. The LVDT clamps should each have four pointed set screws that insure against clamp movement.

3. Load measurement system--load measurements for the varying load used for a haversine pulse produced by the electrohydraulic machine are usually made with a load cell generating a second trace on the chart paper. This may also be used for the two-phase (on-off) load for the square pulse from the pneumatic machine, but is not necessary in this case as it is sufficient to precisely control the air pressure to the air piston. The air pressure is precisely calibrated in terms of load delivered to the sample. Sensitivity in either case must be sufficient to allow accurate calculation of vertical load and thus stress.

4. Temperature control system--One or more temperature chambers having a capacity for 6 specimens may be used to produce temperatures of 40°F, 70°F, and 100°F (5°, 21° and 38°C) controlled to ± 1°F (.5°C).

5. Loading plate--a hardened, steel plate no less than 1/4 inches thick and with a diameter equal to that of the specimen is required to transfer load from the testing machine to the specimen.

Preparation of Specimens

1. Laboratory-prepared Specimens: Most testing agencies have their own means of preparation and compaction to produce density and stability specimens that serve as the basis for material specifications. Compaction of specimens for dynamic modulus testing should be accomplished by the procedures in use by the agency involved. A specimen suitable for vertical compression testing requires modifications to produce a cylindrical specimen twice as long as its diameter.

   One optional procedure is to prepare the bituminous mixture as specified by ASTM Method D 1560, "Test for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of the Hveem Apparatus." Compaction is then accomplished with a California Kneading Compactor using steel molding cylinders with 1/4-inch wall thickness, inside diameters of 4 inches and a height of 10 inches (twice as high as that recommended by ASTM D1561). A pre-heated insulated feeder trough and a paddle are used as in ASTM D 1561 to introduce the mixture into the mold, but in a different manner. One half of the approximately 4000 grams of bituminous mixture is weighed out and introduced into the trough. A paddle is then used to push 30 approximately equal portions into the mold continuously and uniformly while 30 tamping blows at a pressure of 250 psi are applied. The second half of the mixture is compacted in the mold in the same manner. This is followed immediately by application of a static load to the specimen while still in the mold. The load is applied with a compression machine by the double plunger method in which metal followers are employed as free-fitting plungers on top and bottom of the specimen. This load is applied at a rate of 0.05 inches per minute until an applied pressure of 1000 psi is reached. The load is then removed immediately. After the specimen is sufficiently cooled so that it will not deform in the mold, it is removed from the mold and placed on a smooth flat surface to cool to room temperature. The resulting bulk specific gravity is reported to approximate very closely that of specimens prepared as specified by ASTM D 1559, "Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus," and by ASTM D 1561, "Preparation of Specimens by means of California Kneading Compactor."

Whatever the procedure used, the diameter of the specimens should be four or more times the maximum nominal size of aggregate specified. A diameter of 4 inches is normally used with a length of approximately 8 inches. A minimum of three specimens should normally be tested to suitably account for variability in the materials.

2. Pavement cores--During field sampling obtain cores having a minimum height to diameter ratio of 2 and with diameters not less than two times the maximum nominal size of an aggregate particle. Because of the high variability in dynamic modulus found for pavements in the field, six specimens should normally be tested to characterize a pavement section. The cores should be taken from locations selected to provide a representative sample of the pavement section.

   As most highway pavement layers are less than 8 inches thick, it may be necessary to test cores with a two-inch diameter or else it may not be possible to use these procedures. For thin pavements, it is possible to obtain a dynamic modulus through dynamic indirect tensile tests on specimens of 4 to 6 inches in diameter.

TEST PROCEDURES

Capping of Specimens--All specimens should be capped with a sulphur mortar as specified by ASTMC617. The procedures for capping may be the same as those used for Portland cement concrete compression specimens except that a special capping fixture for four-inch diameter specimens must be used.

Place test specimens in a control temperature cabinet and bring them to the specified test temperature. A dummy specimen with a thermo couple in the center can be used to determine when the desired test temperature is reached or the specimens may remain in the controlled temperature environment overnight to insure even distribution of temperature.

Place specimen with strain gauges for strain measurement directly into the loading apparatus (strain gauges are bonded on the specimen prior to placement in the temperature cabinet). Then connect the strain gauge wires to the measurement system, place the hardened steel disk on top of the specimen and center both under the loading apparatus. Adjust and balance the electronic measuring system as necessary.

For specimens using LDVT's for strain measurement, the clamps and LDVT's are to be placed on the specimen as rapidly as possible before the specimen with the hardened steel disk on top is placed under the loading apparatus. The LDVT's must be zeroed prior to continuing.

Apply the selected pulse as previously described without impact, turning on the recorder about every 50 cycles to obtain a few traces of the strains caused by the load pulse. The resilient strain may be measured on the trace as the distance transverse to the edge of the chart paper between the maximum of the trace and the minimum of the trace just before the next load is applied. This measures the strain recovered. Comparisons of the amounts of strain after each 50 cycles should reveal that the amount of strain has stabilized around 200 cycles of loading. The resilient strain after the magnitudes have stabilized may be used in calculating the dynamic modulus.

The specimen may now be removed from the testing machine and disconnected from the strain measurement equipment, and stored until returned to the temperature cabinet in preparation for testing at a new temperature.

It is important to test in order of increasing temperatures of 40°F, 70°F, and 100°F, as the stiffness will decrease almost an order of magnitude between 40°F and 100°F. Testing in this order will allow the least possible amount of permanent strain prior to subsequent testing.

All portions of the procedure should be completed as quickly as possible to minimize the variation in temperature in the sample prior to completion of the test. The testing should be completed on a specimen within two minutes after it is removed from the temperature control cabinet. While this may not be possible when LDVT's are used because it takes that long to place the clamps and LDVT's in position and measure the gauge length and another two minutes to zero them, the test may be conducted rapidly enough to avoid important change in temperature. The clamps and LVDT's can also be placed on the specimen prior to removal from the temperature cabinet. If testing is conducted in a room or a temperature control cabinet meeting the specified temperature control tolerance limits, the requirement for expedited testing may be waved.

In the unlikely event that excessive deformation (greater than 2500 micro units of strain) occurs the maximum loading stress level may be reduced from 35 psi to 17.5 psi, and testing continued as described above.

Calculations

The measured quantity from dynamic modulus testing is the resilient strain taken after sufficient cycles of loading for it to stabilize. If the square load pulse is used, the resilient strain is modified by multiplication by 0.8 to better represent strains from a wheel load. The vertical stress is generally controlled at 35 psi (or 17.5 psi if exceptionally high strains occur as previously discussed).

The general equation for calculation is:

$$E^*(T) = \frac{o}{{}_{o}(T)} \quad \text{or} \quad \frac{35}{{}_{o}(T)} \qquad \text{(A-11.1)}$$

where:

$E^*(T)$ = Dynamic Modulus for the Asphaltic Mixture at temperature T

${}_{o}(T)$ = Resilient unit strain from dynamic modulus test with the specimen at temperature T.

${}_{o}$ = Vertical test stress recommended at 35 psi.

Plot the results as indicated in Figure A-11.2 from the replicate tests. From these plots, a suitable curve may be selected for design or analysis. A mean curve with a rough approximation (dependent on number of replicate tests) of the variation around the mean may be appropriate for most uses.

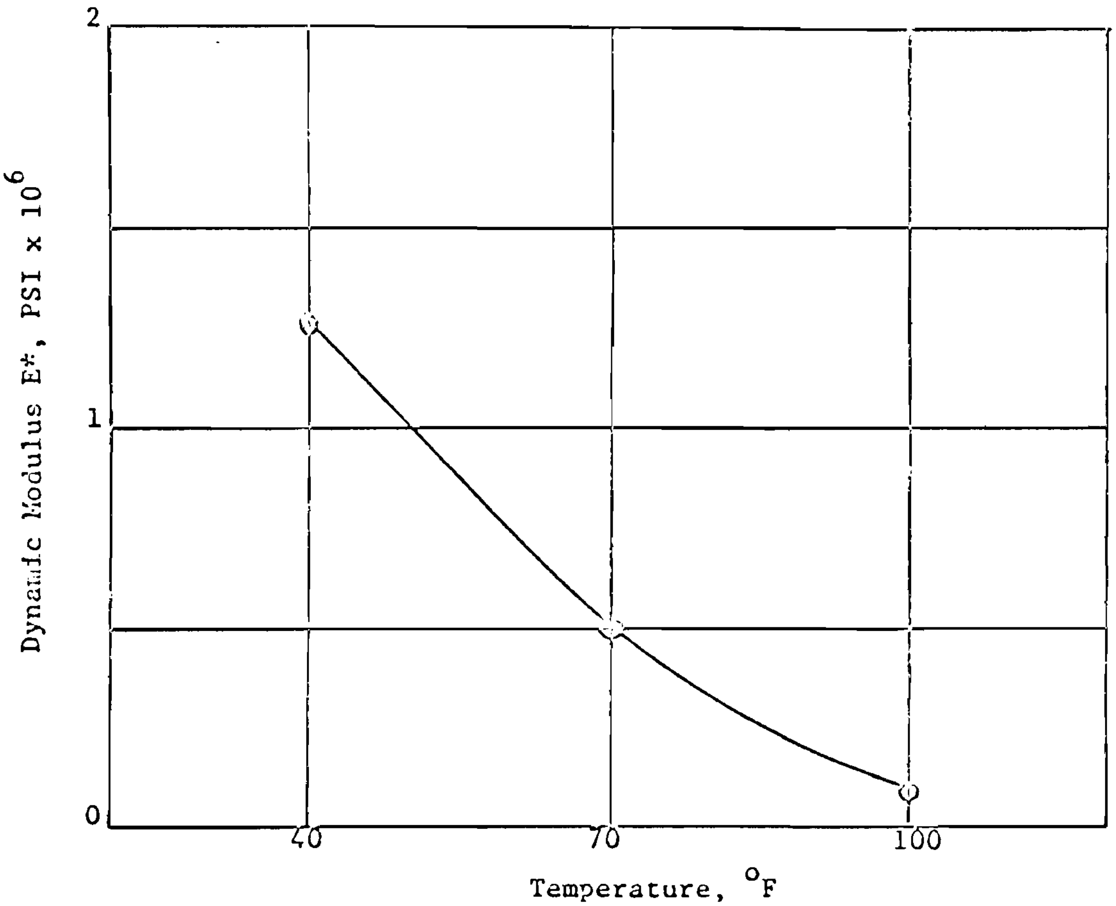


Figure A-11.2. Typical Dynamic Test Results for an Asphaltic Concrete Specimen Tested at Three Temperatures

APPENDIX 12.

INDIRECT TENSILE TEST METHOD FOR DYNAMIC MODULUS OF ASPHALT MIXTURES

APPENDIX 12. INDIRECT TENSILE TEST METHOD FOR DYNAMIC MODULUS OF ASPHALT MIXTURES

GENERAL

At present there is no standard method of testing the dynamic modulus of asphalt mixtures using the indirect tensile test method although a method is currently being considered by the American Society for Testing Materials. The method given here has been used at the Center for Highway Research at The University of Texas at Austin. Most valuable information in this respect has been obtained through private discussion with Dr. Thomas W. Kennedy who has developed and extensively used the test in research through the Center for Highway Research.

This testing method is useful for determining dynamic modulus values on either laboratory prepared specimens or field recovered cores of asphaltic concrete using the repeated-load indirect tensile test. The repeated-load indirect tensile test for dynamic modulus is conducted by applying a compressive load with a haversine, square wave or trapezoidal wave form. The loads act parallel to and along the vertical diametral plane of a cylindrical specimen of asphalt concrete (Fig A12.1). The test can be performed at different temperatures and loading frequencies and magnitudes. The resulting recoverable horizontal deformation of the specimen is measured and used to calculate the dynamic modulus if a value of Poisson's ratio is assumed. By also measuring the recoverable vertical deformation, Poisson's ratio can be calculated as well. This method is applicable to asphalt concrete, blackbase and other asphalt-treated paving mixtures.

EQUIPMENT

Testing Mahine

Loading equipment must be capable of applying a load pulse over a range of frequencies, load durations, and load magnitudes. Electro-hydraulic testing machines with function generators capable of producing the prescribed wave form are highly recommended for indirect tensile testing. Commercially

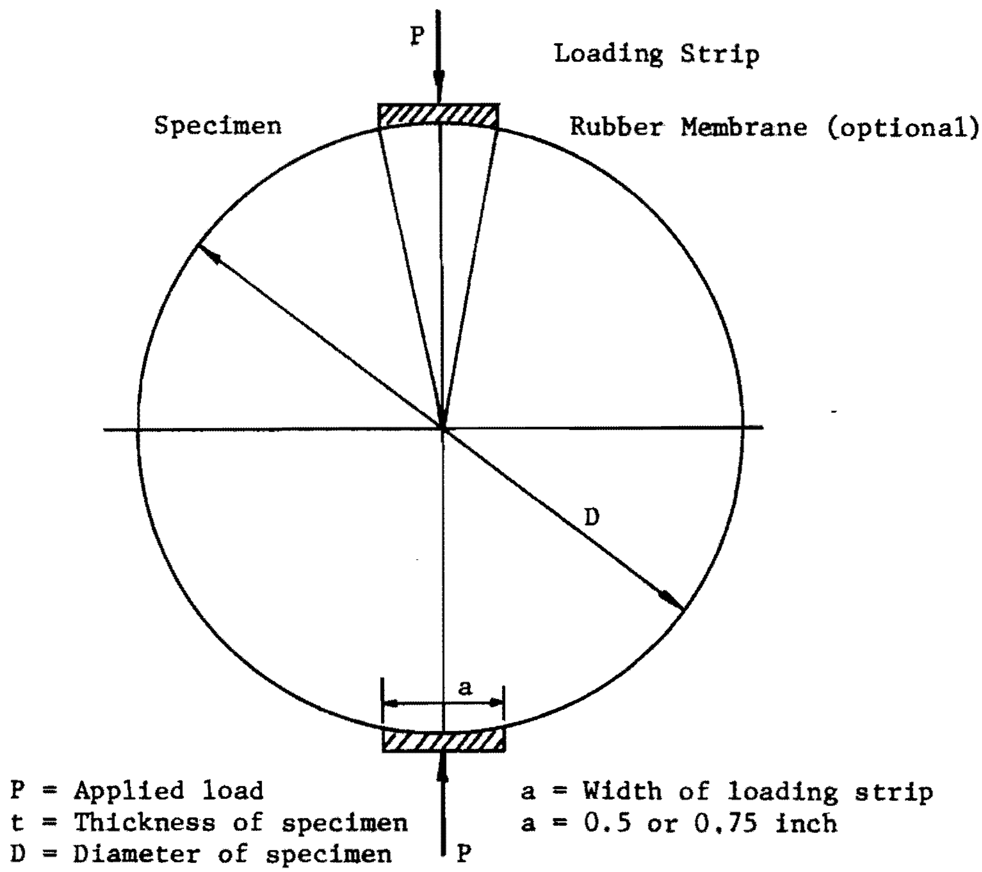


Fig A12.1. Indirect tensile test.

available laboratory testing machines using pneumatic repeated loading can also be used, provided they have the required load capability.

Temperature Control System

This test should be performed at a specific temperature and, therefore, the use of a temperature chamber large enough to house the testing equipment as well as the specimens to be tested is essential. The temperature control system should be capable of controlling the temperature over the full range of testing temperatures.

Measurement System

Loads should be measured and recorded or accurately calibrated prior to testing. A recorder or other measuring device should be included in the measurement system to measure horizontal and vertical deformations to an accuracy of .0001 inch (2.5 microns) of deformation. The recorder should be independent of the frequency of testing up to 1.0 Hz.

Deformation Measurement

LVDT's or other suitable devices are to be used to measure vertical and horizontal deformation. Horizontal deformation measurements are to be taken at mid-height. Trans-TEX Model 350-000 LVDT and Statham UC-3 transducers are recommended for this purpose. The guages should be wired to preclude the effects of eccentric loading so as to give the algebraic sum of the movement of each side of the specimen.

Load Measurement

Loads are to be measured with an electronic load cell.

Loading Strip

A loading strip with a radius equal to that of the specimen is required to transfer the load from the testing machine to the specimen. The strip should be made of steel or aluminum and should be 0.5 or 0.75-inch (12.7 or 19.1-mm) wide for 4 or 6-inch (101.6 or 152.4-mm) specimens, respectively and edges should be rounded in order not to cut the specimen during testing. If Poisson's ratio is assumed and vertical deformations are not to be determined a thin hard rubber membrane between the loading strip and the specimen is useful for specimens with rough textures. This membrane reduces the impact loading effects.

PREPARATION OF SPECIMENS

Either laboratroy prepared specimens or pavement cores can be used in this testing procedure. In both cases the minimum diameter of the specimen must be 4 inches (102 mm) but not less than four times the maximum nominal size of the aggregate particles.

Laboratory prepared specimens should be prepared according to ASTM Method D1561. The specimens should have a height of at least two inches (50 mm).

Pavement cores should have a minimum height of 1.5 to 2 inches (38 to 50 mm) and should have relatively smooth parallel faces.

Testing Procedure

The recommended procedure for conducting the indirect tensile test to determine dynamic modulus is as follows.

(1) Specimens are to be placed in the temperature chamber at the specified testing temperature for at least 24 hours. If a dummy specimen with a thermocouple in the center is used to determine when the specimens have reached the desired temperature, testing can continue as soon as the specified temperature is reached.

(2) Place the specimen in the loading apparatus with loading strips.

(3) Adjust and balance the electronic measuring system as necessary.

(4) Apply the repeated haversine, or other suitable waveform loading, to the specimen. Care should be taken to prevent impact and the load should be applied for the minimum time to obtain uniform deformation readout. The test should be conducted at the specified temperature at frequencies of 0.33, 0.5 and 1.0 Hz. The recommended load range is between 10 and 50 percent of the tensile strength. The tensile strength is to be determined from a destructive test and Eq A12.3.

(5) Monitor the vertical and horizontal deformations during the test, using the recording equipment. Figure A12.2 shows a typical load pulse-deformation plot.

(6) The test should be completed within two minutes after the specimen has been removed from the control cabinet. This requirement pertains only to the situation where testing is not conducted inside a temperature controlled chamber.

(7) At least three tests should be conducted on a specimen, by rotating it and loading through another diametral plane.

Calculations

The dynamic modulus is calculated as follows:

(1) Measure the recoverable horizontal and vertical deformations over at least three loading cycles as indicated on Fig A12.2.

(2) Calculate the resilient dynamic modulus E_R and Poisson's ratio μ using the following equations:

$$E_R = \frac{P(\mu + 0.27)}{t\Delta x}, \text{ psi} \tag{A12.1}$$

and

$$V = 3.59 \frac{\Delta x}{\Delta y} - 0.27 \tag{A12.2}$$

where

P = repeated load, pound,

V = Poisson's ratio,

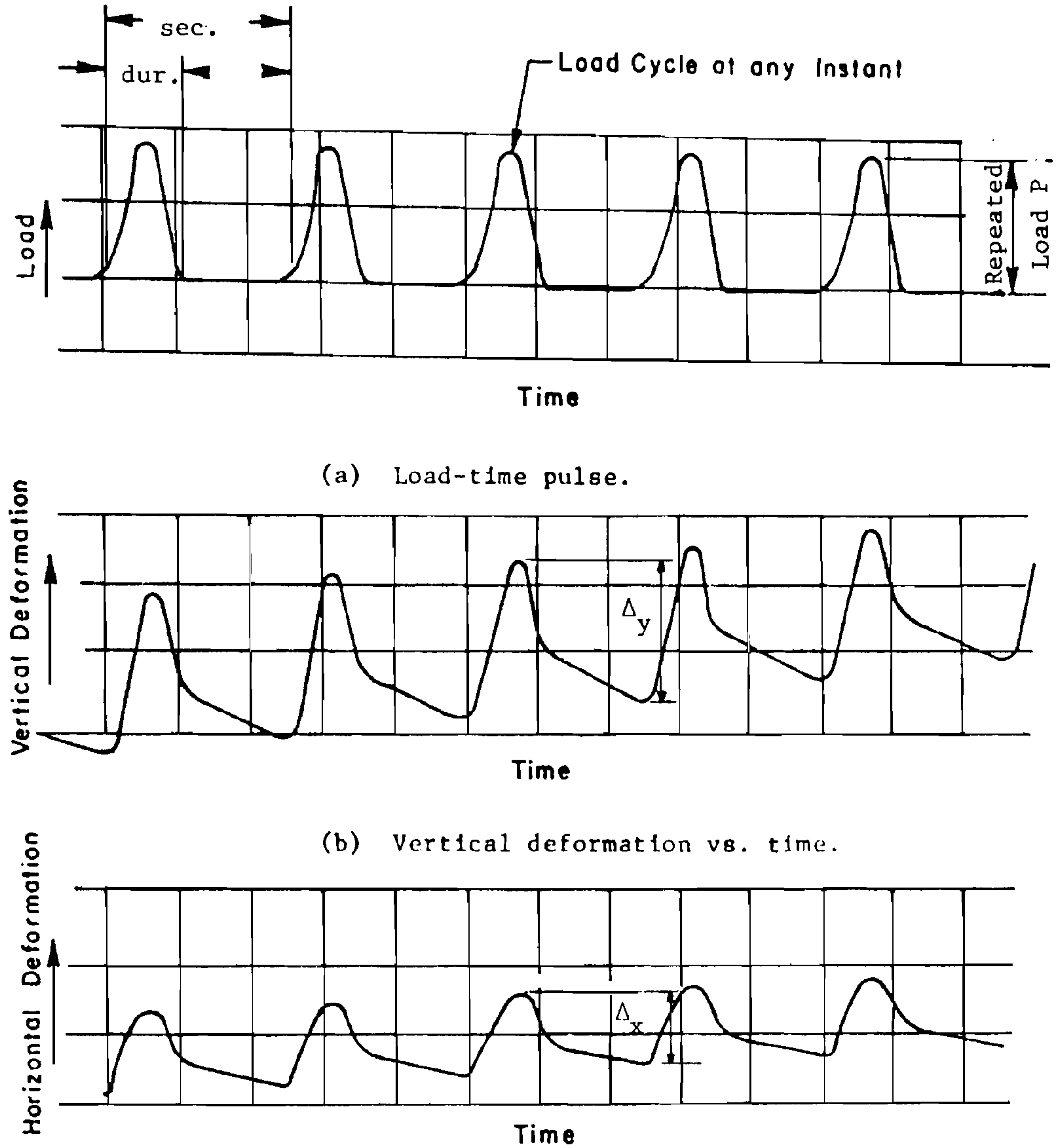
t = thickness of specimen, inches,

Δx = recoverable horizontal deformation, inches, and

Δy = recoverable vertical deformation, inches.

Tensile strength S_T of the asphalt concrete can be calculated from a destructive test as follows:

$$S_T = \frac{2P_{ult}}{\pi t D} \tag{A12.3}$$



(a) Load-time pulse.

(b) Vertical deformation vs. time.

(c) Horizontal deformation vs. time.

Fig A12.2. Typical load and deformation versus time relationships for repeated-load indirect tensile test.

where

P_{ult} = the ultimate applied load required to fail the specimen, pounds,

t = thickness of specimen, inches, and

D = diameter of specimen, inches.

APPENDIX 13

RECOMMENDED TEST PROCEDURE--RESILIENT MODULUS OF ELASTICITY FOR BASE, SUBBASE, AND SUBGRADE MATERIALS

(Reprinted from Austin Research Engineers, Asphalt Concrete Overlays of Flexible Pavements, Vol. 2: Appendix C, pp. 77-81, Report No. FHWA-RD-75-76, June 1975) (Ref 37)

APPENDIX 13. RESILIENT MODULUS TESTING FOR PAVEMENT EVALUATION AND DESIGN (Ref 37)

General

The use of elastic layer theory in the prediction of stresses and deflections in pavement systems gives added importance to accurate determination of the modulus of elasticity of base, subbase and subgrade materials. Overwhelming evidence indicates that the modulus of elasticity for most soils is stress sensitive and varies with repeated loading. An adequate laboratory simulation of soil in a base of subgrade then requires application of loads repetitiously to model the intensities and durations of wheel loads.

The triaxial load cell was developed years ago to allow better simulation of a sample of soil in place in the field. The lateral pressure in the cell simulates the resistance of surrounding soil to lateral displacement of the soil sample under vertical load. Equipment capable of applying closely controlled vertical load pulses to represent the intensity and duration of the stresses induced by a passing vehicle was recently introduced. Linear variable differential transformers (LVDTs) are used to produce electronic signals proportional to the amount of movement in the sample. These signals are conditioned for input to a strip recorder, which plots the deformation versus time. The Resilient Modulus, M_R, is the ratio of stress to resilient strain taken after an appropriate number of cycles of loading and at an appropriate level of vertical stress.

The Resilient Modulus derived under conditions closely simulating those the sample will experience in the field is used in lieu of a statis modulus of elasticity (derived from long-term one-cycle tests) to characterize the material for the particular analytical procedure.

Failure to recognize the effects of repetitive loading on soils will involve overestimation of the modulus of elasticity for clay soils and underestimation for granular soils.

Sample Requirements for Resilient Modulus Testing

Resilient Modulus testing may be conducted on undisturbed samples representing natural state in the field, samples compacted to optimum density or samples compacted to some intermediate density. Samples may be delivered to the laboratory as undisturbed samples wrapped to avoid moisture change and packed to protect the structural integrity of the sample or as disturbed samples to be compacted to some density.

As most of the resilient modulus testing done is conducted on samples with a diameter of 2.8 inches, a 3-inch thin-wall tube should be used for collection of undisturbed samples whenever possible. For cohesive soils, larger tubes may be used and the samples trimmed in the laboratory. Samples with a diameter of 1.4 inches may be tested but require considerable more effort and the results are not considered to be quite as accurate. If the material to be tested is to be used in a new subbase or subgrade for a pavement system, the density must be furnished or determined. This density should be consistent with the density control planned in the field; i.e., if 95% of modified AASHO compaction is to be specified, the optimum density can be established using modified AASHO compactive energy and compact the sample to 95% of that amount. If some natural density is desired, it may be specified and the samples can be compacted to that amount. The latter requires some cut and try. Moisture contents to simulate the field must also be specified or determined. Samples to be compacted in the laboratory may be sent in disturbed state in bags. Four pounds is sufficient for a single triaxial specimen.

Test Design

The repetitive loading triaxial machine allows considerable flexibility in simulation of anticipated field conditions. Those parameters that may be varied include intensity of deviator stress, lateral pressure, load period from 1/10th of a second upward, rest period between cyclic loads on the specimen, sequence of loading and cycles of loading prior to reading test values.

Deviator stresses as low as 1 psi and as high as 64 psi may be applied. Lateral pressures as low as 1/2 psi are not generally applicable as lateral pressure near the surface of the layer should be based on an estimate of the horizontal stresses induced by the load plus the deadload of the overlaying material.

It has been found that 1000 cycles at a specific loading is sufficient to stabilize the resilient modulus for a material and a particular set of loading conditions. 200 cycles will generally be sufficient for granular materials and is frequently adequate for cohesive soils as well.

Standard Test Procedure

The specimen is placed on the triaxial cell base, a membrane applied, the LVDT's clamped in place so that they measure vertical deformation of the middle third of the specimen and a vacuum is applied within the sample and a vacuum chamber to insure that there is no leakage through the membrane. The triaxial cell is then assembled and placed in the triaxial machine. The sample is conditioned by 1,000 cycles of loading at the lowest deviator stress to be applied and at the lateral pressure specified. Measuring equipment is then zeroed after another 200 cycles of loading at the lowest deviator stress. The cyclic load is applied and increased subsequent to test readings at the specified number of cycles for each load level.

The output of the LVDT's is combined for averaging and fed through a signal conditioner to a strip recorder with very rapid response. The recorded cyclic deformation plus the established deviator stress and sample dimensions provide all the information necessary to calculate the resilient modulus at any load level. A resilient modulus is calculated as follows:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \qquad \text{(A-13.1)}$$

where:

M_R = Resilient Modulus

σ_d = Deviator Stress, psi

ε_r = Resilient Strain in in/in

The resilient moduli at the various load levels is then plotted on log-log paper to give clear insight as to the variation in resilient modulus with stress intensity.

Test Results

Test results are summarized in the form of a curve relating resilient modulus to deviator stress level at the specified lateral pressure and loading conditions (See Fig. A-13.1). Additional information and recommendations may also be provided from insight into soil behavior gained during test observations.

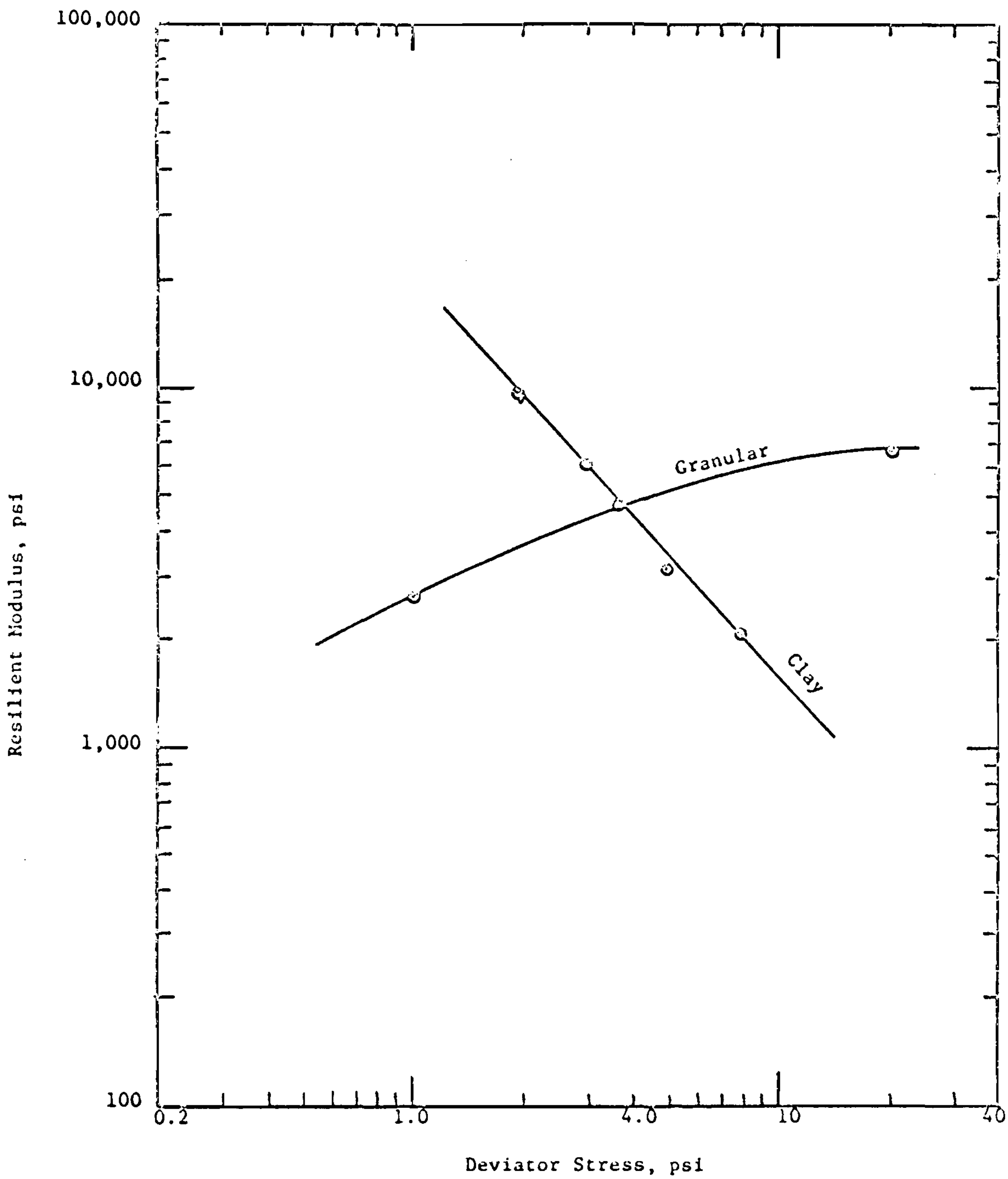


Figure A-13.1. Relation Between Resilient Modulus and Stress for Typical Clay and Granular Soils

APPENDIX 14.

RFLCR1 INPUT GUIDE

APPENDIX 14. RFLCR1 INPUT GUIDE

Data Long Form: (required for first problem of each run).

Card Type 1: Pavement Location Card

1.1 Pavement location (first line) (any combination of letters and/or numbers)

1		80

Card Type 2: Pavement Location Card

2.1 Pavement location (second line) (any combination of letters and/or numbers)

1		80

Card Type 3: Existing Pavement Properties

3.1 Existing pavement type (any combination of letters and/or numbers - right justify) JCP, JRCP, CRCP, etc.

1	2	3	4

3.2 Designation for cracked or uncracked jointed pavement "U" for uncracked or jointed, otherwise cracked pavement is assumed.

5

3.3 Elastic modulus, psi

									•
11	12	13	14	15	16	17	18	19	20

3.4 Thermal coefficient, in./in./°F

•									
21	22	23	24	25	26	27	28	29	30

3.5 Thickness, inches

				•					
31	32	33	34	35	36	37	38	39	40

3.6 Density, pcf

					•				
41	42	43	44	45	46	47	48	49	50

3.7 Joint or crack spacing, feet

					•				
51	52	53	54	55	56	57	58	59	60

3.8 Movement at sliding, inches

		•							
61	62	63	64	65	66	67	68	69	70

Card Type 4: Existing Pavement Reinforcement Properties

(Leave this card blank if pavement is not reinforced.)

4.1 Description
(not required)

S	T	E	E	L					
1	2	3	4	5	6	7	8	9	10

4.2 Elastic modulus of steel, psi
Default value: 29,000,000 psi

									•
11	12	13	14	15	16	17	18	19	20

4.3 Steel thermal coefficient in./in./°F

•									
21	22	23	24	25	26	27	28	29	30

4.4 Area of steel per foot width, in.^{2}

				•					
31	32	33	34	35	36	37	38	39	40

4.5 Perimeter of steel per foot width,
inches

				•					
41	42	43	44	45	46	47	48	49	50

4.6 Steel to concrete bonding stress,
psi

						•			
51	52	53	54	55	56	57	58	59	60

Card Type 5: Horizontal Characterization of Pavement

5.1 Description
(not required)

H	O	R	Z		C	H	A	R	
1	2	3	4	5	6	7	8	9	10

5.2 Mean high temperature, °F

					•				
11	12	13	14	15	16	17	18	19	20

5.3 Joint width at high temperature,
inches

		•							
21	22	23	24	25	26	27	28	29	30

5.4 Mean low temperature, °F

					•				
31	32	33	34	35	36	37	38	39	40

5.5 Joint width at low temperature,
inches

		•							
41	42	43	44	45	46	47	48	49	50

5.6 Minimum temperature observed
since construction of pavement,
°F

					•				
51	52	53	54	55	56	57	58	59	60

Card Type 6: Vertical Characterization of Pavement

6.1 Description
(not required)

V	E	R	T		C	H	A	R	
1	2	3	4	5	6	7	8	9	10

6.2 Mean joint width, inches

		•							
11	12	13	14	15	16	17	18	19	20

6.3 Load transfer from cumulative frequency diagram, percent/100

				•					
21	22	23	24	25	26	27	28	29	30

Card Type 7: Overlay Properties

7.1 Overlay type (right justify) Normally "AC"

1	2	3	4

7.2 Creep modulus, psi

									•
11	12	13	14	15	16	17	18	19	20

7.3 Thermal coefficient, in./in./°F

•									
21	22	23	24	25	26	27	28	29	30

7.4 Thickness, inches

					•				
31	32	33	34	35	36	37	38	39	40

7.5 Density, pcf

							•		
41	42	43	44	45	46	47	48	49	50

7.6 Poisson's ratio (for asphaltic concrete a value of 0.3 is suggested)

		•							
51	52	53	54	55	56	57	58	59	60

7.7 Dynamic modulus, psi

									•
61	62	63	64	65	66	67	68	69	70

7.8 Overlay to existing surface bonding stress, psi

							•		
71	72	73	74	75	76	77	78	79	80

Card Type 8: Overlay Reinforcement Properties

(Leave this card blank if overlay is non-reinforced)

8.1 Overlay reinforcement type (any combination of letters and/ or numbers - right justify)

1	2	3	4	5	6	7	8	9	10

8.2 Elastic modulus, psi

									•
11	12	13	14	15	16	17	18	19	20

8.3 Thermal coefficient

•									
21	22	23	24	25	26	27	28	29	30

8.4 Area of reinforcement per foot width, in^2

					•				
31	32	33	34	35	36	37	38	39	40

8.5 Allowable tensile strain in./in.

	•								
41	42	43	44	45	46	47	48	49	50

Card Type 9: Bondbreaker

(Leave this card blank if no bondbreaker.)

9.1 Description (not required)

B	O	N	D		B	R	E	A	K
1	2	3	4	5	6	7	8	9	10

9.2 Width or length in direction of traffic, feet

					•				
11	12	13	14	15	16	17	18	19	20

Card Type 10: Intermediate Layer Properties

(Leave this card blank if no intermediate layer.)

10.1 Description (not required)

I	N	T	E	R		L	A	Y	R
1	2	3	4	5	6	7	8	9	10

10.2 Creep modulus, psi

									•
11	12	13	14	15	16	17	18	19	20

10.3 Thermal coefficient, in./in./°F

•									
21	22	23	24	25	26	27	28	29	30

10.4 Thickness, inches

					•				
31	32	33	34	35	36	37	38	39	40

10.5 Density, pcf.

						•			
41	42	43	44	45	46	47	48	49	50

10.6 Allowable strain, in./in.

		•							
51	52	53	54	55	56	57	58	59	60

10.7 Dynamic modulus, psi

									•
61	62	63	64	65	66	67	68	69	70

Card Type 11: Design Temperature Changes, Design Load

11.1 Description (not required)

D	E	S	I	G	N				
1	2	3	4	5	6	7	8	9	10

11.2 Design temperature change for existing pavement, °F

					•				
11	12	13	14	15	16	17	18	19	20

11.3 Design temperature change for intermediate layer °F

					•				
21	22	23	24	25	26	27	28	29	30

11.4 Design temperature change for overlay °F

					•				
31	32	33	34	35	36	37	38	39	40

11.5 Design load weight, pounds

									•
41	42	43	44	45	46	47	48	49	50

11.6 Width of design load, inches

						•			
51	52	53	54	55	56	57	58	59	60

<u>Card Type 12: Friction Curve Switch</u>

12.1 Description (not required)

S	L	O	P	E		S	W	C	H
1	2	3	4	5	6	7	8	9	10

12.2 Switch to designate whether slope of friction curve is to be multiplied by ratio of original overburden weight to new overburden weight
If greater than 0.0, slope will be multiplied by this ratio

								•	
11	12	13	14	15	16	17	18	19	20

<u>Card Type 13: New Problem Switch</u>

13.1 Switch to designate type of data following or to terminate run (any combination of letters and/or numbers -- right justify)
"ALL" - read new data using long form
"PART" - read new data using short form
"STOP" - terminate run

1	2	3	4

<u>Data Short Form</u>: (applicable to successive problems where only changes in following inputs are necessary).

<u>Card Type S: Data Change Card</u>

S.1 Overlay creep modulus, psi

									•
1	2	3	4	5	6	7	8	9	10

S.2 Overlay dynamic modulus, psi

									•
11	12	13	14	15	16	17	18	19	20

S.3 Overlay thickness, inches

						•			
21	22	23	24	25	26	27	28	29	30

S.4 Bondbreaker width, feet

					•				
31	32	33	34	35	36	37	38	39	40

S.5 Design temperature change for existing pavement, °F

					•				
41	42	43	44	45	46	47	48	49	50

S.6 Design temperature change for intermediate layer, °F

					•				
51	52	53	54	55	56	57	58	59	60

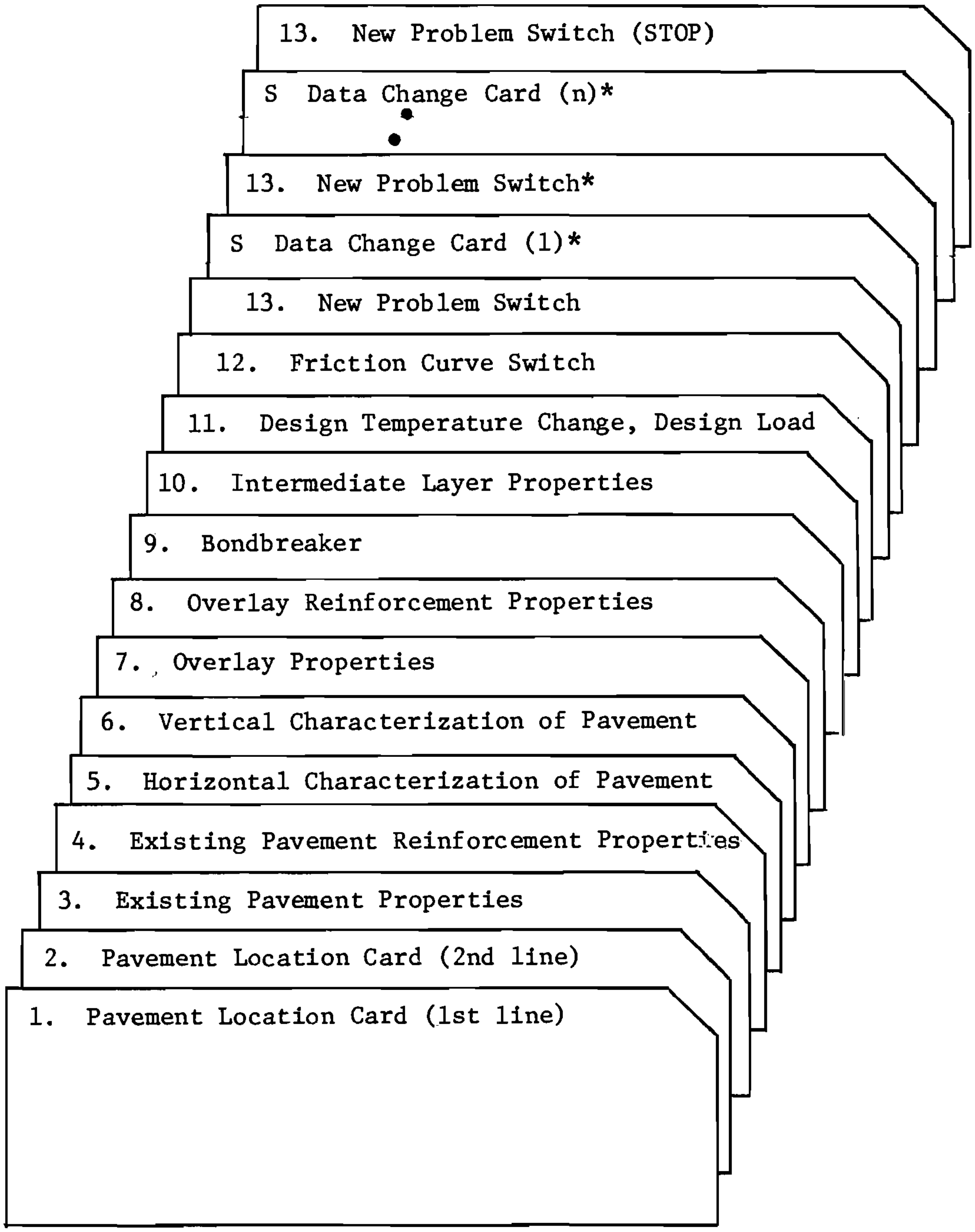
S.7 Design temperature change for overlay, °F

					•				
61	62	63	64	65	66	67	68	69	70

S.8 Switch to designate whether slope of friction curve to be multiplied by ratio of original overburden to new overburden weight (If switch > 0.0 slope will be multiplied by this ratio.)

								•	
71	72	73	74	75	76	77	78	79	80

This card to be followed by a Card Type 13



*If short form is used.

Fig 14A.1. Assembly of RFLCR1 data.

THE AUTHORS

Otto Schnitter received his B.Sc. in Civil Engineering at the University of Pretoria, South Africa, in March 1967. He has had several years experience as a consulting engineer in South Africa and gained wide experience in the field of Civil Engineering and more specific experience in design and construction of pavements. He was closely associated with evaluation and design of highway and airfield pavements and supervised construction of several projects. He joined the Center for Highway Research, The University of Texas at Austin, as a Research Assistant, in the Fall of 1976. He is a member of several professional societies.

W. Ronald Hudson is a Professor of Civil Engineering at The University of Texas at Austin and is Technical Director of a four-year project sponsored by the Brazilian Government, The United Nations Development Program, and the World Bank to study the road development costs in Brazil. He has a wide variety of experience as a research engineer with the State Department of Highways and Public Transportation and the Center for Highway Research at The University of Texas at Austin and was Assistant Chief of the Rigid Pavement Research Branch of the AASHO Road Test. He is the author of numerous publications and was the recipient of the 1967 ASCE J. James R. Croes Medal. He is presently concerned with research in the areas of (1) analysis and design of pavement management systems, (2) measurement of pavement roughness performance, (3) slab analysis and design, and (3) tensile strength of stabilized subbase materials.

B. Frank McCullough is a Professor of Civil Engineering at The University of Texas at Austin. He has strong interests in pavements and pavement design and has developed design methods for continuously reinforced concrete pavements currently used by the State Department of Highways and Public Transportation, U. S. Steel Corporation, and others. He has also developed overlay design methods now being used by the FAA, U. S. Air Force, and FHWA. During nine years with the State Department of Highways and Public Transportation he was active in a variety of research and design activities. He worked for two years with Materials Research and Development, Inc., in Oakland, California, and for the past eight years for The University of Texas at Austin. He participates in many national committees and is the author of over 100 publications that have appeared nationally.