

FLEXIBLE PAVEMENT SYSTEM - SECOND GENERATION,  
INCORPORATING FATIGUE AND STOCHASTIC CONCEPTS

by

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

## PREFACE

This report presents the Flexible Pavement System Second Generation for the design of flexible pavements incorporating fatigue theory, linear elastic layered theory, and stochastic concepts. In terms of elastic and fatigue material properties and their stochastic variations with both space and time, loading, and environmental conditions, new models to predict pavement performance are developed for distress manifestations such as cracking, rut depth, and roughness. The proposed models can be directly used for the design of flexible pavements and can also be included in the Flexible Pavement Systems Computer Program already developed for the Texas Highway Department.

This report is also meant to be a background document for further work to be done to include the effects of temperature and other stresses in the flexible pavement systems model.

This is one of the reports in a series that describe the work done by the Center for Highway Research in the project entitled "The Development of a Feasible Approach to Systematic Pavement Design and Research." The project proposes a long-range comprehensive research program to develop pavement systems analysis and is unusual in that it is a joint effort by three separate research agencies. The project is supported by the Texas Highway Department in cooperation with the Federal Highway Administration Department of Transportation.

The AASHO Road Test data were a very good source of information and were used extensively throughout the analysis in this report for verification of the proposed models. The computer programs were written for the CDC 6600 computer in FORTRAN language.

This report is a product of the continued assistance of many people. The entire staff of the Center for Highway Research at The University of Texas at Austin must be thanked for their cooperation and contributions. Thanks are due to Nancy Braun for her very valuable assistance in the computer programming.

Finally, the support of the Federal Highway Administration and the Texas Highway Department is gratefully acknowledged.

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## LIST OF REPORTS

Report No. 123-1, "A Systems Approach Applied to Pavement Design and Research," by W. Ronald Hudson, B. Frank McCullough, F. H. Scrivner, and James L. Brown, describes a long-range comprehensive research program to develop a pavement systems analysis and presents a working systems model for the design of flexible pavements.

Report No. 123-2, "A Recommended Texas Highway Department Pavement Design System Users Manual," by James L. Brown, Larry J. Buttler, and Hugo E. Orellana, is a manual of instructions to Texas Highway Department personnel for obtaining and processing data for flexible pavement design system.

Report No. 123-3, "Characterization of the Swelling Clay Parameter Used in the Pavement Design System," by Arthur W. Witt, III, and B. Frank McCullough, describes the results of a study of the swelling clay parameter used in pavement design system.

Report No. 123-4, "Developing A Pavement Feedback Data System," by R. C. G. Haas, describes the initial planning and development of a pavement feedback data system.

Report No. 123-5, "A Systems Analysis of Rigid Pavement Design," by Ramesh K. Kher, W. R. Hudson, and B. F. McCullough, describes the development of a working systems model for the design of rigid pavements.

Report No. 123-6, "Calculation of the Elastic Moduli of a Two Layer Pavement System from Measured Surface Deflections," by F. H. Scrivner, C. H. Michalak, and W. M. Moore, describes a computer program which will serve as a subsystem of a future Flexible Pavement System founded on linear elastic theory.

Report No. 123-7, "Annual Report on Important 1970-71 Pavement Research Needs," by B. Frank McCullough, James L. Brown, W. Ronald Hudson, and F. H. Scrivner, describes a list of priority research items based on findings from use of the pavement design system.

Report No. 123-8, "A Sensitivity Analysis of Flexible Pavement System FPS2," by Ramesh K. Kher, B. Frank McCullough, and W. Ronald Hudson, describes the overall importance of this system, the relative importance of the variables of the system and recommendations for efficient use of the computer program.

Report No. 123-9, "Skid Resistance Considerations in the Flexible Pavement Design System," by David C. Steitle and B. Frank McCullough, describes skid resistance consideration in the Flexible Pavement System based on the testing of aggregates in the laboratory to predict field performance and presents a nomograph for the field engineer to use to eliminate aggregates which would not provide adequate skid resistance performance.

Report No. 123-10, "Flexible Pavement System - Second Generation, Incorporating Fatigue and Stochastic Concepts," by Surendra Prakash Jain, B. Frank McCullough, and W. Ronald Hudson, describes the development of new structural design models for the design of flexible pavement which will replace the empirical relationship used at present in flexible pavement systems to simulate the transformation between the input variables and performance of a pavement.

## ABSTRACT

Design of flexible pavement is a complex procedure involving numerous variables. The systems approach can be considered as the best method for solving design problems. An important part of any pavement design system involves upgrading it in order to include the best possible technology. One of the distress mechanisms included in the conceptual flexible pavement design system, as a part of the fracture failure mode, is fatigue of the pavement materials. Its consideration on some rational basis and stochastic variations of the material properties in space and time need particular attention in the development of a working systems model.

New structural design models, for the second generation of the flexible pavement system, based on linear elastic layered theory, fatigue theory, and probability theory, are presented. Probability theory is used for variation in material properties and fatigue life and for calculation of the cracking index, based on probability of damage. The new design models are proposed to replace the empirical relationship used at present to simulate the transformation between the input variables and performance of a pavement. The serviceability and performance concepts from the AASHO Road Test have also been utilized. The fatigue phenomenon is considered and the inputs of the system are correlated in terms of elastic and fatigue material properties and their stochastic variations, loading, environmental conditions, and compaction characteristics under repeated loading to the distress manifestations, such as cracking and rut depth. Based on AASHO Road Test data, a correlation between cracking and slope variance was developed. Thus, models are developed for the cracking index, rut depth index, and roughness index to predict the pavement performance and present serviceability index. Computer programs have been developed for these models to aid in the various stages of the design.

The models have been verified by comparing predicted performance with that observed at the AASHO Road Test for 28 sections. The models compare very well and predict the observed data within the acceptable accuracy. Results of the sensitivity analysis for the cracking index model are included. It is

seen that the fatigue parameter  $B$  is the most significant and very sensitive variable and should be estimated very accurately.

Example problems are shown to compare the proposed models with the existing FPS models. The proposed design method shows various improvements and gives more realistic flexible pavement designs. A new rational procedure for an overlay design using damage theory is explained and is based on sound theoretical fundamentals. This is followed by a chapter on implementation of the proposed models for the flexible pavement system second generation. It is noted that the stress and strain calculations in the present analysis, by the direct use of the layered program, should be improved and replaced by a more efficient approach.

Conclusions have been based on the overall experience gained while working on this project. It is noted that only a few bonafide design procedures for flexible pavements exist at present and those in practice need improvements. The use of the proposed design procedure based on the theories discussed earlier give a new dimension to the flexible pavement design field. The characterization of material properties is a very important part of the whole design process and requires proper attention.

Recommendations have been made to aid planning of future work. The proposed design models are based on sound fundamentals, using the best state-of-the-art information, and are recommended for the design of flexible pavements and to be included in the pavement systems design computer programs already developed for the Texas Highway Department.

## SUMMARY

New structural design models for the design of flexible pavement have been developed which will replace the empirical relationship used at present in flexible pavement systems to simulate the transformation between the input variables and performance of a pavement. Computer programs have been developed to quantify the distress manifestations, cracking, roughness, and rut depth in a pavement which are used to predict its performance. The models have been verified by comparing predicted performance with that observed at the AASHO Road Test. The overlay design procedure is improved and takes account of the damage to the existing pavement system.

The proposed method can be directly used for the design of flexible pavements and can also be included in the pavement system design computer program already developed for the Texas Highway Department for updating the system.

The development has the advantage of an immediate direct application and gives the background for further improvements in the existing design system.

## IMPLEMENTATION STATEMENT

A separate chapter is included in this report discussing the details of implementation. The proposed structural design models can be used directly for the design of flexible pavements and can also be included in the existing FPS computer program.

The proposed method eliminates the present practice of expensive field measurements of material properties. The use of elastic constants, which are measured in the laboratory, can be more economical, convenient, and accurate. The laboratory measurements of elastic constants, tensile characteristics, and fatigue properties of pavement materials, are already in progress under a project at the Center for Highway Research at The University of Texas. Moreover, a computer program to calculate the elastic moduli of a two-layer system from measured surface deflection is already available and further work to complete the in-situ values of elastic moduli is in progress at Texas Transportation Institute, Texas A&M University. The proposed method has the new capability of predicting the nature of distress, i.e., cracking, roughness, and rutting, which cannot be done by any existing methods.

The proposed models can evaluate the effects of compaction, fatigue, and stochastic variations in material properties. The proposed models could also be used to give better evaluation of some of current black bases being proposed for pavements by the Texas Highway Department. The Flexible Pavement System FPS is already in use by several districts of the Texas Highway Department; hence, only a revised version of FPS, incorporating the proposed models, needs to be formulated. Thus, there is an excellent scope of the implementation of the proposed models in the near future without much efforts and organizational changes.

TABLE OF CONTENTS

PREFACE . . . . . iii  
LIST OF REPORTS . . . . . v  
ABSTRACT . . . . . vii  
SUMMARY . . . . . ix  
IMPLEMENTATION STATEMENT . . . . . x

PART I. BACKGROUND

CHAPTER 1. INTRODUCTION

Background . . . . . 1  
Objective . . . . . 3  
Scope . . . . . 3

CHAPTER 2. REVIEW OF EXISTING THEORIES AND FLEXIBLE PAVEMENT DESIGN PROCEDURES

Existing Theories and Pavement Design Procedures . . . . . 5  
Existing Flexible Pavement Systems Models . . . . . 7

CHAPTER 3. DEVELOPMENT OF SECOND GENERATION FLEXIBLE PAVEMENT SYSTEMS

Introduction . . . . . 14  
Evaluation of the Existing Models . . . . . 14  
Proposed Revision of FPS . . . . . 18  
Scope of the Present Report . . . . . 21  
Basic Work Plan . . . . . 24

PART II. REVIEW OF AVAILABLE TECHNIQUES TO BE USED IN THE MODELS

CHAPTER 4. FATIGUE OF PAVEMENT MATERIALS

Introduction to Fatigue . . . . . 26  
Asphaltic Concrete . . . . . 26  
Untreated Granular and Fine Grained Materials . . . . . 36  
Summary . . . . . 41

## CHAPTER 5. CHARACTERIZATION OF FLEXIBLE PAVEMENT MATERIALS

Introduction . . . . .	42
Asphaltic Concrete . . . . .	44
Base and Subbase Granular Materials . . . . .	47
Characterization of Subgrade "Fine Grained Cohesive Soils" . . . . .	56
Summary . . . . .	58

## CHAPTER 6. USE OF ELASTIC THEORY AND LAYERED ANALYSIS IN THE DESIGN OF FLEXIBLE PAVEMENTS

Introduction . . . . .	61
Behavior of Ideal Materials . . . . .	62
Comparison of Predicted and Observed Behavior . . . . .	64
Summary . . . . .	65

## PART III. DEVELOPMENT OF MODELS

## CHAPTER 7. DEVELOPMENT OF DISTRESS MODELS

Ideal Distress Index Model . . . . .	67
Development of Distress Index Model . . . . .	69
Quantification of Distress Index Model . . . . .	71
Verification of Distress Index Models . . . . .	72

## CHAPTER 8. DEVELOPMENT OF CRACKING INDEX MODEL

Stochastic Concepts Applied to Cracking Index in Flexible Pavements . . . . .	75
Quantification of Cracking Index . . . . .	79
Procedure for Modeling the Cracking Index . . . . .	84
Computer Program . . . . .	87

## CHAPTER 9. DEVELOPMENT OF RUT DEPTH INDEX MODEL

Quantification of Rut Depth Index Model . . . . .	88
Procedure to Compute the Rut Depth . . . . .	96
Computer Program . . . . .	96

## CHAPTER 10. DEVELOPMENT OF ROUGHNESS INDEX MODEL

Theory . . . . .	98
Quantification of Roughness Index . . . . .	101
Selection of Model . . . . .	109
Procedure for Computation of Roughness Index . . . . .	110
Computer Program . . . . .	110



## CHAPTER 11. VERIFICATION OF DISTRESS MODELS

Cracking Index Model . . . . .	113
Rut Depth Index Model . . . . .	120
Verification of Roughness Index Model . . . . .	124
Verification of the Performance Model . . . . .	129

## PART IV. VERIFICATION AND PROPOSED USE OF MODELS

## CHAPTER 12. SENSITIVITY ANALYSIS

Results . . . . .	144
Observations . . . . .	152
Summary . . . . .	156

## CHAPTER 13. SUMMARY OF PROPOSED FATIGUE MODEL

Proposed Fatigue Model . . . . .	157
Proposed FPS Second Generation . . . . .	160
Example Problem - Comparison of the Present FPS and Proposed Fatigue Models . . . . .	163

## CHAPTER 14. IMPLEMENTATION

Stress and Strain Computations . . . . .	171
Overlay Design . . . . .	172
Repeated Load-Deformation Data . . . . .	172
Equivalencies . . . . .	174
Present Serviceability Index . . . . .	174
Traffic Computations . . . . .	174
Time Subroutine . . . . .	175
Laboratory Investigations . . . . .	175
Limitations for Surface Treatment and Thin Asphaltic Concrete Surfaces . . . . .	175
Prediction Errors in the Models . . . . .	176

## PART V. CLOSURE

## CHAPTER 15. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary . . . . .	178
Conclusions . . . . .	179
Recommendations . . . . .	180

REFERENCES . . . . .	182
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## APPENDICES

Appendix 1.	Summary of Research Needs, Advisory Committee, HRB Workshop on Structural Design of Asphalt Concrete Pavement Systems Held in Austin, Texas, December 7-10, 1970 (Ref 63) . . . . .	200
Appendix 2.	Details of Material Characterization . . . . .	206
Appendix 3.	Outline of Computer Program Available for Analysis of Stresses, Strains, and Displacements in a Five-Layered Elastic System Under a Load Uniformly Distributed on a Circular Area . . . . .	242
Appendix 4.	Computer Program and Analysis	
Appendix 4.1.	Flow Chart . . . . .	248
Appendix 4.2.	Listing of Computer Program for Cracking Index and Rut Depth Index . . . . .	255
Appendix 4.3.	Guide for Data Input . . . . .	272
Appendix 4.4.	Input Data Sample . . . . .	277
Appendix 4.5.	Cracking Index and Rut Depth Index Example Problem . . . . .	279
Appendix 4.6.	Regression Analysis for Cracking-Patching Versus Roughness Index . . . . .	286
Appendix 4.7.	Computer Program and Calculated Values of Roughness Index and Present Serviceability Index . . . . .	298
Appendix 5.	Flexible Pavement Performance Record . . . . .	302
Appendix 6.	Nomenclature . . . . .	305
Appendix 7.	AASHO Road Test and Present Serviceability Concept . . . . .	309
Appendix 8.	Comparison of Distress Models - Plots . . . . .	314
Appendix 9.	Computer Input and Output for Example Problems . . . . .	335
THE AUTHORS	. . . . .	343

PART I

BACKGROUND

## CHAPTER 1. INTRODUCTION

### BACKGROUND

The design of flexible pavements requires knowledge of complex structural systems. Many variables are involved, including the behavior of soils and paving materials, combinations of static and dynamic loading, and different environmental and climatic conditions. Early design procedures for flexible pavements were primarily rule-of-thumb. In time, many empirical and semi-empirical methods of design were developed. The empirical nature of the methods is due in part to limited knowledge of the behavior of materials and of actual failure mechanisms and in part to the limitations of analytical techniques in handling the complex mathematical functions required.

The inability to predict pavement performance under certain conditions with any existing design method has been due to the manner in which design procedures were developed; a particular development was applicable only within certain limited geographic boundaries and suitable only for the characteristics of available materials, environmental conditions, and traffic loads within these boundaries.

Therefore, a more rational method of pavement design was needed, one which could predict the performance of a pavement under various sets of conditions. Such a method may be organized within the framework of the "systems approach" and must consider various variables, including physical, social, and economic. A project which proposed a long-range comprehensive research program to develop a pavement system analysis, "The Development of a Feasible Approach to Systematic Pavement Design and Research," was initiated in December 1968 by the Texas Highway Department, the Center for Highway Research at The University of Texas at Austin, and the Texas Transportation Institute at Texas A&M University, under the Cooperative Highway Research Program. Under this project, now entitled "Systematic Pavement Design," a computer program based on the systems approach and known as the Flexible Pavement Systems or FPS was developed for the design of flexible pavements (Ref 81). A general description of FPS and its development is given in Chapter 2. The basic models used

in FPS were obtained from Research Report 32-11 (Ref 162), which was the outgrowth of an attempt to apply the AASHO Road Test results to Texas conditions. More than 50 physical inputs and constraints are used in the FPS models and the output is a set of recommended pavement design strategies based on the present net worth of the lowest total cost. Total cost consists of initial construction, maintenance, overlays, users, seal coat, and salvage costs. The approach gives the designer considerably expanded scope and flexibility in exploring design options.

The performance subsystem, which is only a part of the whole flexible pavement systems model, uses the empirical relationship between the input variables and the pavement performance. A performance history is obtained from the prediction of present serviceability index (Ref 158), and failure of the system is evaluated in terms of minimum serviceability level and the total cost of the system. However, the present serviceability index is not obtained from the actual distress manifestations, i.e., magnitude of cracking, patching, roughness, and rut depth, but simply from some function of their combined values. This function, which was statistically derived from the AASHO Road Test data, is assumed to represent the present serviceability index at any time. The use of these empirical relationships, for materials not used at the AASHO Road Test, different environmental conditions, locations outside the limited boundaries, and with passage of time resulting in revision of the standards of safety and comfort, is questionable. In spite of all the technological developments and the theoretical background available in the present state-of-the-art for the design of flexible pavements, no existing design procedure, including FPS, can predict quantitatively the distress manifestations, such as cracking, rut depth, and roughness, which will appear in a pavement during its performance period. The distress mechanisms which are considered in the systems design approach for flexible pavements include, as a part of the fracture failure mode, fatigue of pavement materials. Fatigue plays an important role in the design of a pavement structure and its complete consideration on some rational basis is particularly important in the development of a working system model. Stochastic variations of material properties with space and time also need to be taken into account in a realistic design approach. Proper application and use of elastic layered theories need investigation. The problem of computation of permanent deformation should be analyzed. No rational overlay design procedure which is operational considers

the actual damaged and consolidated condition of the pavement at the time of an overlay.

#### OBJECTIVE

The general objective of this study is to upgrade the existing flexible pavement systems by attacking the problems of computing fatigue cracking, permanent deformation, and roughness and developing new structural design models. As discussed in the previous section, no existing pavement design method can predict, or attempts to, the condition of failure in a pavement at the end of the design period. In simple terms the main objective of the proposed developments is to quantify the distress manifestations in a pavement system in order to predict its performance and failure conditions. Inclusion of these new models in the performance subsystem of the existing flexible pavement systems, with necessary revision in the physical models (structural design models), will assist in the development of a second generation flexible pavement systems design model.

#### SCOPE

The approach described herein utilizes a theory of linearly elastic layers which is commonly termed "layered theory." It takes into account the fatigue behavior of the materials and their stochastic variations with space and time. The probability concept in the output of the system is considered in the analysis. The serviceability-performance concept of the AASHO Road Test has also been used. With the fatigue phenomenon considered, the inputs of the system are correlated to its distress manifestations, such as cracking and rut depth. Based on AASHO Road Test data, a correlation between cracking and the roughness index of the pavement is developed. Thus models for the cracking index, roughness index, and rut depth index are developed to predict the pavement performance and present serviceability index. The models are verified with AASHO Road Test data and example problems which predict the performance within the acceptable accuracy. These new models can be used directly for the design of flexible pavement and can also be included in the design computer programs for flexible pavement systems already developed for the Texas Highway Department.

This report is divided into five parts, each consisting of several chapters. Part I, the first three chapters, covers background material. Chapter 2 reviews existing theories and methods of flexible pavement design along with their limitations and contains a description of the flexible pavement system. Chapter 3 briefly gives background data on development of the proposed design procedure.

Part II, Chapters 4 through 6, reviews techniques used in development of the models proposed for the design of flexible pavements. Chapter 4 summarizes the concept of fatigue and its application to the design of flexible pavements. Chapter 5 contains a discussion on the characterization of materials and stochastic variations; the procedure for characterizing material properties, including the AASHO Road Test materials, is explained in detail. Chapter 6 explains the use of elastic theory and layered analysis in the design of flexible pavements.

Part III consists of Chapters 7 through 10, which describe the development of distress models for quantification of total distress index, cracking index, rut depth index, and roughness index, respectively.

Part IV, Chapters 11 through 14, is devoted to verifying the developed models with the AASHO Road Test data and describes the use of the proposed procedure. Chapter 11 contains the verification of the distress models developed in Chapters 7 through 10. Chapter 12 summarizes the results of a small sensitivity analysis of the parameters in the cracking index model and establishes a format for a proposed detailed sensitivity study. Chapter 13 summarizes the proposed fatigue models, contains example problems, and compares the present FPS with the proposed fatigue procedure. This chapter also describes the revision of the present FPS model. Chapter 14 is devoted to implementation.

Part V, Chapter 15, is the summary, conclusions, and recommendations.

## CHAPTER 2. REVIEW OF EXISTING THEORIES AND FLEXIBLE PAVEMENT DESIGN PROCEDURES

### EXISTING THEORIES AND PAVEMENT DESIGN PROCEDURES

Flexible pavement design procedures in the beginning were primarily "rules-of-thumb," i.e., procedures based on past experience. During the period between the first and second World Wars, engineers made concerted efforts to evaluate pavement performance and plate theory, and some rational methods for the design of rigid pavements were developed. Efforts to evaluate the structural properties of subgrade soil and to classify soils for use in correlating pavement performance with subgrade type also continued. The limitations to obtaining successful and satisfactory results were partly due to the limited knowledge of the behavior of materials and appropriate failure mechanisms and in part to the limited analytical solution techniques available for the complex functions required. Application of Boussinesq's theory of stresses in ideal masses was developed in 1883, but it was not until 1943 that Burmister first put forward his layered theory for two layers and conceptually presented the solution for three-layered system, giving some rational basis for the design of flexible pavements (Ref 14).

With the advent of World War II, the U. S. Army Corps of Engineers initiated a study of design methods that eventually led to the CBR design method. Following World War II, many state highway departments also started studies to develop pavement design procedures. Many independent design procedures were developed, based on various soil tests that were correlated with pavement performance, environmental considerations, experience, and theories, and at the present time numerous design procedures are in use.

Over the years, several road tests (Table 2.1) have provided a wealth of experimental data and observations. The AASHO Road Test, which cost about \$30 million, was one of the most successful. One of the major objectives of the AASHO Road Test was to provide information which would be used in developing pavement design criteria and design procedures.



TABLE 2.1. IMPORTANT ROAD TESTS

Road Test Name	Year	Agency	Pavement Type
Arlington Test (Virginia)	1919	BPR	flexible and rigid
Bates Road Test (Illinois)	1920-23	BPR	flexible and rigid
Pittsburgh Road Test (California)	1930-40	Columbia Steel Co.	rigid
Stockton Road Track (California)	1930-40	Corps of Engineers	flexible
Hybla Valley (Alexandria, Virginia)	1944-54	HRB, AI, BPR	flexible
Maryland Road Test (Maryland)	1950-51	AASHO	rigid
WASHO Road Test (Idaho)	1952-53	WASHO	flexible
AASHO Road Test (Illinois)	1958-61	AASHO	flexible and rigid

BPR - Bureau of Public Roads

HRB - Highway Research Board

AI - Asphalt Institute

AASHO - American Association of State Highway Officials

WASHO - Western Association of State Highway Officials

The Committee on Theory of Pavement Design of the Highway Research Board recently prepared a review of existing theories and methods of pavement design (Ref 183). In their report, the design procedures are grouped under the following headings:

- (1) elasticity methods,
- (2) ultimate strength methods,
- (3) semi-empirical and statistical methods, and
- (4) empirical and environmental methods.

The elasticity methods are based on the criterion of limited stresses or strains as determined by calculations based on the theory of elasticity for certain values established empirically as safe. The ultimate strength method assumes that a pavement possesses an adequate safety factor against an assumed shear failure of the pavement system. The semi-empirical and statistical methods are based on studies of observed field behavior, followed by statistical analysis of data to correlate performance and other design factors involved. In the empirical and environmental methods, the pavement is designed based on soil classifications and environmental conditions. It can be seen that the bases for these four methods are quite limited in scope, and none of the methods can predict the actual distress manifestations during and at the end of the design life.

Methods based on systems approach, which can be considered the latest and best available, are discussed separately in more detail for the following reasons:

- (1) to give background information for development of the new models developed in this dissertation;
- (2) to show the lack of a rational basis and the improvements needed; and
- (3) because the new design models developed in this dissertation, when included in the existing flexible pavement systems (FPS), will lead to the flexible pavement system - second generation.

#### EXISTING FLEXIBLE PAVEMENT SYSTEMS MODELS

It is practically impossible to describe completely pavement behavior with a single equation or model. To define this behavior and solve the problem of pavement design, a systems approach is required. It is a framework within which the multitude of physical and socio-economic variables involved can be sorted out and related in a meaningful way. For this study, systems approach is

defined as a systematic way of approaching, modeling, and solving a problem, utilizing available manpower, money, material, and time in the best possible way.

A 1967 NCHRP project led to the first work applying systems engineering to pavement design (Ref 78). In a similar but independent effort, Hutchinson and Haas (Ref 82) applied the systems approach to structuring the overall problem and several of the subsystems design problems. A phase development and description of the pavement systems is fully presented in Ref 81. The systems approach was recognized as the most logical by a large number of pavement design engineers at the Highway Research Board Workshop on Structural Design of Asphaltic Concrete Pavements at Austin, Texas, December 1970.

#### Development of Existing Flexible Pavement System

Two systems models for the design of flexible pavements, one based on deflection and the other on structural number, have recently been developed for the Texas Highway Department under the Cooperative Research Program (Ref 81). The primary purpose of the existing flexible pavement systems method was to provide the designer with a means for investigating a large variety of pavement design options in a systematic and efficient manner. It was not intended to replace a designer's decision-making prerogative, but rather to give him increased scope and flexibility (Ref 81).

The mathematical models developed for FPS are based on the established objective of providing from available materials a pavement capable of being maintained above a specific level of serviceability over a specified period of time, at a minimum overall cost. The computer program was written to provide an output of feasible pavement designs sorted by increasing total cost, to help the designer or decision-maker to make his choice as quickly and easily as possible (Ref 81).

#### Inputs and General Description of FPS (Ref 81)

Each of the two FPS models consists of a set of mathematical models that may be broken down into four types: (1) physical, (2) economic, (3) optimization, and (4) interaction.

A large number of input variables are considered in FPS to simulate the total pavement design approach as closely as possible.

Physical Models. These are simulations of the real-world performance of a pavement during the analysis period. Traffic models predict the traffic during the analysis and performance periods; environmental models take into account environmental conditions, considering temperature, regional factor, and swelling clay parameters; performance models predict the behavior of the pavement, based on the present serviceability index (PSI) concept developed at the AASHO Road Test, and include a pavement strength model based on either (1) surface curvature index (SCI) or deflection model (Ref 162), or (2) structural number and soil support models from the AASHO Road Test.

In the deflection model, the material in each layer is characterized by a stiffness coefficient which is entirely different from the structural number coefficients in the AASHO Interim Guides. The stiffness coefficient values for different materials are based on field measurements of pavement deflection.

The structural number model is based on the structural number and soil support parameters developed at the AASHO Road Test. Using the information from the AASHO Road Test, the AASHO Committee on Pavement Design developed a design method that was issued in the form of Interim Design Guides (Refs 64 and 65) in 1962. It was emphasized that the design guides were interim in nature and subject to adjustment based on experience and additional research. It was noted that careful consideration was required to assign strength coefficient values to materials not used at the Road Test. The design equations were derived for

- (1) a specific set of paving materials,
- (2) a single environment,
- (3) an accelerated traffic period (two years compared to a normal design period of 20 years), and
- (4) identical traffic (mixed traffic was not applied).

Though the Interim Guides approach is sound in that it recognizes the importance of soil support, traffic load applications, and climatic conditions, however, the problem is to quantify the effects of all these factors on some rational basis. In spite of large variability, certain weighted average values have been considered as constants and are used as the coefficients of relative strength in the pavement design procedure of the Interim Guide. The term "coefficients of relative strength" is misleading as these are essentially the regression coefficients in the structural number of thickness index equations

and supposedly represent some material characteristics. The values of 0.44, 0.14, and 0.11 represent weighted averages of coefficients of relative strength  $a_1$ ,  $a_2$ , and  $a_3$  determined from an analysis of performance and design (Ref 70). Actually these coefficients in the analysis varied from 0.83 to 0.33, 0.25 to 0.11, and 0.11 to 0.09, respectively. It is difficult, therefore, to consider the design performance relationships of AASHO as exact. To establish coefficients of relative strength for any other material as constants is also very difficult. Though it was appreciated and pointed out that these coefficients were related to the physical properties of the materials, no definite formulation was offered for the correct properties. Different agencies have made efforts to predict the correct values of these coefficients through correlations with CBR, cohesiometer values, and Marshal stability, but these correlations are also empirical.

In NCHRP Project 1-11 (Ref 117), a method was developed for selecting the structural coefficients based on layered elastic theory. Vertical compressive strain on the subgrade, surface deflection, and tensile strain of the asphaltic concrete were selected as the criteria to establish structural layer equivalency. It was shown that the equivalencies can vary according to various geometric environment and loading conditions and that several assumptions were required to account for these conditions. Charts were developed in terms of selected material properties, but these are also only approximate.

Even such a major effort as the AASHO Road Test could produce only an interim design guide, subject to adjustment based on experience and additional research.

NCHRP Project 1-11 (Ref 117) was conceived to evaluate the various techniques used and the results obtained by the individual states after applying the guides to pavement structure design. This information was collected from the various states and the results were summarized (Table 1 of Ref 117). The importance of the AASHO Interim Guides is apparent from its use by about 32 states. They are being widely used, partly because of the unavailability of any other, better, and more rational design procedure.

Economic Models. Economic models are used to determine the total cost of a design as well as a breakdown of the cost. All costs are converted to present value at appropriate interest rates which are supplied by the user. The present value represents the amount of money which would, if invested at the

present time, generate funds to accomplish the design scheme as specified. There are seven types of economic models used in FPS.

- (1) initial construction cost model, which determines the cost of the initial construction based on the cost per compacted cubic yard of each material used;
- (2) seal coat cost model, which calculates the present cost of the seal coats required during the performance period;
- (3) overlay construction cost model, which together with a physical model determines when and how much to overlay as well as the cost;
- (4) routine maintenance model, which predicts the cost of routine maintenance based on the optimum overlay and seal coat schedules;
- (5) user's cost model, which determines the cost to the user due to traffic delays during overlay construction;
- (6) salvage value model, which determines the value of the pavement remaining at the end of the analysis period; and
- (7) total overall cost model, which relates all costs during the analysis period to their present value at the beginning of the period.

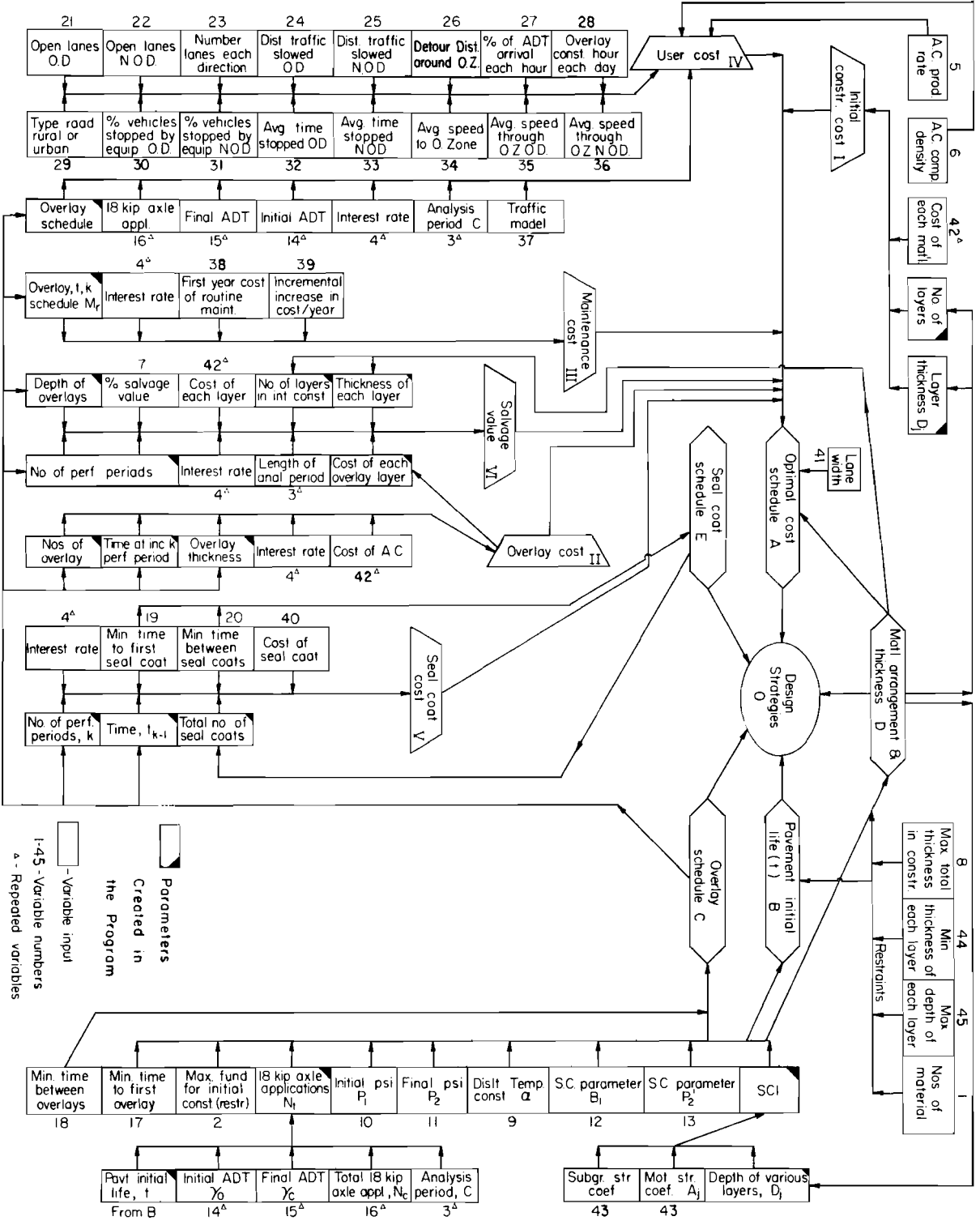
Optimization Models. The two optimization models used in FPS to determine a set of optimal designs, based on overall cost, are

- (1) modified branch and bound technique, which systematically determines which initial construction designs will lead to a set of optimal designs.
- (2) determination of the optimal overlay policy for each initial design, considering all possible policies.

Interaction Models. An interaction model is an algorithm which defines the interactions between two or more other models. For example, in finding the life of initial and overlay construction designs, a time must be determined which will satisfy both performance and traffic models. Because of the complexity of these models, it is necessary to use an iterative technique.

Design Flow Chart of FPS. A design flow chart for the deflection version, FPS2, is shown in Fig 2.1. The flow chart for the structural number version of FPS is similar except for a few changes in the list of parameters. This chart shows all parameters involved in the various models of FPS2. The design strategies consist of schedules giving optimal cost, pavement life, overlays, material arrangement and thickness, and seal coat. Each schedule is calculated by consideration of the various parameters, shown in boxes. From the flow

FIG. 2-1 FPS-2 DESIGN FLOW CHART



1-45 - Variable input  
 a - Repeated variables  
 Parameters Created in the Program

chart, it can be seen that the program involves a large number of variables (the number of inputs into the program is  $\overline{6n + 44}$ , where  $n$  is the number of materials considered for use above the foundation) which are intercorrelated in a complex optimization technique.



## CHAPTER 3. DEVELOPMENT OF SECOND GENERATION FLEXIBLE PAVEMENT SYSTEMS

### INTRODUCTION

The review of existing theories and various design procedures for flexible pavements in Chapter 2 showed the diversity and lack of rational basis for some present design procedures and the need for development of an improved design procedure.

The two existing flexible pavement systems models were also discussed in Chapter 2. A detailed evaluation of the FPS models in this chapter will show the need for updating and improving these models. Improved and updated structural design models, based on proper fatigue and stochastic considerations, are developed later in this report. These structural design models technically would fit into both existing FPS computer programs.

A basic work plan outlined in this chapter for the fatigue subsystem establishes the format of work plans for other areas, such as temperature stresses and stochastic variations in input design variables, which will be included in the existing FPS models at the appropriate stages.

### EVALUATION OF THE EXISTING MODELS

Performance of a pavement is a measure of the accumulated service it provides and a function of the present serviceability history of the pavement, according to the AASHO concept of present serviceability index (PSI). The distress mechanism is the response which can lead to some form of distress when carried to a limit. Figure 3.1 shows the categories and examples of distress mechanisms in the pavement system.

Literature review shows that the best means presently available to account for all the distress modes in a pavement in the three categories shown in Fig 3.1 is the present serviceability index equation developed at the AASHO Road Test (Ref 70). The roughness in the AASHO Road Test (Eq A7.1) is a function of distortion and disintegration modes. The cracking and patching terms are related to all three distress modes, and rut depth is a function of distortion mode only (Ref 78).

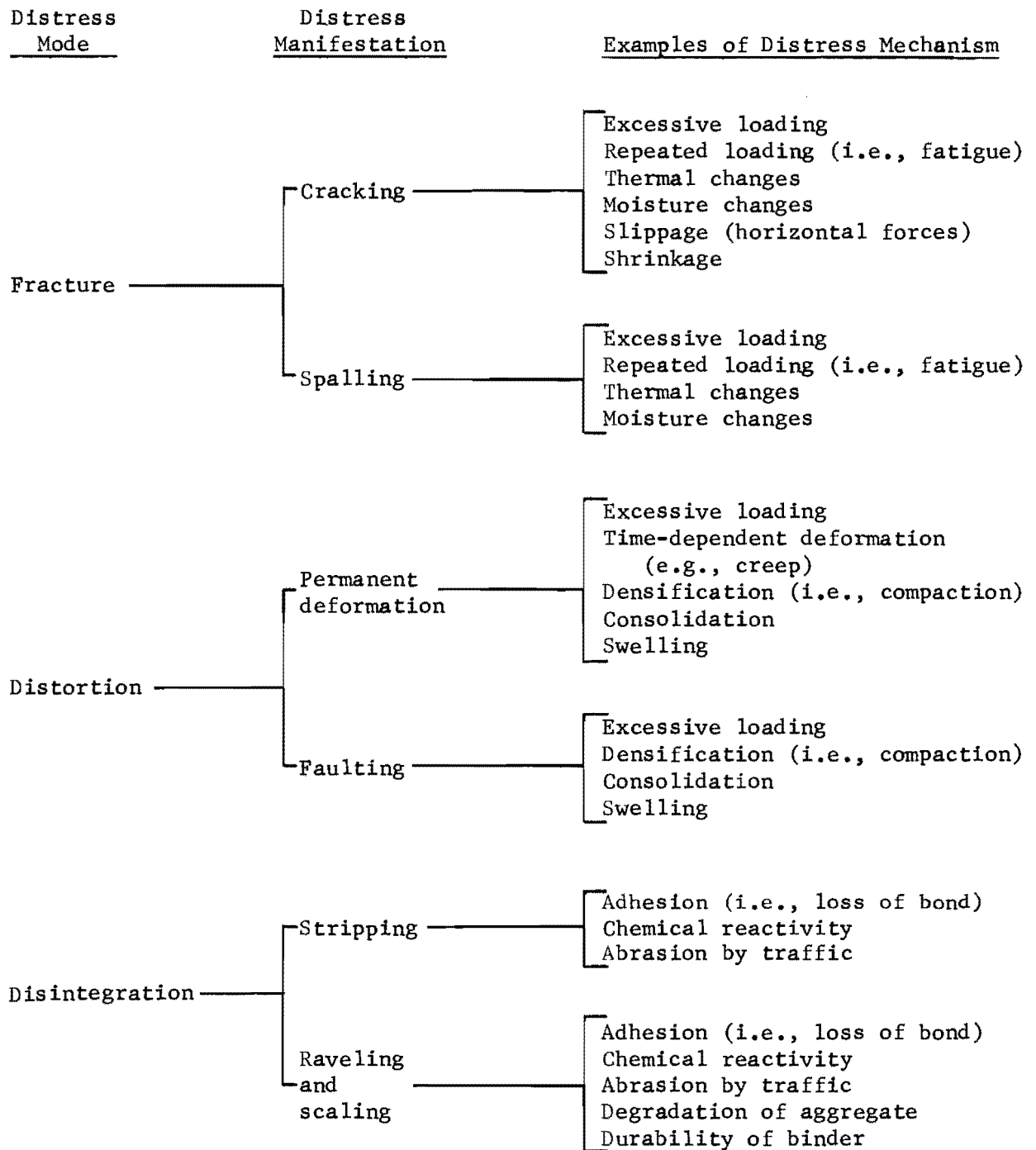


Fig 3.1. Categories of pavement distress (After Ref 78).

The FPS models (Ref 81) utilize the AASHO concept of pavement performance and are based on field results from AASHO Road Test and Texas Transportation Institute (Ref 158) test sections. The FPS also incorporated many variables to select a best and most economical design. Therefore, the FPS models represented the latest and best available design procedures. Though an effort was made to include as many factors from Fig 3.2 as the present state-of-the-art would permit, many factors still required improvements and considerations, as discussed below.

One of the distress mechanisms included in the systems approach for pavement design as a part of the failure mode is the fatigue of pavement materials (Fig 3.2). Fatigue plays a very important role in the design of a pavement structure and it should receive particular attention in the development of a working systems design model. This important mode of failure has not been given complete consideration in FPS, although the number of repetitions of axle load in FPS considers some kind of fatigue mode. The number of repetitions  $N$ , however, are related to PSI only empirically without any theoretical basis and without consideration of actual fatigue behavior of materials under repeated stress and strain. Fatigue theory, as it applies to the new design procedure, is discussed in detail in Chapter 4.

In the deflection model of the FPS, the materials in each layer are characterized by a stiffness coefficient, but no way has been found for defining or predicting the values of these coefficients from laboratory tests with suitable accuracy. These must be estimated from deflection measurements made on the same type of material on an existing pavement located in the same general area as the planned facility. The accuracy of the prediction of these coefficients by this method for the other materials is doubtful. In the AASHO model, the values of strength coefficients are empirical and, as discussed in Chapter 2, cannot be determined accurately by any available test method.

In the present FPS, the history of change in material properties during the lifetime of the pavement is not taken into account. At the time of an overlay, the material thicknesses and their original strength coefficients are assumed.

Structural number SN or surface curvature index SCI (Ref 81) are directly related to present serviceability index PSI without consideration of the stresses and distress in individual layers. Sections with the same SN or

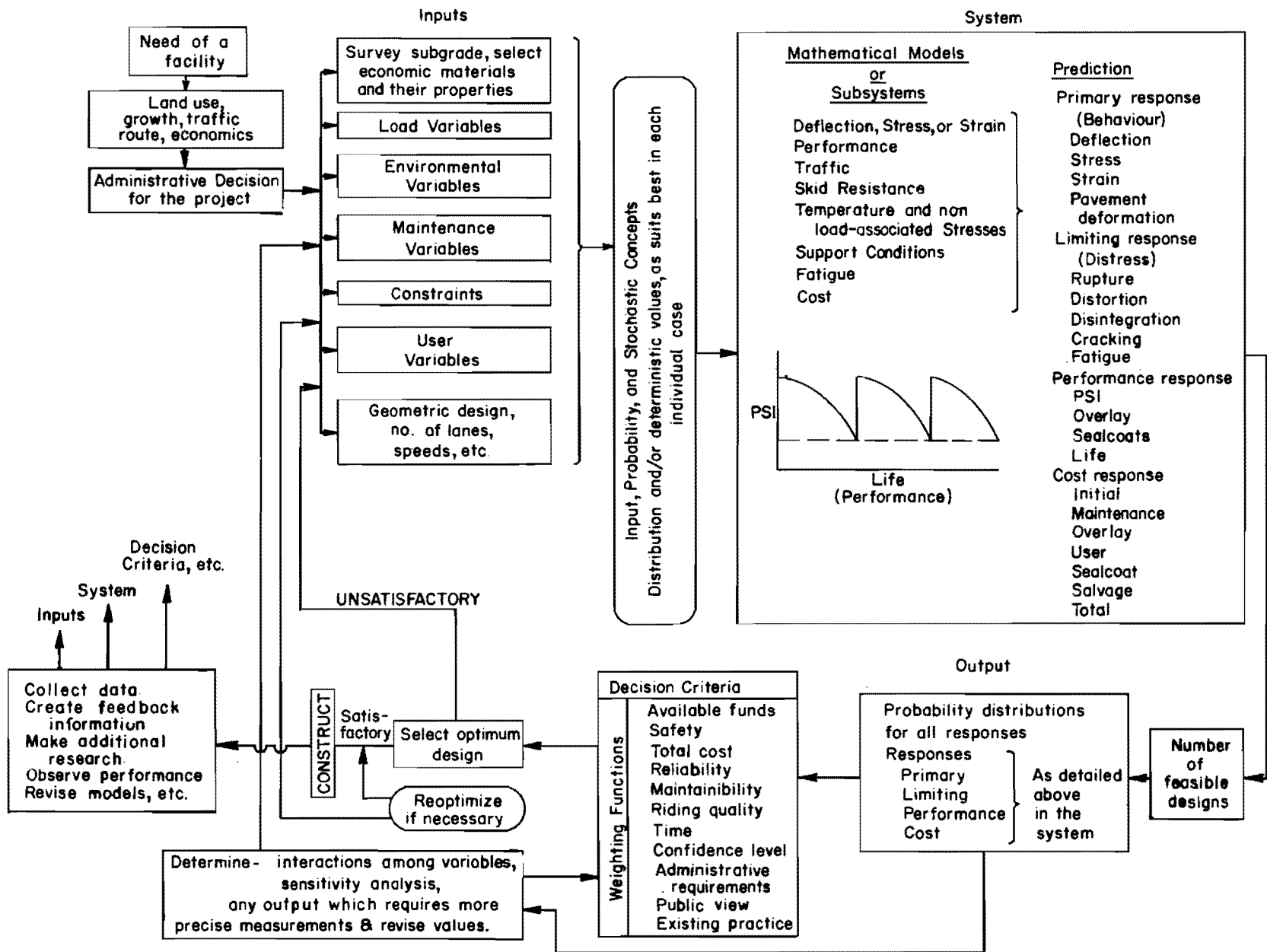


Fig 3.2. Conceptual pavement design system.

SCI are assumed to behave in the same way, irrespective of different combinations of thicknesses.

These methods use either strength coefficients or structural numbers; but neither strength coefficients nor structural numbers can be correctly defined nor assigned units, nor can their values be accurately predicted from laboratory tests.

Engineers working with materials recognize that the properties of materials in a specimen vary considerably from point to point and from time to time. These variations are certain to occur in a pavement structure also. Although these variations are recognized from a practical standpoint, the FPS or any other current design procedures do not take this variation into account directly.

In the present FPS model, as shown in Fig 3.3, the material properties, loading conditions, axle applications, and environmental conditions as input are related to the output, i.e., PSI, only empirically. Some rational and theoretical basis is needed for correlating the above factors. Different distress manifestations are not quantified separately.

The swelling clay parameters in the present FPS are very empirical in nature and need to be quantified on some theoretical basis.

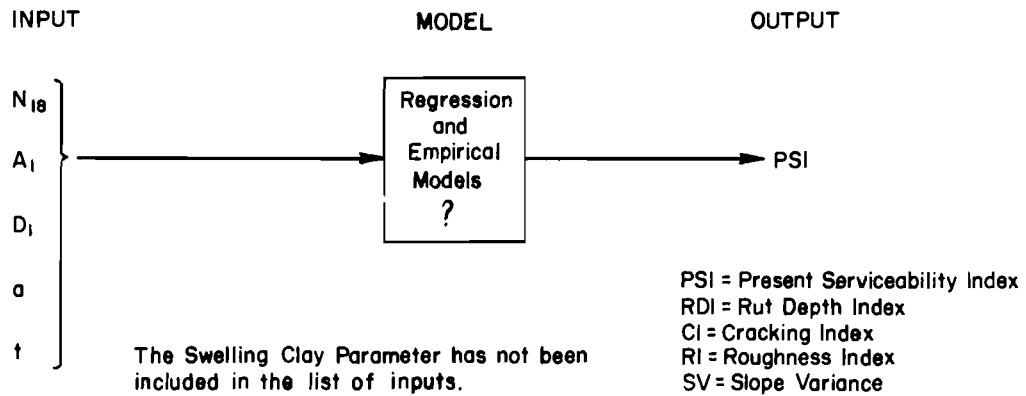
#### PROPOSED REVISION OF FPS

Based on some noted discrepancies of FPS design methods and other factors discussed herein, a revision to the existing FPS is presented.

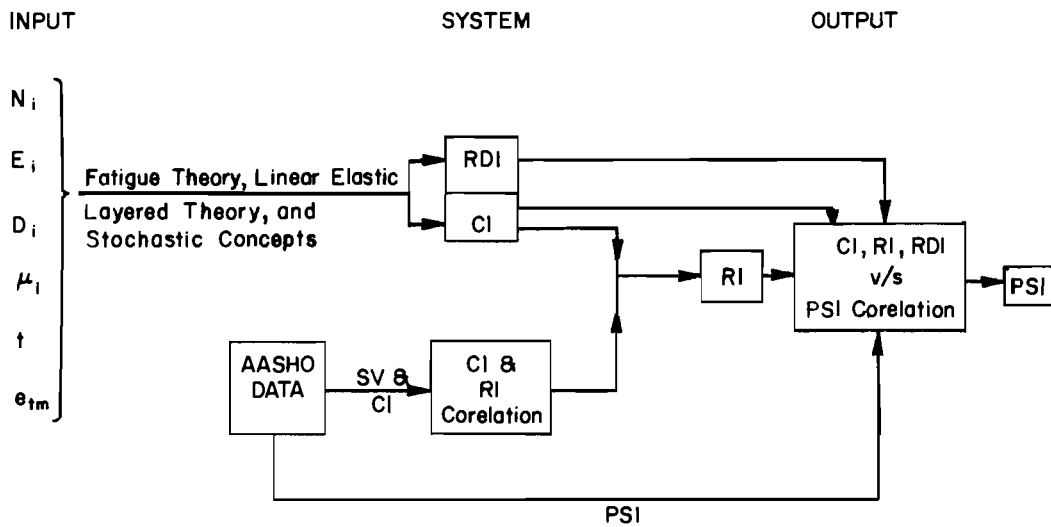
##### Factors to be Considered in the Design of Flexible Pavements

The design of flexible pavement requires consideration of several complex and interrelated factors. The conceptual pavement design system shown in Fig 3.2 details the inputs to the system, the different models needed, the predictions they provide, and the output from the system. It also includes the decision criteria and gives steps in selection of a best design. In the revision of the FPS, consideration of this conceptual pavement system is very important to assure that as many factors are included as the state-of-the-art permits.

Based on the work of Barksdale and Leonards (Ref 6) and other available literature, it appears that the following factors are those most important for the design of flexible pavements.



(a) Present FPS Model



(b) Proposed FPS Model-Second Generation

- $N_i$  = Number of Single axle applications of  $i^{th}$  load group
- $N_{18}$  = Number of Equivalent 18 kip axle Applications
- $A_i$  = Structural Number or Strength Coefficient in AASHO & Deflection Model
- $D_i$  = Thickness of the Pavement Layers
- $E_i$  = Modulus Values of the Pavement Materials
- $\mu_i$  = Poisson's Ratio
- $a$  = Daily Temperature Constant
- $t$  = Time since Initial Construction
- $e_{tm}$  = Environmental Effect of Temperature & Moisture Content

Fig 3.3. Present FPS model and proposed FPS second generation.

- (1) Cracking and/or rutting due to stress and strain from wheel loads (Ref 6).
- (2) Fatigue failure in the surface materials due to repeated flexing induced by elastic deformations in the underlying components of the pavement. Cracking of the surface materials can lead to deterioration of the entire pavement due to the resulting increase in transmitted stresses (Ref 6).
- (3) Cracking and rutting of the surface material due to shear displacement and/or compaction of the base and subbase. Compaction of base materials generally leads to increased stability. Patching and resurfacing will restore the pavement, and further deterioration due to this cause is likely to be relatively minor. In any case, improved methods of compacting granular materials in the field and use of stabilized bases have reduced the occurrence of this defect (Ref 6).
- (4) A general (punching) shear failure due to inadequate shear strength of the subgrade. Such failures occur rapidly under the action of a few heavy wheel loads and damage the pavement severely. A large increase in water content, due to frost action, for example, may lower the strength of the subgrade excessively. Proper subgrade sample analysis may help to avoid this type of failure (Ref 6).
- (5) Cracking and rutting due to cumulative permanent deformation of the subgrade, base, and subbase layers which increases with increased stresses, traffic volume, and time (Ref 6).
- (6) Aftereffects of cracking and rutting in the form of surface roughness or slope variance.
- (7) Surface cracking due to extreme temperature variations.
- (8) Other environmental effects, including the effects of foundations movements, swelling clays, asphalt oxidation, and change in support conditions.
- (9) Effects due to stochastic variations in the material properties with space and time.

#### Extent to Which the Above Factors Are Considered at Present

On the basis of current theories, the ultimate strength methods discussed in Chapter 2 consider failure mechanisms (1) and (4) above. Layered theory can be utilized to calculate the stress and strain in the pavement layers to avoid failure mechanisms (1) through (5). However, none of the present procedures considered all the failure mechanisms (1) through (5). No theoretical approach is available to quantify the roughness of the pavement stated in item (6) above, other than the actual measurement of this distress on the pavements under consideration. Quantification of this distress by any theoretical means is open for future research. In this report, the surface roughness has been quantified by statistical analysis based on field data. Though a great deal of work has

been done for items (7), (8), and (9) and there are several ways to get qualitative information as to their effect on pavement, no quantitative and rational procedure is available which considers them in pavement performance.

#### SCOPE OF THE PRESENT REPORT

The proposed models for the design of flexible pavements developed in this dissertation utilize linear elastic layered theory, fatigue theory, and probability theory. Based on these theories and concepts as shown in Fig 3.3, factors (1) through (6) have been quantified on a more rational and theoretical basis. Factor (9) has been considered. The strength and stiffness coefficients of FPS are replaced by more realistic measurable properties, i.e., moduli of materials.

Considering the fatigue phenomenon, the systems input are correlated, in terms of measurable material properties, loading, and environmental conditions, to its distress manifestations, such as cracking and rut depth. Based on AASHO Road Test data, the correlation between cracking and roughness index is developed. The serviceability and performance concept of the AASHO Road Test has also been utilized. Thus, models are developed for distress manifestations to predict pavement performance and present serviceability index. The models have been verified with the AASHO Road Test data. In the present report, theoretical and empirical approaches have been combined to give the best design procedure possible within the present state-of-the-art.

In the revision of the FPS models, the new design models will replace the empirical relationship used at present to simulate the transformation between the input variables and performance of a pavement as shown in Fig 3.3. This revision will lead to the second-generation FPS. To develop second-generation FPS, the existing structural models for traffic load applications are replaced by the proposed design models, and existing economic and other models are used to study the various design strategies and obtain the best alternative design. The replacement of the existing FPS structural models for fatigue is explained in the following paragraphs.

The present serviceability index (PSI) of a pavement can be conceptually represented as

$$\text{PSI} = f[\text{fatigue (traffic load applications), swelling clay, temperature stresses}] \quad (3.1)$$



For the second-generation FPS, the first term on the right of Eq 3.1 is quantified in this dissertation by improved performance and distress index models to replace the existing FPS structural performance model for traffic applications. The FPS swelling clay performance equation remains unchanged, but further improvements (Ref 187) in the models need to be investigated.

The last term of Eq 3.1, representing the deterioration in PSI due to major temperature stresses, is not presently considered in the FPS. Research on this item is in progress at the Center for Highway Research. The models developed for temperature stresses are planned for the second-generation FPS.

The existing FPS performance equation includes traffic and swelling clay parameters as given in Eq 3.2 (Ref 113).

$$P = 5 - \left[ \sqrt{5 - P_{k-1}} + \frac{\beta S_k^2}{\bar{\alpha}} (N - N_{k-1}) \right]^2 - 0.335C_1C_2 \left[ e^{-\theta t_{k-1}} - e^{-\theta t} \right] \quad (3.2)$$

where

- P = the present serviceability index at time t ,
- $P_{K-1}$  = the present serviceability index at time  $t_{k-1}$  ,
- $\beta$  = a constant = 53.6 ,
- $S_K$  = the surface curvature index for  $K^{\text{th}}$  performance period ,
- N = the number of 18-kip equivalent load applications adjusted by the risk factors to give an acceptable confidence limit at time t ,
- $N_{K-1}$  = the number of 18-kip equivalent load applications at the confidence level which occurred at the end of the  $(K - 1)^{\text{th}}$  performance period ,
- $\bar{\alpha}$  = a temperature constant which varies geographically ,
- $C_1$  = the fraction of a roadway length that has expansive clay in locations that are likely to promote volume change ,
- $C_2$  = the maximum amount of differential heave that is likely to be noted along a roadway ,
- $\theta$  = a constant which determines the rate of heaving of the expansive clay .

$$N = \frac{N_c}{C_L(\gamma_c + \gamma_o)} \left[ 2\gamma_o t + \frac{\gamma_c - \gamma_o}{C_L} t^2 \right] \quad (3.3)$$

where

$N_c$  = the number of accumulated 18-kip equivalent load applications during the analysis period adjusted by the risk factor to give an acceptable confidence limit,

$C_L$  = analysis period in years,

$\gamma_o$  = average daily traffic at the beginning of the analysis period,

$\gamma_c$  = average daily traffic at the end of the analysis period.

The underlined portion of Eq 3.2 represents the terminal PSI due to traffic load applications, and the other portion represents loss in the PSI due to swelling clay.

In the proposed models, the underlined portion of the existing FPS is replaced by the performance model developed in Chapter 7 (Eq 7.5) and distress index models developed in Chapters 8 to 10 (Eqs 8.7, 9.1, and 10.15). Based on values of distress indices computed from distress index models, the present serviceability index is obtained from the performance model (Eq 7.5). The PSI thus obtained is substituted for the underlined portion of Eq 3.2 and the final PSI is computed by subtracting the loss in PSI due to swelling clay. The distress index models and therefore the proposed performance model, as detailed in Chapters 7 to 10, is a function of several parameters, such as traffic load, actual number of traffic applications each month, the month in which the facility is opened for traffic, total time, several material properties and their stochastic variations, confidence level, deformation characteristics of materials, and environmental conditions, as compared to the factors in the existing FPS shown in the underlined portion of Eq 3.2.

The proposed procedure utilizes the actual load repetitions each month for each load group, instead of only one 18-kip equivalent load group. The traffic load repetitions  $N_{jt}$  for  $t^{\text{th}}$  month for  $j^{\text{th}}$  load group separately can be computed from Eq 3.4 if the traffic growth rate  $\gamma_j$  and initial traffic repetitions  $N_j$  of a load of level  $j$  are known:

$$N_{jt} = N_j(1 + \gamma_j)^t \quad (3.4)$$

However, if desired, with modification in the proposed procedure, Eq 3.3 can also be utilized for traffic computations.

#### Work on Items Not Covered in the Present Report

This report covers only a part of the whole work required to idealize the FPS models and continued research efforts are being made by various agencies and individuals (Ref 81) in this direction. Even for the second generation of FPS, further efforts are required and M. Y. Shahin and M. I. Darter, both of the Center for Highway Research, The University of Texas at Austin, are working to quantify the effects of surface cracking due to extreme temperature variations and stochastics for other variables not considered in this report, which will also be included in the second generation of FPS.

At present, the effects of foundation movements, asphalt oxidation, change in support conditions, etc., still need to be taken into account on some rational and theoretical bases and are fields open for further research.

#### BASIC WORK PLAN

The structural design procedure based primarily on fatigue and stochastic concepts and developed in this report can be considered as a subsystem of the whole "systems of pavement analysis, design, and management" or the "ideal pavement systems design" model. A flow diagram representing the work plan for developing this subsystem is shown in Fig 3.4. This figure represents a basic work plan for the subsystem developed in this report and it also establishes a format for other areas, such as the effect of extreme temperature variations, to be included in the pavement system in subsequent studies by others.

This report covers the steps that lead to development of a satisfactory design process; after that stage, the remaining process involves putting the concept into practice.

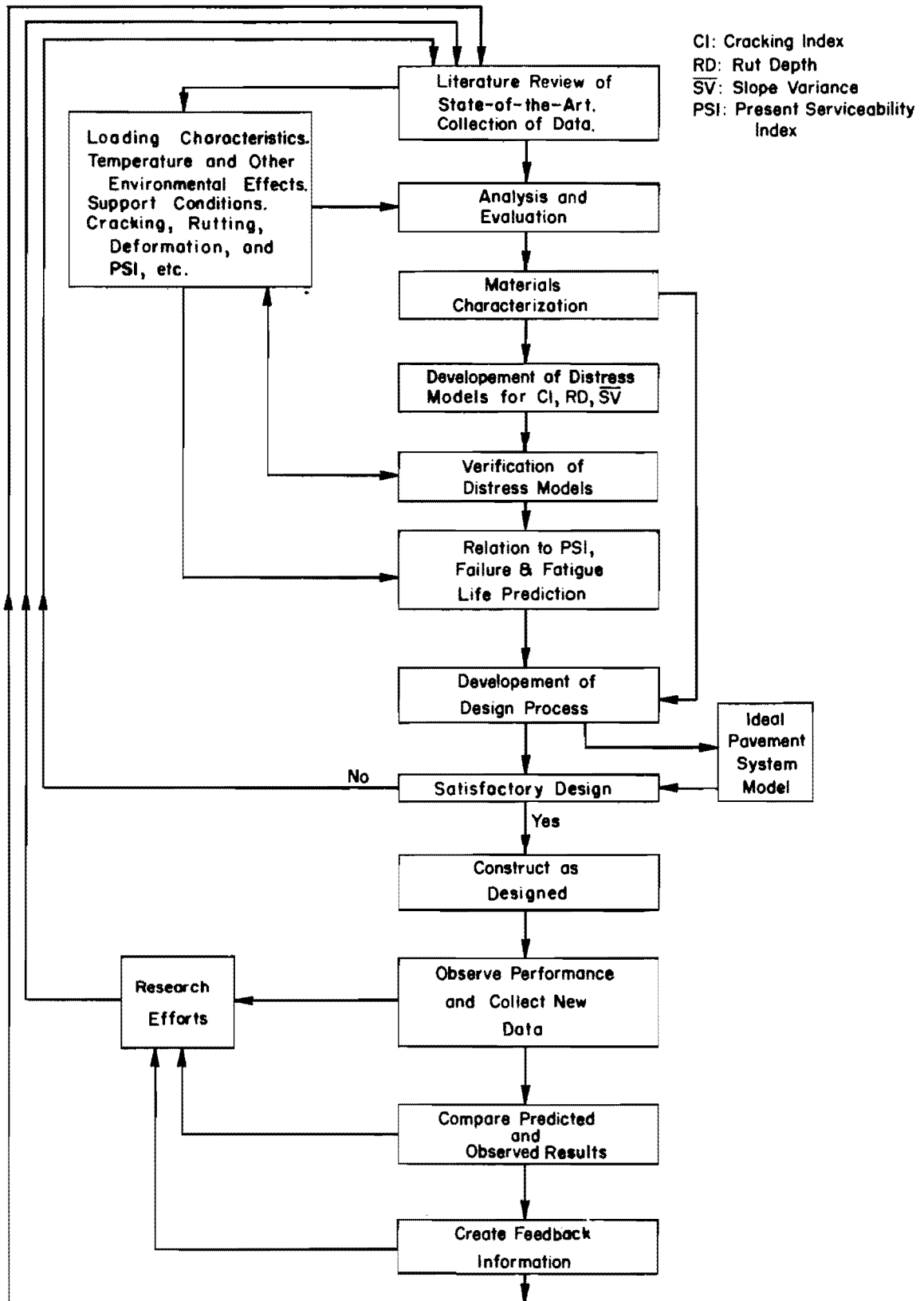


Fig 3.4. Basic work plan for a pavement design subsystem.

PART II

REVIEW OF AVAILABLE TECHNIQUES TO BE USED IN THE MODELS

## CHAPTER 4. FATIGUE OF PAVEMENT MATERIALS

The importance of the proper consideration of fatigue\* in pavement systems design and the proposed revision of the existing FPS models was discussed in Chapter 3. The object of this chapter is to provide an up-to-date review of fatigue theory as it applies to the design of flexible pavements in the design procedure proposed in this report.

### INTRODUCTION TO FATIGUE

Fatigue type failure in the surface layer of a pavement, indicated by cracking on the surface, is caused by repeated tensile flexural strains from moving loads. As a wheel load passes over a pavement, it is subjected to a rapid build-up and decrease in stress, and the extreme fibers of the surface layer are subjected to repeated flexural strains. To simulate and study the effects of dynamic wheel loads, repeated load tests of surface, base, subbase, and subgrade materials are required. The material samples must be prepared and tested according to a procedure which closely simulates the field conditions.

Generally, the use of nomenclature in available literature for flexural fatigue tests on asphalt concrete and repeated load deformation tests on base, subbase, and subgrade materials has not been consistent and clear. The nomenclature used in this report is given in Appendix 6.

### ASPHALTIC CONCRETE

Only in recent years has the fatigue behavior of bituminous materials been closely scrutinized; thus, the knowledge of asphaltic concrete fatigue behavior is not as well developed as it is for metals. In recent years

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\* The fatigue has been defined (Ref 42) as "Phenomenon of a fracture under repeated or fluctuating stress having a maximum value less than the tensile strength of the material."

considerable evidence has been accumulated to attest to the fact that flexible pavements exhibit distress due to flexural fatigue caused by the repetitive application of vehicular loads (Ref 100). Descriptions of fatigue studies development are given by Deacon (Ref 24) and Finn (Ref 42).

In 1953, Nijboer and van der Poel (Ref 24) suggested that fatigue may be a significant cause of cracking in asphalt pavements. Hveem (Ref 84) has presented evidence that distress due to fatigue cracking can and does occur in flexible pavements, especially when highly resilient subgrades are encountered. Extensive laboratory studies of asphaltic concrete mixture fatigue behavior have been carried out by Monismith et al at the University of California (Refs 124, 126, 127, 128, 129, and 130). Other investigators who contributed knowledge of fatigue in asphaltic concrete include Heukelom and Klomp (Ref 60), Saal and Pell (Ref 156), Papazian and Baker (Ref 141), Jiminez and Gallaway (Ref 95), Kirk (Ref 105), Vallergera (Ref 180), Garrison (Ref 48), Bazin and Saunier (Ref 5), and Finn and Hicks (Ref 181).

Finally, the WASHO and AASHO Road Tests proved that fatigue distress and failure are due to fatigue cracking in flexible pavements. Distress due to fatigue in pavements is influenced by heavy loads, a large number of repetitions, and the type of foundation materials.

#### Classes of Fatigue Cracking

Fatigue cracking in flexible pavements is generally characterized by map patterns (Ref 24). Four types of cracking were defined at the WASHO Road Test (Ref 74). In the AASHO Road Test (Ref 70), cracking was divided into three categories. Class 1 cracking was the earliest type observed and consisted of fine disconnected hairline cracks. As distress increased, Class 1 cracks lengthened and widened until cells were formed, causing alligator cracking, known as Class 2. When the segments of Class 2 cracks spalled more severely at the edges and loosened until the cells rocked under traffic, the situation was called Class 3 cracking.

#### Fatigue Failure Hypothesis

Pavement experiencing fatigue starts developing cracks which leads to other forms of distress. The combined effect of these distress manifestations is the measure of pavement performance. The process of fatigue deterioration may be described as

- (1) existing flaws in the pavement, random distribution;
- (2) nonvisible cracking: load repetition increases the number of flaws and widens existing flaws; this widening is not enough to be visible, but enough to cause distress and deformation. This stage is just prior to Class 1 cracks as defined earlier.
- (3) visible cracking: Class 1 to Class 6 cracking as defined at the WASHO Road Test and Class 1 to Class 3 as in the AASHO Road Test. The increase in this form of cracking results in further increase of deformation in the form of roughness and rutting. Water percolation through these cracks may initiate the distress manifestations.

Cracking itself may be of a little significance in the PSI equation, but from the above discussion it seems that cracking is a good overall indicator of pavement performance and other forms of distress in the pavement. A hypothesis that cracking is preliminary to other forms of distress in a pavement, and the correlation of other distresses with the cracking index seems reasonable. Further development of design principles based on this type of hypothesis is dealt with in Chapter 10 of this report.

It is further hypothesized that as the fatigue cracking in asphaltic concrete starts from the existing flaws and the initial distribution of flaws in a structure is stochastic, the whole process of distress development and pavement performance prediction should be based on stochastic principles.

#### Laboratory Fatigue Tests

In fatigue testing the variation in the number of cycles to failure is usually quite large. The ratio of cycles to failure for identical specimens subjected to a given stress level has been reported to be as high as 100 to 1 (Ref 42). This fatigue is recognized as a stochastic process, and a sufficient number of specimens must be tested to predict a probability distribution (Ref 42).

Fatigue behavior in the asphaltic concrete is generally determined in repeated flexural tests in the laboratory in two ways:

- (1) controlled constant load, or stress; and
- (2) controlled constant deflection, or strain.

The controlled stress mode of loading results when the magnitude of the repetitive load applied to the test specimen is maintained constant. In such a test, the deflection of the specimen under each successive load application will gradually increase as damage occurs. In the controlled strain test, the



deflection or strain within the test piece is maintained constant by controlled reduction of each load applied to the specimen as damage is accumulated. Figure 4.1 illustrates each of these test modes (Ref 100).

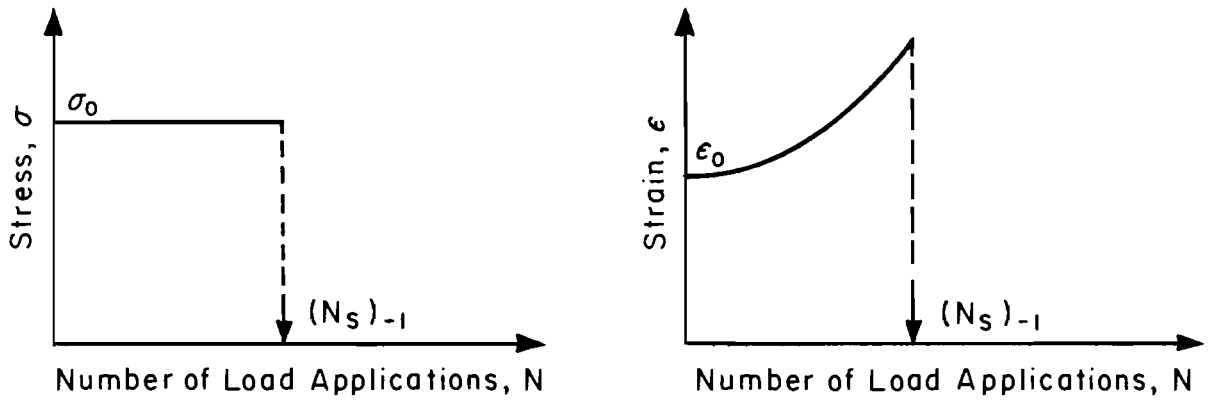
Hicks (Ref 62) has attempted to evaluate the applicability of the controlled stress and strain tests on the basis of computations of elasticity applied to a three-layer pavement. Computations were based on a uniform surface load of 70 psi over a 5-inch radius. Figures 4.2 and 4.3 summarize the results of computations for tensile strain in the under side of the surface layer. In Ref 42 it was shown that a 1-inch thickness of asphaltic concrete surfacing would, for a given loading, be subjected to constant strain regardless of the total thickness of the pavement and the stiffness modulus of the asphaltic layer. Therefore, a constant-strain fatigue test was suggested for thin surface layers. Computations for stress are shown in Figs 4.4 and 4.5. These indicate that the thicker sections are subjected to a relatively constant stress, which suggests the constant stress mode of testing for thicker pavement surfaces (Ref 42).

In a fatigue life study of asphalt and cement-treated bases Gallaway (Ref 46) has made some plots based on linear elastic layered theory and verifies that thicker sections are subjected to a relatively constant stress condition.

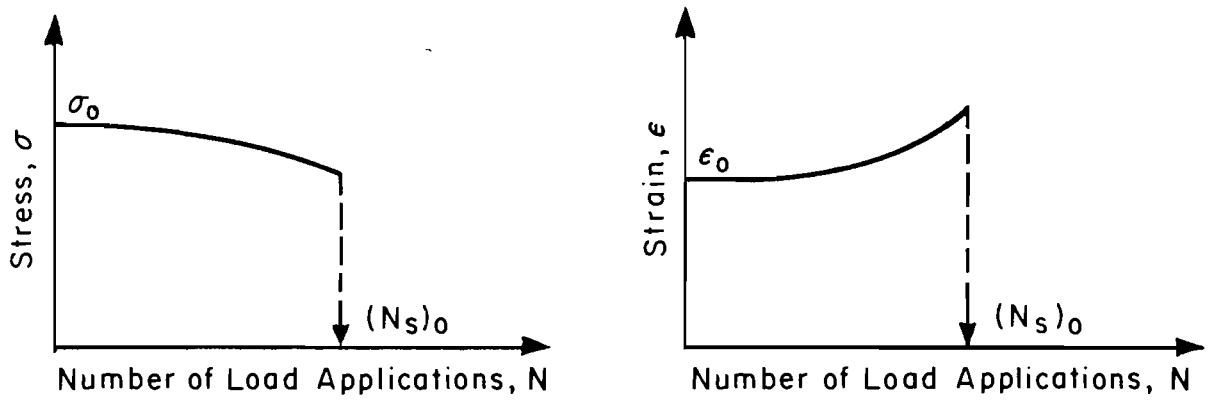
Monismith in a paper presented at the University of Nevada in 1966 has suggested that for surface layers less than 2 inches thick the controlled strain mode of testing is applicable, while for asphaltic concrete layers 6 inches thick or greater, the controlled stress mode of loading is appropriate. Between these two thicknesses some intermediate mode of loading should be applied (Ref 100).

In NCHRP Report 39 (Ref 42), Finn explained that in addition to other reasons the in-situ pavement will generally be subjected to constant load conditions, and the loads during the lifetime will not be reduced to maintain a constant strain in the asphaltic layer. From this he concluded the constant stress test to be a more logical mode of laboratory testing for pavement designs.

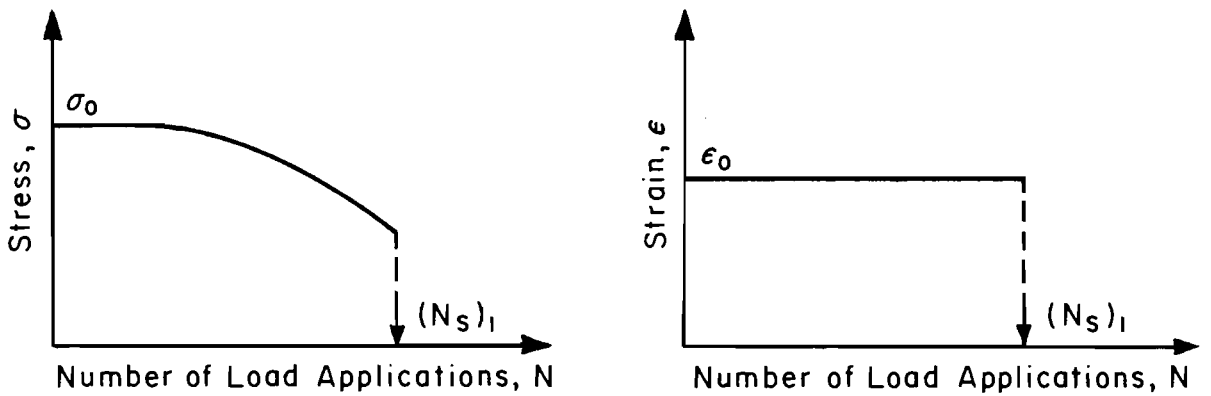
Based on the following considerations, Kaisianchuk (Ref 100) suggested the controlled stress mode of loading to determine the fatigue response of the asphalt concrete:



(a) Controlled - stress loading.



(b) Intermediate mode of loading.



(c) Controlled - strain loading.

Fig 4.1. Schematic representation of fatigue behavior of asphalt paving materials for various modes of loading (after Monismith and Deacon).

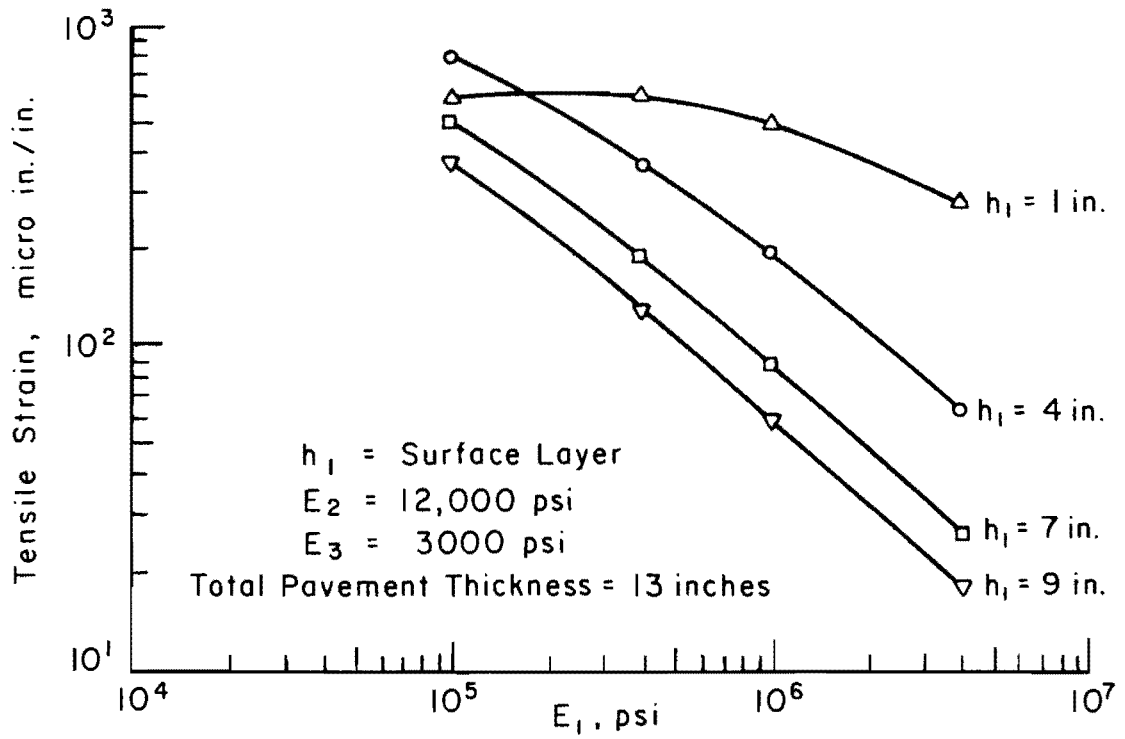


Fig 4.2. Induced tensile strain as a function of surface modulus, thin section (after Hicks).

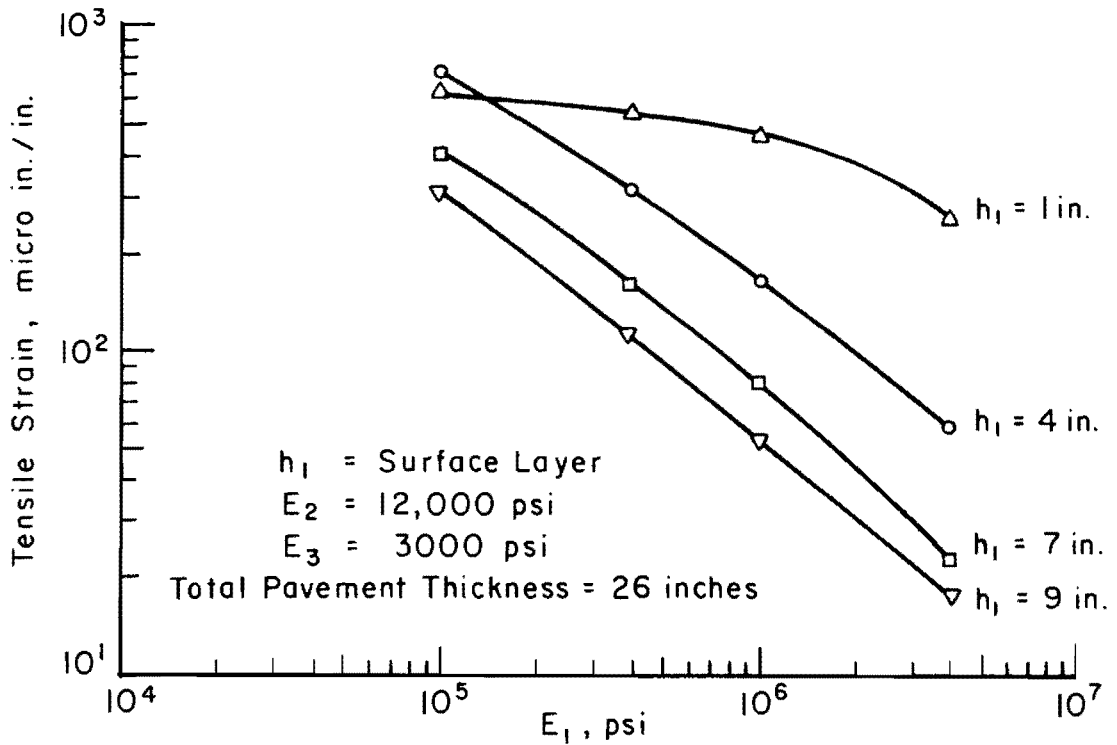


Fig 4.3. Induced tensile strain as a function of surface modulus, thick section (after Hicks).

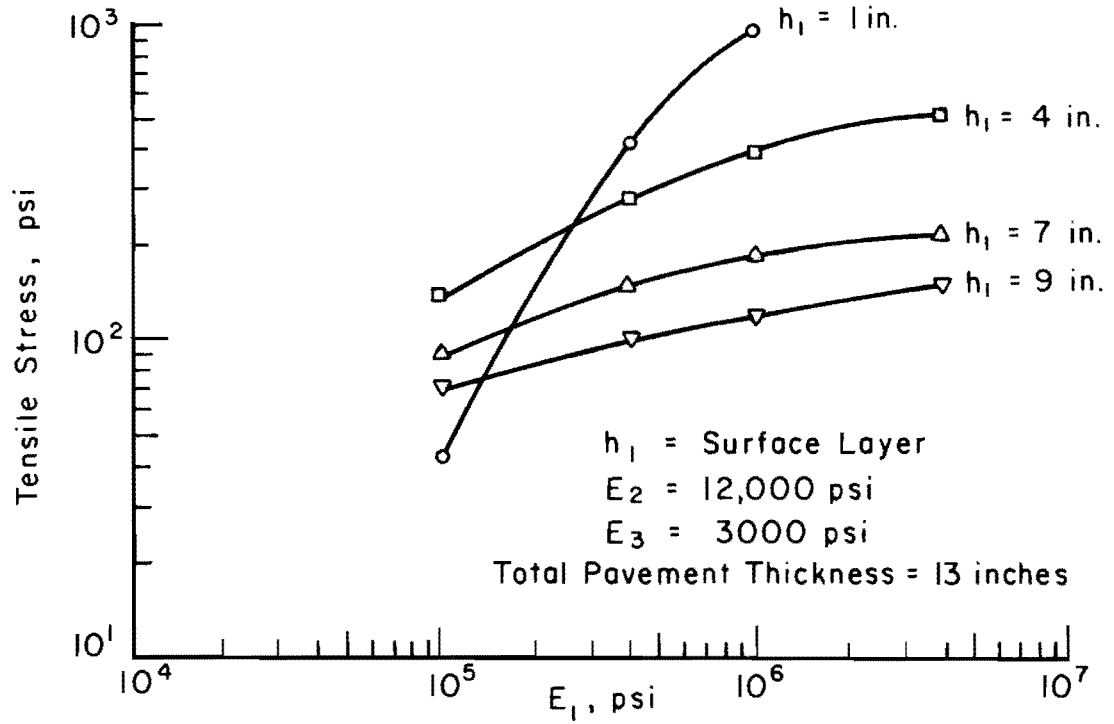


Fig 4.4. Induced tensile stress as a function of surface modulus, thin section (after Hicks).

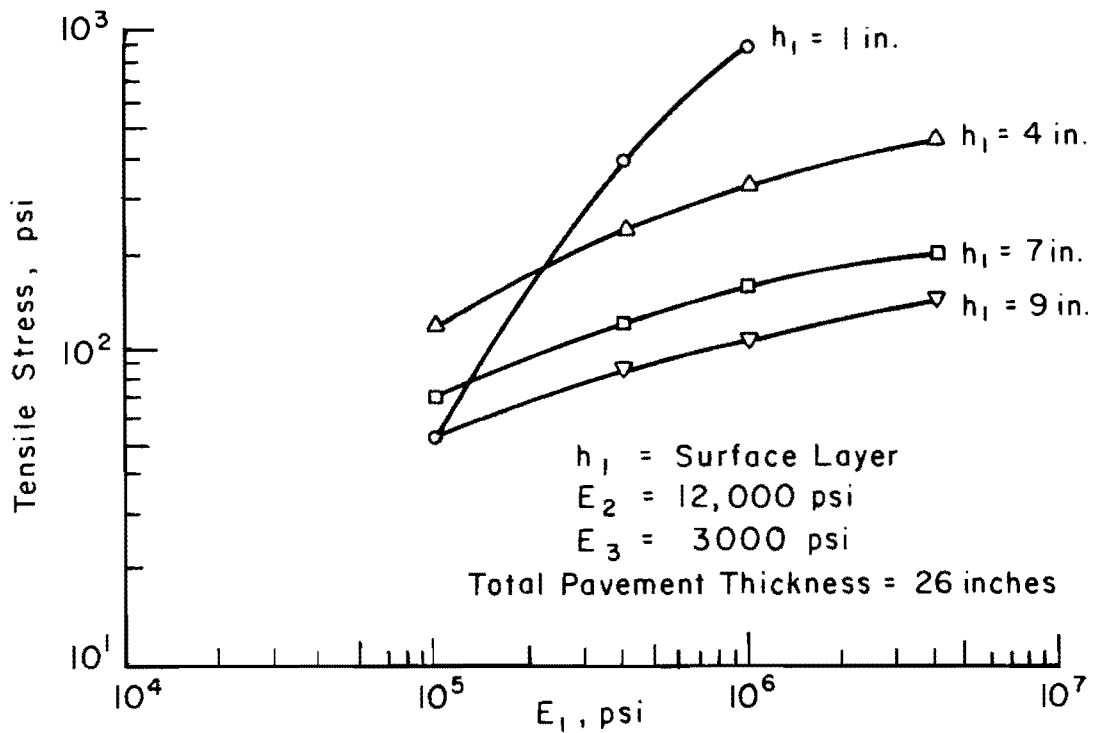


Fig 4.5. Induced tensile stress as a function of surface modulus, thick section (after Hicks).

- (1) The majority of pavements in which fatigue in asphaltic concrete need be considered will be those in which high traffic volumes and weights will require relatively thick asphaltic concrete layers. In these cases, the controlled stress mode of loading is applicable.
- (2) In the relatively small number of cases in which the controlled strain mode of test is applicable, the controlled stress mode will lead to shorter predicted lives and is, consequently, conservative.
- (3) The controlled stress mode of loading fatigue test results in complete fracture of the test specimen so that no difficulties arise regarding the definition of service life. The test can also be more easily performed in that no regulation of loads is required.

In view of these discussions, fatigue test results based on the controlled stress mode of loading will be adopted in this report, as given in the following paragraph.

#### Fatigue Test Results

Laboratory fatigue test results are typically plotted as fatigue life against some measure of the load magnitude repeatedly applied to the test specimen. For the case of the fatigue testing of asphaltic concretes there is evidence (Ref 42) that this relationship may be adequately represented by a straight line on a plot of the logarithm of the fatigue life against the logarithm of the tensile strain level. For the controlled stress mode of testing, in which the strain level varies throughout the test, this linear relationship holds when the initial level of strain is employed. The logarithmic linear relationship can be expressed, as has been done by Pell (Ref 146) and Deacon (Ref 24), by an equation of the form:

$$N_j = A \left( \frac{1}{\epsilon_j} \right)^B \quad (4.1)$$

where

$N_j$  = cycles to failure at a particular stress level,

$\epsilon_j$  = bending strain,

A and B = constants depending on mixture characteristics.

NCHRP Report 39 (Ref 42) contains a discussion of asphaltic concrete fatigue behavior under repeated loading. The following is a summary of the significant results of pertinent field and laboratory studies given in the report:

- (1) Fatigue behavior of asphaltic concrete is similar to that of metal, wood, portland cement concrete, etc., and it appears in laboratory tests as well as in the field.
- (2) A linear relationship exists between the log of stress or strain level and the log of repetitive loads to failure.
- (3) Generally constant stress-type tests will respond with an increasing fatigue life to any mix property which increases the stiffness of the asphaltic concrete. For constant strain tests, the effect of stiffness modulus is reversed. However, at a very low temperature (approximately 32° F), the fatigue life is unaffected by the mode of testing. Table 4.1 exhibits some basic parameters to be considered in the discussion of the laboratory fatigue life test results applicable to the design of pavements. The table exhibits the effect of these parameters on the stiffness and fatigue behavior of asphalt concrete mixtures.
- (4) Longer durations of load application are associated with reduced fatigue life.
- (5) The change in stiffness modulus, deflection, or modulus of rupture during repetitive loading tests may be used to measure fatigue damage. A higher rate of damage appears to occur with the first 10 percent of the repetitive loadings, with a relatively constant and somewhat reduced rate for the next 80 percent of the loadings, followed by an abrupt change to failure.
- (6) Tensile strain is the prime determinant of fatigue life. The test results when converted from stress to strain are essentially independent of the rate of loading (at least for less than 30 applications per minute) and temperature and closely follow the straight line relationship given in Eq 4.1. Any difference in the test results was explained as due to the difference in the rate of crack proportion.
- (7) Stress reversal appears to have little effect on the rate of the asphalt concrete cumulative damage.
- (8) As long as the temperature and rate of loading do not vary markedly, a mixture of asphaltic concrete will act elastically up to approximately 0.1 percent strain. Thus, it is possible to analyze asphaltic mixtures according to the theory of elasticity for a given situation as represented by a modulus of elasticity or stiffness modulus value.
- (9) Test procedures described in the report can be combined with the multilayered theory for computing stress and strain in the asphaltic surfacing and used, at least qualitatively, to predict expected performance.

TABLE 4.1. FACTORS AFFECTING THE STIFFNESS AND FATIGUE BEHAVIOR OF ASPHALT CONCRETE MIXTURES  
(After Kasianchuk)

Factor	Change	Result of Change		
		Stiffness	Fatigue Life in Controlled Stress Mode of Test	Fatigue Life in Controlled Strain Mode of Test
Asphalt penetration	Decrease	Increases	Increases	Decreases
Asphalt content	Increase	Increases <sup>(1)</sup>	Increases <sup>(1)</sup>	Decreases <sup>(2)</sup>
Aggregate type	Increase roughness and angularity	Increases	Increases	Decreases
Aggregate gradation	Open to dense gradation	Increases	Increases	Decreases <sup>(2)</sup>
Air void content	Decrease	Increases	Increases	Increases <sup>(2)</sup>
Temperature	Decrease	Increases <sup>(3)</sup>	Increases	Decreases

(1) Reaches optimum at level above that required by stability considerations.

(2) Not based on significant amount of data but seems reasonable on basis of other information.

(3) Approaches upper limit at temperature below freezing.

The use of constant stress test results in pavement design reduces the efforts required for laboratory fatigue investigations and provides the basis for development of a rational pavement design procedure based on fatigue, by use of Eq 4.1 and fatigue damage hypothesis.

Further simplification of laboratory investigations would make the design method even more practical. As observed by Kaisianchuk (Ref 100), attempts are being made to provide more simplifications.

#### Damage Hypothesis

Deacon (Ref 24) performed an analysis of the applicability of various compound loading hypotheses to the prediction of asphaltic concrete fatigue life from simple loading test results. The best available hypothesis seems to be the simple linear summation Miner's hypothesis (Chapter 8), and it will be used in this report.

#### Application of Fatigue Equation and Miner's Hypothesis

The application of the fatigue equation (4.1) and Miner's hypothesis is explained by the flow diagram shown in Fig 4.6.

The strain induced by the applied load is calculated by layered analysis. Substitution of the strain value in Eq 4.1 gives the value of  $N_j$ , the number of load applications of level  $j$  which will cause failure in simple loading. This value of  $N_j$  when substituted in Miner's hypothesis along with the known value of actual number of load applications of level  $j$ ,  $n_j$  will give the "used life" of the pavement. The use of this life prediction in the actual design procedure as developed in this report is explained in Chapter 8, under development of the cracking index model.

#### UNTREATED GRANULAR AND FINE GRAINED MATERIALS

Untreated granular and fine grained materials have different fatigue problems than asphaltic concrete. Repeated applications of loads may result in sufficient cumulative permanent deformations in pavement layers consisting of these materials to cause failures, although a single application of the load would not. These materials in a pavement are normally subjected to a triaxial state of stress. Therefore, the fatigue behavior of these materials under an imposed traffic loading sequence must be analyzed for induced deformations under triaxial states of stress. Although it is unlikely that a method



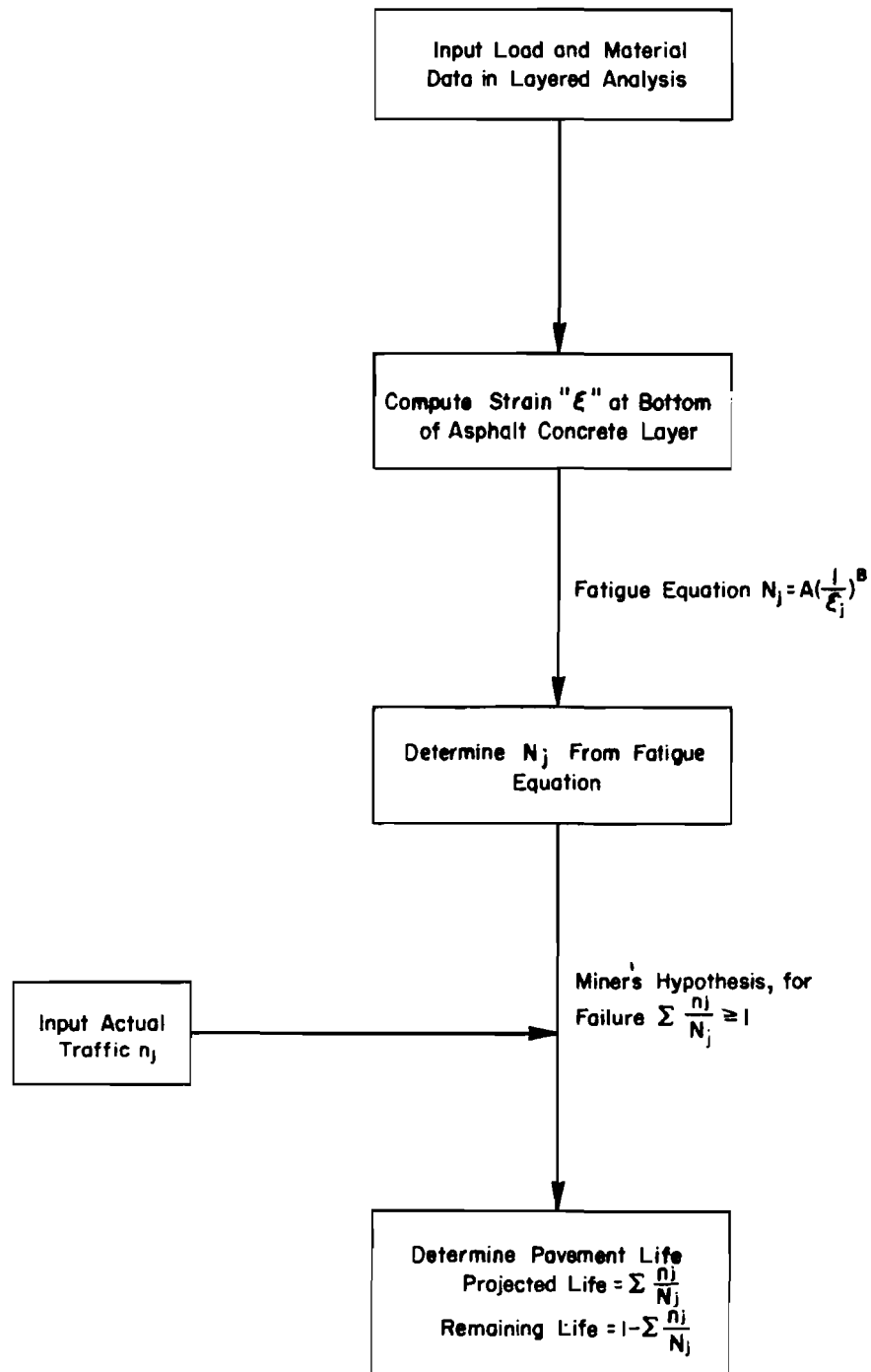


Fig 4.6. Life prediction from fatigue and Miner's hypothesis.

of pavement design will ever be developed to account for the true behavior of the complex polyphase materials used in flexible pavement, the following steps may prove to be a reliable practical approach for taking into account the proper fatigue behavior of these materials:

- (1) establishment of deformation characteristics under repeated triaxial loading;
- (2) analysis of stress, strain, and deformation; and
- (3) analysis of distress and performance.

Though study of the relationship of stress repetition and deformation in roadway materials is not new, information on the deformation characteristics of materials under repeated loading with different combinations of axial and confining pressures which can actually be used directly in the development of a rational design method is very limited. The available information which can be used in these developments is discussed in Chapter 5.

#### Resilient Modulus

To characterize materials for the elastic layered analysis, the modulus of elasticity can be represented by the resilient modulus. NCHRP Report 35 (Ref 164) gives laboratory data for the repeated load test on granular and fine grained materials. By measuring the resilient (or elastic) strain in a repeated-load triaxial compression test, a resilient modulus can be determined at any number of load repetitions from

$$M_r(t) = \frac{\sigma_d}{e_r(t)} \quad (4.2)$$

where

$M_r(t)$  = modulus of resilient-deformation, psi (analogous to an elastic modulus) corresponding to a particular number of stress repetitions;

$\sigma_d$  = repeated deviator stress, psi;

$e_r(t)$  = resilient axial strain corresponding to a particular number of stress repetitions, inches per inch.

Plate load tests at the subgrade surface indicate that the resilient modulus of clay soils varies with applied pressure and water content. The

resilient modulus decreases rapidly in the stress range of 1 to 10 psi (a range to be expected in the subgrades of well-designed pavements) and tends to have a constant value at higher stress levels. At equal ratios of applied stress to failure stress, values of resilient moduli of the subgrade soil determined from laboratory repeated load and plate load tests are essentially the same. The factors influencing the resilience of clays under repeated loads can be summarized as follows (Ref 164):

- (1) Resilient deformations generally decrease with an increase in the number of load repetitions.
- (2) Samples compacted to a high degree of saturation increase in strength with time.
- (3) The resilient modulus generally increases with a decrease in the intensity of stress.
- (4) A method of compaction which produces a dispersed structure tends to produce a lower resilient modulus.
- (5) An increase in the degree of saturation at compaction decreases the resilient modulus (AASHO subgrade soil).
- (6) In general, as the water content of the soil increases due to water absorption after placement, the resilience increases.

So long as there is no shear failure, repeated load triaxial compression tests on dry granular materials indicate the following relationship:

$$M_R = k\sigma_3^n \quad (4.3)$$

where

$M_R$  = resilient modulus,

$\sigma_3$  = confining pressure,

$k, n$  = constants.

The factors influencing the resilience of granular materials can be summarized as follows (Ref 164):

- (1) Higher frequency of load repetitions increases the value of the modulus.

- (2) The type of aggregate and percentage of material passing the No. 200 sieve have a definite effect on the resilient modulus.
- (3) The difference between the moduli of loose and dense sand can be as much as 50 percent.
- (4) An increase in saturation leads to a decrease in the resilient modulus.
- (5) The resilient modulus is independent of the stress level so long as the stress is below a level that causes excessive plastic deformation.

The determination of an appropriate resilient modulus value for subgrades is not a simple problem since the selected subgrade modulus should take the previously noted factors into account. However, using an appropriate laboratory method it is now possible to simulate closely any desired field condition of a soil. For example, kneading compaction produces laboratory specimens with resilience characteristics similar to those observed in field specimens (Ref 164) for the same conditions of test. Thixotropy influence becomes insignificant after about 50,000 repetitions, which is only a fraction of the number of stress repetitions applied to a pavement (Ref 164). The influence of time which is much shorter in the laboratory, needs consideration though the deformation obtained in the laboratory will give conservative estimates of the performance in the field.

For granular materials, also, the laboratory evaluation of resilient modulus imposes several problems. In laboratory testing, estimates must be made for the void ratio, the expected degree of saturation, a reasonable rate of loading consistent with moving traffic, frequency of load applications, a representative number of repetitions consistent with the field conditions, and a representative stress condition based on best judgment and experience.

#### Applications of Repeated Load Test Results

The modulus of resilience of granular and fine grained materials is utilized in the layered elastic analyses to determine the stress and strain in the pavement layers. Then based on stress and strain values and cumulative deformation characteristics of these layers under repeated triaxial loading, the permanent deformation of layers in the form of rut depth is calculated. The computed values of rut depth are finally utilized for pavement performance computations. Development of a rut depth model, in which the above information and procedure are used, is further discussed in Chapter 10.

## SUMMARY

In this chapter the proper fatigue theory and results of repeated triaxial loading tests as applicable to the flexible pavement design were discussed. To design the pavement on these principles, proper characterization of materials is needed to determine the characteristics which are used in the proposed design procedure. The material characterization is discussed next in Chapter 5.

## CHAPTER 5. CHARACTERIZATION OF FLEXIBLE PAVEMENT MATERIALS

### INTRODUCTION

A system transforms its input into output according to certain definite relationships which can be simulated by mathematical models using certain material properties. The basic properties of materials are complex physical functions. However, output responses for engineering analysis can be obtained by characterizing the materials for certain significant engineering properties such as stiffness, strength, etc. The literature shows that a great deal of effort has been devoted to measuring such material properties. However, uniformity in the test procedures and analysis of test results seems to be lacking. Many variables involved in material characterization which affect the material response in a system are given in Appendix 2, Table A2.13. Table A2.14 in Appendix 2 shows the details of the test configurations and Table A2.15 gives the various shapes of test specimens. Various variables shown in these tables affect the material response and point out the importance of having a uniformity in test procedures involved in determination of the basic material properties.

Hudson et al (Ref 78) through the systems approach to pavement design have demonstrated the need for characterizing material properties by means of constitutive equations which in turn can be used in mathematical models of pavement systems. The present analysis is based on linear elastic layered theory using fatigue properties of the materials and their stochastic variation in space and time. Therefore, the following material properties and their variations are required:

- (1) elastic constants resilient or elastic modulus  $E$  and Poisson's ratio  $\mu$  ;
- (2) stochastic variations of elastic constants; and
- (3) stress-strain relationships of materials as affected by time, temperature, and fatigue characteristics.

Table 5.1 is a summary of the tests required for material characterization for the design procedure discussed in this report. In the present analysis

TABLE 5.1. SUMMARY OF SAMPLING AND TESTING REQUIREMENTS (Ref 116)

Material Type	Elastic Constants		Strength and Deformation Tests	
	Sampling	Testing	Sampling	Testing
Asphaltic concrete	Intact or fragmented	<u>Heukelom and Klomp</u> 1. Asphalt penetration at 77° F 2. Ring and Ball softening point ° F 3. Aggregate volume concentration 4. Air voids 5. Time of loading 6. Temperature at which stiffness is required  <u>Nijboer Method</u> 1. Marshall stability 2. Flow value 3. Time of loading 4. Temperature at which stiffness is required	Previously established or reproduced laboratory specimens	Fatigue
Granular Base and subbase	1. Density and moisture 2. Remolded specimens	Resilient modulus $M_R$	1. Density and moisture 2. Remolded specimens	Rupture envelope from triaxial shear. Repeated load triaxial tests varying axial and confining pressures for permanent and resilient strains.
Fine grained subgrade	1. Undisturbed push barrel 2. Density and moisture 3. Remolded specimens	Resilient modulus $M_R$	1. Undisturbed push barrel 2. Density and moisture 3. Remolded specimens	Fatigue.

distress and performance models developed in this report will be verified by the AASHO Road Test performance data. It is necessary, therefore, to characterize the materials used at the AASHO Road Test. Though the details which follow deal specifically with the characterization of the AASHO Road Test materials, the procedure, in general, is applicable for characterizing the materials which will be used with this design procedure.

## ASPHALTIC CONCRETE

### Stiffness

The response of asphaltic concrete to stress and strain is influenced by time and temperature to a pronounced degree. Asphaltic concrete under stress exhibits instant and time dependent strain, both of which may be partly recoverable and partly permanent. The time dependent part may be viscous or non-viscous. Instantaneous strain under moving traffic forms a large proportion of the total strain. Stress history is also important. The material's true response is nonlinear. Table 4.1 outlines the general effects of some variables on the stiffness of asphaltic concrete. However, the elastic properties of asphaltic concrete mixtures have been shown to be represented by its stiffness at a particular time of loading and temperature. A quasi-elastic modulus termed stiffness can be obtained by (1) the Heukelom and Klomp method (Ref 42) as modified by Van Draat and Somner (Ref 38) for greater air voids and (2) the Nijboer method (Ref 173). The parameters required to define the stiffness by these methods are given in Table 5.1.

Appendix 2 details the calculation of the stiffness values of the asphalt concrete used at the AASHO Road Test. Table 5.2 gives the stiffness values adopted for the present analysis.

### Poisson's Ratio

The Poisson's ratio of asphaltic concrete is not a very sensitive parameter in the layered analysis. Any standard test can be adopted to compute the value of this variable. According to NCHRP Report 39 (Ref 42) Poisson's ratio in general varies from 0.3 to 0.5 for a small deformation. A value of 0.3 is appropriate at cold temperatures (less than 40° F) and at a loading time of 0.1 second. At higher temperatures and slower rates of loading the value may increase to 0.5. A value of 0.3 is reported by Deacon (Ref 26). For the present analysis, a value of 0.3 is adopted.



TABLE 5.2. STIFFNESS VALUES FOR ASPHALTIC CONCRETE  
(AASHO ROAD TEST MATERIAL)

Month	Temperature, °F	Stiffness Modulus, psi × 10 <sup>5</sup>
January	21 <sup>o</sup>	16.0
February	25 <sup>o</sup>	14.2
March	27 <sup>o</sup>	13.7
April	41 <sup>o</sup>	9.0
May	54 <sup>o</sup>	6.0
June	66 <sup>o</sup>	4.0
July	70 <sup>o</sup>	3.5
August	75 <sup>o</sup>	3.0
September	65 <sup>o</sup>	4.2
October	51 <sup>o</sup>	6.5
November	43 <sup>o</sup>	8.3
December	28 <sup>o</sup>	13.1

### Stochastic Variation in Elastic Properties

The stochastic variation in stiffness values of asphaltic concrete in space have been considered in terms of the coefficient of variation, as detailed in Appendix 2. The values of standard deviations have been obtained from the available field and laboratory test results. An average value for the coefficient of variation is computed to be about 25 percent of the mean value. Variation of the stiffness value in time has been determined by monthly variation in temperature. Due to the relative insensitivity of Poisson's ratio in the layered analysis the stochastic variation in this parameter has not been taken into account.

### Fatigue Test Data

Repetitive applications of tensile stresses smaller than the tensile strength ultimately cause fatigue cracking in asphaltic concrete. For controlled-stress loading, the mean fatigue life  $N$  is related to the initial tensile strain  $\epsilon$  by Eq 4.1. The values of  $A$  and  $B$  depend upon the type of mixture, the condition of testing, and the failure definition. The values for  $B$  reported for the controlled stress mode of loading (Ref 26) vary from 2.5 to 5.9. The values of  $A$  for asphaltic concrete have been shown to vary from  $10^{-6}$  to  $10^{-10}$  (Ref 38). For asphaltic concrete used at the AASHO Road Test, no direct fatigue test results are available. A value of  $B = 3.1$  and  $A = 6.5 \times 10^{-7}$  was adopted for this analysis. These values correspond to the test results obtained at the University of California (Refs 24, 38, and 100) for similar asphaltic concrete mixtures.

Fatigue test data exhibit extreme variability. However, the fatigue life of specimens tested in simple, controlled stress loading with identical testing conditions can be approximated by a logarithmic normal distribution (Ref 24). The log of the standard deviation of fracture life varies from about 0.2 to 0.4 (Ref 24). However, Kaisianchuk (Ref 100) in his study of asphalt concrete has shown that the logarithm of the standard deviation of fracture life depends on the stress level, but could be assumed to be about 0.25 for a wide range of asphalt mixes. In view of the above and the fact that no fatigue test data for asphaltic concrete at the AASHO Road Test are available, the value of 0.25 was adopted for the purpose of analysis.

## BASE AND SUBBASE GRANULAR MATERIALS

### Resilient Modulus

A review of the limited data on the modulus of deformation for the materials used at the AASHO Road Test reveals a wide range of values. No direct test results applicable to the present analysis are available for the modulus values of AASHO Road Test materials. Various approaches through which modulus  $M_R$  values for AASHO materials for each month were selected are given in Appendix 2. Table 5.3 gives the  $M_R$  values adopted for this analysis.

### Poisson's Ratio

Poisson's ratio is relatively insensitive in the elastic layered analyses, and typical values of 0.4 for the base and 0.45 for the subbase were adopted.

### Stochastic Variation

Information on stochastic variations of elastic properties is not available for base and subbase granular materials used at the AASHO Road Test. Therefore, the standard deviation of modulus value was based on observed variations in the test results of other significant properties having direct relationship to the modulus value. An approximate value of coefficient of variation of about 25 percent was computed. The details of these computations are shown in Appendix 2. No stochastic variation in Poisson's ratio was considered.

### Deformation Properties Under Repeated Loading

The behavior of granular materials under repeated loading is highly dependent on the degree of confinement. Haynes and Yoder (Ref 57) presented the results of undrained repeated-load triaxial compression tests on gravel and crushed stone used as base course at the AASHO Road Test. In these tests, a lateral pressure of 15 psi and a deviator stress of 55 psi were used. For the present analysis, curves representing the actual developed stresses in the pavement sections were required. A literature review revealed that the results of a study performed at Texas A&M University (Ref 35) on nine types of granular materials could be used to obtain this information. To ascertain the possibility of using this information to characterize the properties of the granular materials used at the AASHO Road Test, a comparison of various properties of the two materials was made. This comparison (Table 5.4) shows that the angular medium aggregate used for the A&M University test is similar to the AASHO base material and the rounded fine aggregate is similar to the AASHO subbase material.

TABLE 5.3. MONTHLY VALUES OF MODULUS OF RESILIENCE OF BASE AND  
SUBBASE OF AASHO MATERIAL

Month	$M_R$ , psi	
	Base	Subbase
January	24,000	13,200
February	24,000	13,200
March	24,000	13,200
April	15,600	7,200
May	18,000	8,600
June	19,600	9,800
July	21,600	10,800
August	23,200	11,600
September	24,000	12,200
October	24,000	12,400
November	24,000	12,800
December	24,000	13,200

TABLE 5.4. COMPARISON OF ASSHO BASE AND SUBBASE MATERIAL WITH A&M TYPICAL AGGREGATE

Properties	Base Material		Subbase Material	
	AASHO	A&M	AASHO	A&M
Gradation	See Figures 5.1 and 5.2			
Optimum moisture content, %	7.6	7.0	7.7	7.3
Maximum unit weight	137.9	136.0	133.1	134.0
Texas triaxial class	1	1	3.7	3.0
Plasticity				
a) liquid limit	-	17.8	-	21.3
b) plasticity index	N.P. - 4.3	2.3	N.P. - 3.4	7.4
c) linear shrinkage	-	2.4	-	5.6
Los Angeles abrasion (500 revolutions)	23.9 - 28.3	25.3	25 - 35.4	27.3
Specific gravity	2.78	2.63	2.69	2.64
Permeability (ft/day)	.006 - 140	0.003	.0003 - $20 \times 10^7$	0.006
Brief description	Crushed limestone	Angular medium crushed limestone	Natural sand and gravel	Rounded fine limestone mixed with sand and other calcium carbonate

Table 5.5 compares the repeated load test results given by Haynes and Yoder (Ref 57) for the AASHO Road Test base material (curve s-1-c of Fig A2.1) with those given in the A&M University study for angular medium aggregate. The comparison is made for the total strain at an axial pressure of 70 psi and a confining pressure of 15 psi. The values of total strain in the two cases are approximately the same at 10,000 repetitions. A relatively large difference exists at 100,000 repetitions, which is not likely to influence the average results since the samples were near the failure point at these levels of strains and number of applications. There are many reasons for the difference between the total strain values. A part of the difference can be assigned to the difference in frequency and time of loading during the test in the two cases, as shown in Figs A2.2 and A2.3. Higher strain values would not have been obtained for the AASHO Road Test material if the time and frequency of loading were the same as those for the A&M test materials. For the reasons outlined above and since better data were not available, it is considered appropriate to characterize the fatigue characteristics of the AASHO Road Test base and subbase materials respectively by the angular medium and rounded fine aggregates used at the A&M University test.

#### Models to Characterize the "Repeated Load-Deformation" Characteristics

The values of permanent strain and corresponding load repetitions are tabulated in Tables 5.6 and 5.7 for various combinations of vertical and confining pressures. The range of values for stresses is selected to be comparable with the expected values in the pavement structures under normal traffic loads.

So that the data given in Tables 5.6 and 5.7 could be conveniently used for the present analyses a regression analysis was performed to predict the total strain value as a function of the number of load repetitions, vertical stress, and confining stress. The regression equations are given below.

Base Material:

$$\text{Correlation coefficient } R^2 = 0.9938$$

$$\text{Standard error of residuals } \sigma = 0.0745$$

$$\epsilon = 0.57852 - 0.20640 \sigma_3 + 0.07854 \sigma_1 - 0.01464 \sigma_3 \log N$$

$$- 0.00121 \sigma_1 \log N - 0.00408 \sigma_1 \sigma_3 + 0.03846 (\log N)^2$$

TABLE 5.5. COMPARISON OF TOTAL STRAIN FOR AASHO ROAD TEST AND  
TEXAS A&M UNIVERSITY TEST MATERIALS

Number of Applications	Total strain for AASHO Road Test material, %	Total strain for angular medium aggregate at A&M University test, %
100	0.21	0.15
1,000	0.41	0.6
10,000	1.08	1.0
100,000	4.4	1.3

TABLE 5.6. LOAD REPETITIONS AND DEFORMATION DATA FOR BASE MATERIAL IN % STRAIN E (FIG A2.8)

log N or psi % psi	2.0					3.0					4.0					5.0				
	10.0	20.0	30.0	40.0	50.0	10.0	20.0	30.0	40.0	50.0	10.0	20.0	30.0	40.0	50.0	10.0	20.0	30.0	40.0	50.0
1.0	1.2	1.7	2.1	2.5	3.1	1.3	1.8	2.2	2.6	3.2	1.4	2.0	2.3	2.7	3.3	1.5	2.2	2.4	2.8	3.5
3.0	0.5	1.0	1.6	2.0	2.5	0.6	1.1	1.7	2.1	2.6	0.7	1.2	1.8	2.2	2.7	0.8	1.33	1.9	2.3	2.8
5.0	0.2	0.5	0.9	1.4	1.8	0.3	0.6	1.0	1.5	1.9	0.4	0.7	1.1	1.6	2.0	0.5	0.8	1.2	1.7	2.1
7.0	0.05	0.2	0.5	0.8	1.2	0.1	0.3	0.6	0.9	1.3	0.15	0.4	0.7	1.0	1.5	0.2	0.5	0.8	1.1	1.7
9.0	0.01	0.05	0.2	0.5	0.9	0.04	0.1	0.3	0.6	1.0	0.07	0.15	0.4	0.7	1.1	0.1	0.2	0.5	0.8	1.2



TABLE 5.7. LOAD REPETITIONS AND DEFORMATION DATA FOR SUBBASE MATERIAL IN % STRAIN E (FIG A2.9)

$\log N$ $\sigma_1$ psi $\sigma_3$ psi	2.0					3.0					4.0					5.0				
	5.0	10.0	15.0	20.0	25.0	5.0	10.0	15.0	20.0	25.0	5.0	10.0	15.0	20.0	25.0	5.0	10.0	15.0	20.0	25.0
1.0	0.3	0.8	1.0	1.5	2.0	0.5	1.0	1.5	2.0	3.0	0.7	1.2	2.0	2.5	4.0	0.9	1.4	2.5	3.0	5.0
2.0	0.2	0.7	1.0	1.2	1.5	0.3	0.8	1.1	1.6	2.0	0.4	0.9	1.3	1.8	3.0	0.5	1.0	1.5	2.0	4.0
3.0	0.1	0.2	0.8	1.0	1.4	0.2	0.4	0.8	1.2	1.6	0.3	0.6	1.0	1.4	1.8	0.4	0.8	1.2	1.6	2.0
4.0	0.01	0.5	0.75	0.9	1.25	0.1	0.6	0.8	1.0	1.4	0.2	0.65	0.85	1.1	1.5	0.3	0.7	0.9	1.2	1.6
5.0	0.08	0.2	0.3	0.5	0.9	0.09	0.3	0.4	0.6	1.0	0.2	0.4	0.5	0.7	1.2	0.3	0.5	0.6	0.8	1.3

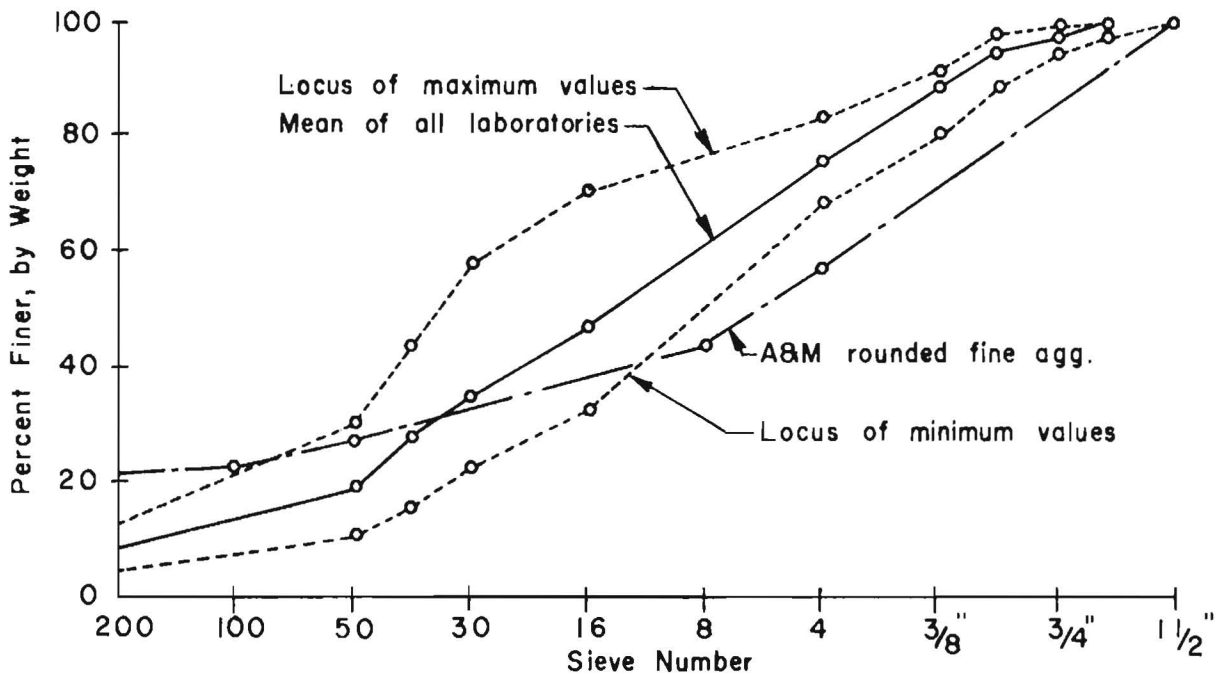


Fig 5.1. Grading curve for subbase.

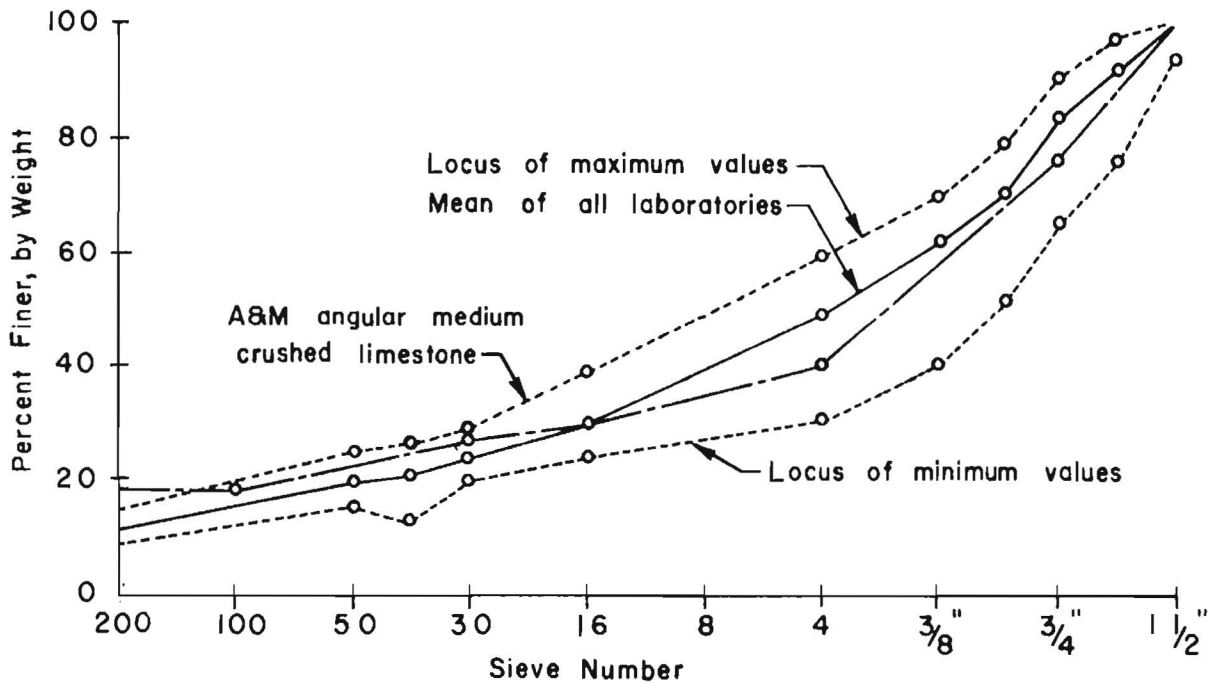


Fig 5.2. Grading curve for base.

- Notes: (1) Dotted and firm lines show the result of AASHO Road Test materials (Ref 66).  
 (2) Chain line shows the result of A&M materials (Ref 35, Figs 3.5 and 3.6).

$$\begin{aligned}
& - 0.00093 \sigma_1^2 - 0.00062 \log N \sigma_3^2 - 0.00292 (\log N)^3 \\
& + 0.00204 \sigma_3^3 + 0.0001 \sigma_1^3 - 0.004 \sigma_3^2 \sigma_1 + 0.00006 \sigma_1^2 \sigma_3 \\
& + 0.00046 \sigma_1 \sigma_3 \log N
\end{aligned} \tag{5.1}$$

Subbase Material:

$$\text{Correlation coefficient } R^2 = 0.9772$$

$$\text{Standard error of residuals } \sigma = 0.1442$$

$$\begin{aligned}
\epsilon = & -0.75465 + 0.25605 \log N + 0.17009 \sigma_1 - 0.14433 \log N \sigma_3 \\
& + 0.01187 \log N \sigma_1 + 0.01139 \sigma_1 \sigma_3 + 0.04947 \sigma_3^2 - 0.01132 \sigma_1^2 \\
& + 0.03340 \log N \sigma_3^2 + 0.00115 \log N \sigma_1^2 - 0.01885 \sigma_3^3 \\
& + 0.00025 \sigma_1^3 + 0.00367 \sigma_3^2 \sigma_1 - 0.00072 \sigma_1^2 \sigma_3 \\
& - 0.01018 \sigma_1 \sigma_3 \log N
\end{aligned} \tag{5.2}$$

where

$\sigma_3$  = radial or confining stress, psi;

$\epsilon$  = percent permanent strain;

$N$  = number of stress applications;

$\sigma_1$  = vertical stress, psi.

Each of the above equations is based on 100 observations. For an actual design problem, the designer will replace these equations with the actual properties obtained for the materials to be used.

## CHARACTERIZATION OF SUBGRADE "FINE GRAINED COHESIVE SOILS"

Elastic Constants

A resilient modulus  $M_R$  of the subgrade soil can be determined by measuring the resilient strain in a repeated load triaxial compression test. Since the  $M_R$  value is sensitive to many factors, as outlined in Chapter 4, the choice of a correct value is difficult. However, as detailed in Appendix 2, various approaches led to the selection of suitable values based on available information. Resilient modulus values at different moisture contents were studied and the monthly values adopted are shown in Table 5.8. Because Poisson's ratio is relatively insensitive in the layered analysis, a mean typical value of 0.5 for subgrade soils was adopted.

Stochastic Variation

No direct information was available for the stochastic variation of the  $M_R$  value. However, based on the general variations in other properties having a direct relationship with the  $M_R$  value, as outlined in Appendix 2, a standard deviation of about 25 percent of the mean value was adopted.

Deformation Properties Under Repeated Loading

The procedure developed in this dissertation requires stress-strain plots for various axial and confining pressure combinations. The AASHO Road Test subgrade soil was tested in the repeated load test (Ref 165) at a confining pressure of 3.5 psi for various axial stresses (Fig A2.10). The test was made at a moisture content of 15.3 percent. A variation in permanent deformation characteristic is observed due to variation in moisture content, but at the low stress levels encountered in the pavements this variation will be very small. For practical application of the method, repetitive load tests at various moisture contents expected in the field can be obtained for increased accuracy.

For AASHO Road Test subgrade soil, the repetitive load test curves are available only for 3.5 psi confining pressure. For the analysis, similar data are required for various confining pressures in the range expected in the analysis. To make use of the available information, it has been assumed that the total axial deformation is the same for the same deviator stress. Knowing the deviator stress  $(\sigma_1'' - \sigma_3'')$  in the actual pavement, total strain corresponding

TABLE 5.8. MONTHLY VALUES OF MODULUS OF RESILIENCE OF AASHO SUBGRADE MATERIAL

Month	$M_R$ , psi
January	6600
February	6600
March	6600
April	3600
May	4300
June	4900
July	5400
August	5800
September	6100
October	6200
November	6400
December	6400

to  $\sigma_1' - 3.5$  can be obtained from the curves developed for a confining pressure of 3.5 psi. Equivalent vertical stress  $\sigma_1'$  will be computed as follows:

$$\sigma_1' - 3.5 = \sigma_1'' - \sigma_3'' \quad (5.3)$$

where

$$\sigma_1'' - \sigma_3'' = \text{deviator stress in actual pavement,}$$

$$\sigma_1' = \text{equivalent vertical stress.}$$

Table 5.9 shows the values of the permanent strains vs stress applications for various axial stresses at a constant confining pressure of 3.5 psi.

#### Regression Model to Characterize Deformation-Repeated Load Characteristics

To utilize the available information a regression analysis was performed on the data shown in Table 5.9, and the following regression model was obtained.

For  $\sigma_3 = 3.5$ , psi in compression

The correlation coefficient  $R^2 = 0.99$ , and the standard error of residuals = 0.16 = 0.0016 in/in.

$$\begin{aligned} \epsilon = & 0.35461 \sigma_1 - 0.04064 \sigma_1 \log N - 0.06511 \sigma_1^2 + 0.00283 \sigma_1^3 \\ & + 0.00744 \sigma_1^2 \log N \end{aligned} \quad (5.4)$$

where symbols are as previously defined.

In the case of actual design problems, the user may replace this regression equation by the data obtained from the tests on actual subgrade material.

#### SUMMARY

In this chapter the characterization of the materials applicable to the proposed models is described with special reference to the AASHO Road Test materials. Summary of sampling and testing requirements are shown in Table 5.1. The elastic moduli of the AASHO Road Test materials are shown in Tables 5.2, 5.3, and 5.8, while the repeated load-deformation characteristics are described by Eqs 5.1, 5.2, and 5.4. For actual problems, the user may characterize the

TABLE 5.9. REPETITIVE LOAD AND DEFORMATION DATA OF AASHO ROAD TEST  
SUBGRADE MATERIAL (FIG A2.10)

Confining pressure  $\sigma_3 = 3.5$  psi

Moisture content = 15.3%

Axial stress $\sigma_1$	Number of stress repetitions N	Total strain % $\epsilon$
0.0	1	0.0
0.0	10	0.0
0.0	100	0.0
0.0	1,000	0.0
0.0	10,000	0.0
0.0	100,000	0.0
0.0	1,000,000	0.0
6.6	1	0.1
6.6	10	0.2
6.6	100	0.3
6.6	1,000	0.4
6.6	10,000	0.6
6.6	100,000	0.8
6.6	1,000,000	1.0
9.7	1	0.2
9.7	10	0.4
9.7	100	0.6
9.7	1,000	0.8
9.7	10,000	1.0
9.7	100,000	1.2
9.7	1,000,000	1.5
16.0	1	0.6
16.0	10	1.8
16.0	100	3.0
16.0	1,000	4.3
16.0	10,000	5.7
16.0	100,000	7.0

materials as described in this chapter and may replace the regression models (Eqs 5.1, 5.2, and 5.4) by the data obtained from the tests on actual materials.



## CHAPTER 6. USE OF ELASTIC THEORY AND LAYERED ANALYSIS IN THE DESIGN OF FLEXIBLE PAVEMENTS

The concept of linear elastic layered theory has been utilized in the design approach developed in this report. Thus, this chapter discusses the use of this theory in the proposed procedure for the design of flexible pavements.

### INTRODUCTION

The use of linear elastic theory and layered analysis in the design of flexible pavements is becoming more feasible because of the relative ease of solution with the present generation of computers. In the past, direct application of the results to pavement design was generally unsuccessful for the following factors (Ref 131):

- (1) complexity of solutions,
- (2) difficulty in isolating the particular cause of distresses affecting the pavement performance, and
- (3) lack of agreement between measured and predicted stress and strains.

The first factor has been partially eliminated as a problem by the development of computers. As far as the second factor, it is generally recognized that tensile strain in an asphaltic material is a major factor in determining the fatigue life, although any single theory based on elastic behavior of materials cannot account for all complexities and factors involved. Many discrepancies in the predicted and observed stresses and strains caused by the third factor are eliminated by a better understanding of material characterization and by more general methods of calculation. Thus, a rational design procedure is established in this report using linear elastic layered theory to calculate the tensile strains in the asphaltic concrete layers. The stresses and strains in the other layers are used to determine rut depth in a pavement system.

## BEHAVIOR OF IDEAL MATERIALS

In the previous two chapters, the behavior of real materials was described. In the remaining portion of this chapter, the behavior of ideal materials, required by the theory, is described.

The strain of an ideal elastic body and the strain rate of a viscous fluid are both proportional to stress and independent of time. The strain of an ideal elastic body is recoverable upon unloading, but this is not the case for an ideal viscous body (Ref 183).

A deformation is said to be anelastic (to have delayed elasticity) if it is time-dependent and completely recoverable. An ideal elastic body may be represented by a spring, and an ideal viscous fluid by a dash-pot. Any combination of spring and dash-pot is said to represent viscoelastic behavior. Certain combinations of springs and dash-pots give rise to anelastic behavior (Ref 183).

Plastic deformations may or may not be time-dependent. Aspects of linearity or nonlinearity aside, the main difference between viscous deformation and time-dependent plastic deformation is the irreversibility of the latter. If the direction of the load is reversed, a viscous deformation will be completely reversible, but a plastic deformation will not be reversible (Ref 183).

For low stresses, asphaltic concrete and other pavement materials may behave in a linearly elastic fashion, while at higher loads, the stress-strain curve is nonlinear. Ideally, the pavement material may exhibit one or all of the following major types of deformation behavior (Ref 183).

<u>Type of Deformation</u>	<u>Behavior for Instantaneous Deformation</u>	<u>Behavior for Time-Dependent Deformation</u>
Recoverable	Elasticity	Anelasticity
Irrecoverable	Plasticity	Viscosity

In addition to the above factors, ideal materials are considered homogeneous and isotropic. For homogeneous materials, the elastic properties are identical throughout the material and in isotropic materials, the elastic properties are identical in all directions at any point within the material.

### Elastic Materials

Elastic theories have been used for pavement within granular and fine grained materials, not because they are ideal elastic bodies but due to the availability of solutions. However, properly used, the theories give solutions which are accurate. The first and most widely known theory is that of Boussinesq (Ref 8), which deals with stresses in a homogeneous, isotropic, linearly elastic solid of semi-infinite extent subjected to a load applied normally to the surface. This theory is not fully utilized, since pavements with their layered structure do not satisfy the requirements of homogeneity. More realistic are the two and three-layered solutions developed by Burmister (Ref 14). With the advent of computers, solutions for up to 15 layers have been developed (Ref 116). Many solutions have been published for a layered homogeneous elastic solid loaded by a uniform vertical load over a circular area. These solutions are tabulated by Seed et al (Ref 164) and Morgan and Scala (Ref 131).

In an isotropic medium, only Young's modulus  $E$  and Poisson's ratio  $\mu$  are required. In the Boussinesq solution, all stress components are independent of  $E$  and only the radial and tangential stresses are affected by the value of  $\mu$ . In layered elastic systems, the stresses are influenced principally by modular ratios and not by absolute values. Displacements are influenced by the magnitude of  $E$  in a single layer and by the modulus ratios, as well as magnitudes of  $E$  in the multilayer system. Poisson's ratio also influences displacements but in a nonproportional fashion. The influence of  $\mu$  is quite insignificant and has been largely ignored.

The difficulty in the use of the anisotropic solution lies in the difficulty in determining some of the parameters of real materials. This determination may not be needed for the accuracy required from a practical standpoint. The effect of nonhomogeneity at various depths of granular materials, where stiffness changes with confining pressure, has been considered by various authors and was discussed in Chapter 4.

### Viscoelastic Materials

The stiffness of asphaltic concrete varies with temperature and rate of loading. To account for this, the viscoelasticity theory should be applied to solve for stress and strain. However, because of the additional complexity involved in assuming viscoelastic behavior, much asphalt pavement analysis has

been carried out using elastic theory. Complications of viscoelasticity in asphaltic concrete can be avoided by accounting for the influence of loading rate and temperature on asphalt stiffness by testing samples at the same rate of loading and temperature as observed in the field. Various methods of computing the stiffness have been developed, as explained in Chapter 5.

Although pavement and subgrade materials ideally exhibit viscoelastic behavior, the extent is considerably less than for asphalt concrete. Therefore, for these materials the complications of viscoelastic behavior can be avoided by the proper choice of testing technique.

#### COMPARISON OF PREDICTED AND OBSERVED BEHAVIOR

Ultimate decisions about the applicability of elastic theory to pavements can be based on the comparison of the following measured and predicted characteristics:

- (1) stress in single and multilayered systems,
- (2) vertical strains and deflections, and
- (3) horizontal strains in asphalt layer.

##### Stress in Single and Multilayered System

The stress estimation from strain measurement has been reported to be one of the most straightforward ways to evaluate the usefulness of elastic theory (Ref 131). These measurements have been reported for both uniformly prepared sand masses and fine grained soils (Ref 131). Considering the results of measurement in both single and multilayered systems, the following conclusions were derived (Ref 131).

- (1) Vertical stress distributions for the appropriate boundary conditions are given with reasonable accuracy by both the Boussinesq single layer and the Burmister multilayered theories. For two-layered systems, the modular ratio used for unbound bases is probably only two to three, and the difference between the stresses predicted by the two theories is small. Variations from the assumed conditions of isotropy and homogeneity are unlikely to influence the vertical stress significantly.
- (2) Radial stresses, except close to the surface in single-layered systems, are underestimated by both the single and multilayered theories. It has been suggested that better agreement would be obtained if the consideration of proper anisotropy of the material in the horizontal and vertical directions is taken.

### Vertical Strains and Deflections

Direct application of elastic theory may not give very accurate results. However, approximate approaches based on the elastic theory have been developed and are discussed in Chapter 10.

### Horizontal Tensile Strain in Asphaltic Material

The horizontal tensile strain at the base of an asphaltic layer has been widely accepted as the criterion for fatigue failure of these materials. The comparison between observed and predicted values from layered theory shows reasonable agreement. The strain values at the base of the layer are given most accurately and these are the ones which are used in fatigue design.

### SUMMARY

The discussion in this chapter indicates that for all practical purposes, the use of linear elastic theory in pavement design gives solutions which are accurate enough from a practical standpoint. Complexity of solutions has been partially eliminated by the development of computers, making the use of the theory more feasible. A rational design procedure can be established by the use of stress and strain, which are calculated by this theory.

PART III

DEVELOPMENT OF MODELS

## CHAPTER 7. DEVELOPMENT OF DISTRESS MODELS

The term failure as applied in the design of many engineering structures cannot be used for pavement systems. For example, a pavement could be considered to have failed according to structural design standards, such as appearance of cracks, but may still be capable of performing at a reduced level. A pavement should be designed and evaluated in terms of the level of service or performance it can provide. The categories of distress manifestations affecting the performance of a pavement system were introduced in Chapter 3. In this chapter, the distress index model for computing the pavement performance is developed.

There is a complex interrelationship between pavement component materials, pavement behavior, and performance of the pavement. As defined in NCHRP 1-10 (Ref 78), behavior is the reaction or response of a pavement to load, environment, and other inputs. Performance is a measure of the accumulated service provided by a facility and is a direct function of the history of the present serviceability index of the pavement according to the AASHO concept of PSI, as discussed in Appendix 7 of this report.

Distress mechanisms have been defined (Ref 78) as responses which lead to some form of distress when carried to an extreme limit. Figure 3.1 gives the three categories of pavement distress model which are limiting responses. In general, the distress index (quantification of the limiting responses) is expressed as some function of the measure of the limiting responses in space and time, the limiting responses being the function of distress mechanism, shown in Fig 3.1. When the distress index exceeds some acceptable level, the pavement system is considered to have failed.

### IDEAL DISTRESS INDEX MODEL

A conceptual distress index can be expressed as follows (Ref 78):

$$\underline{DI}(\underline{x}, t) = \int_{s=0}^{s=t} [\underline{C}(\underline{x}, s), \underline{S}(\underline{x}, s), \underline{D}(\underline{x}, s)] \underline{x}, t] \quad (7.1)$$

where

$t$  = time;

$\underline{x}$  = position vector of a point referred to a coordinate system;

$\underline{DI}(\underline{x},t)$  = distress index, a matrix function of space and time;

$\underline{C}(\underline{x},t)$  = measure of fracture, a matrix function of space and time;

$\underline{S}(\underline{x},t)$  = measure of distortion, a matrix function of space and time;

$\underline{D}(\underline{x},t)$  = measure of disintegration, a matrix function of space and time.

The distress index is a function of the history of the variable shown from time zero to current time  $t$ . In a systems framework, the parameters in Eq 7.1 must be quantified from the input parameters. The three modes of distress may be expressed as a function of load, environment, construction, maintenance, and structural variables in space and time.

For fracture:

$\underline{C}(\underline{x},t)$  is a function of load, environment, construction, maintenance, and structural variables, space and time; (7.2)

For distortion:

$\underline{S}(\underline{x},t)$  is a function of load, environment, construction, maintenance, and structural variables, space and time; (7.3)

For disintegration:

$\underline{D}(\underline{x},t)$  is a function of load, environment, construction, maintenance, and structural variables, space and time. (7.4)

The substitution of Eqs 7.2, 7.3, and 7.4 for fracture, distortion, and disintegration into Eq 7.1 gives a measure of a distress index. Based on the riding quality, economics, and safety as required in particular circumstances, acceptable limits to the distress index can be assigned. These limits define the failure of the pavement, thus giving a criterion for pavement design.



## DEVELOPMENT OF DISTRESS INDEX MODEL

Development of an ideal distress index model is a complex problem; however, the AASHO Road Test concept of present serviceability index is recognized as the best to-date effort in this direction. The present serviceability index equation developed in the AASHO Road Test (Ref 70) is a widely accepted statistically derived regression equation which relates the distress manifestations to the present level of service. It has been found that in the view of highway users, the distress index can be very well explained and correlated in terms of

- (1) slope variance  $\overline{SV}$  (Appendix 7), which can be related to disintegration and distortion;
- (2) rut depth  $\overline{RD}$ , which can be related to distortion;
- (3) area of cracking  $C$  per thousand square feet, which is related to fracture; and
- (4) area of patching  $P$  per thousand square feet, which is related to fracture, disintegration, and distortion.

At the AASHO Road Test, these four factors were measured and the distress index or PSI of the sections was calculated and defined according to the following equation for flexible pavements (Ref 70, Appendix F):

$$PSI = 5.0 - 1.9 \log (1 + \overline{SV}) - 1.375\overline{RD}^2 - 0.01\sqrt{C + P} \quad (7.5)$$

The pavement design models were developed statistically, correlating PSI with axle load, repetitions of load, and the design variables (depths of various layers).

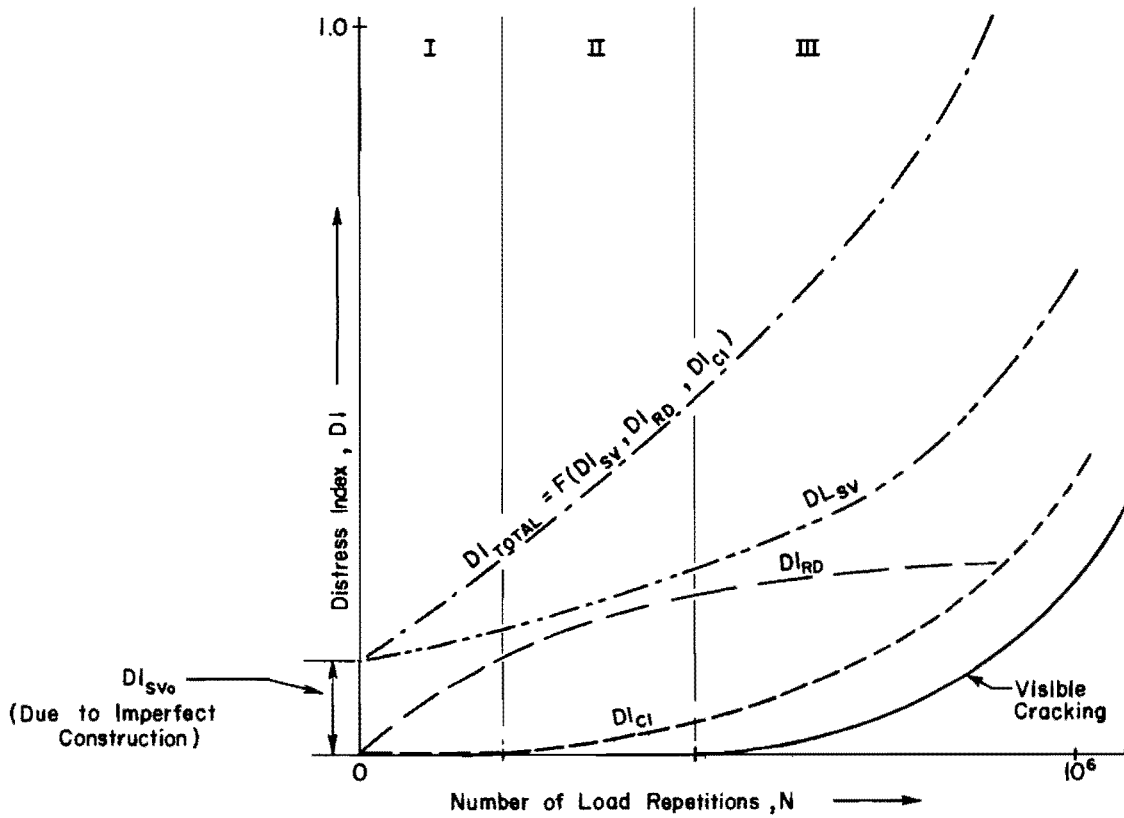
A distress index curve is shown in Fig 7.1. An increase in load repetitions will increase the distress in the pavement. The form of distress development is shown by curves for distress indices for cracking ( $DI_{CI}$ ), rut depth ( $DI_{RD}$ ), and roughness or slope variance ( $DI_{SV}$ ). The cracking index curve shows that although there is cracking at the beginning of Stage III, theoretically actual distress in the pavement due to cracking starts at the beginning of Stage II. Once the visible cracking starts at Stage III, this effect tends to progress rapidly. The pavement has some roughness due to imperfect construction even in the beginning, and the roughness increases further with the number of load repetitions, as shown. The rut depth distress due to permanent

$DI_{CI}$  = Distress Index due to Cracking

$DI_{RD}$  = Distress Index due to Rut Depth

$DI_{SV}$  = Distress Index due to Roughness or Slope Variance

$DI_{TOTAL}$  = Total Distress Index



Note: For three stages I, II, & III shown above see Chapter H.

Fig 7.1. Distress index curves for flexible pavements.

deformation in pavement layers will progress at a decreasing rate. The total distress index curve, as shown, is the total effect of all three distress indices.

$$DI_{TOTAL} = f(DI_{SV}, DI_{RD}, DI_{CI}) \quad (7.6)$$

#### QUANTIFICATION OF DISTRESS INDEX MODEL

Examples of various distress mechanisms responsible for distress in pavement are shown in Fig 3.1. For an ideal pavement system design model, all possible distress mechanisms should be taken into consideration. However, for a real-world situation, this is not always possible and a compromise, based on the state-of-the-art, resources, and time, is necessary. Due to the limited scope of the present analysis, a direct consideration of shrinkage and slippage has not been possible. Because the AASHO Road Test was basically a fatigue test of short duration with no apparent effects of swelling clays and major temperature effects, these factors were not considered. The rupture distress mechanism has been computed in terms of the cracking index  $CI$ , and distortion in terms of slope variance  $\overline{SV}$  is correlated to  $CI$ . Disintegration is one other factor which is not being considered directly in the present analysis.

In the AASHO Road Test, cracking and patching were the measured limiting responses. The measure of this response has been obtained theoretically in this report, based on fatigue and stochastic principles. This has been defined as the cracking index  $CI$ . Development of this model is explained in Chapter 8. The distress due to rut depth is represented as the rut depth index  $RDI$ , and the model is developed in Chapter 9. The distress due to slope variance, which is the measure of variation of a roadway surface from a desirable profile, is represented in the form of roughness index  $RI$ . The roughness index model is detailed in Chapter 10. The verification of the models has been performed with the AASHO Road Test data and is included in Chapter 11.

Based on the above discussion and the performance concept of the AASHO Road Test, the present serviceability index of a flexible pavement can be represented mainly as a function of the cracking index, roughness index, and rut depth index at any time during the pavement performance. If the above

three factors are known, an estimate of the pavement performance in terms of PSI can be made.

Mathematically,

$$\text{PSI} = f(\text{DI}) = f(\text{CI}, \text{RI}, \text{RDI}) \quad (7.7)$$

where

PSI = present serviceability index,

CI = cracking index,

RI = roughness index, and

RDI = rut depth index.

The steps involved in the development of an actual distress index model from Eq 7.7 are shown in a flow chart in Fig 7.2. This procedure requires the availability of some performance data. Since the procedure developed in this report utilizes the AASHO Road Test concept of performance, the present serviceability index equation, Eq 7.5, is adopted for the distress index model for the present analysis.

#### VERIFICATION OF DISTRESS INDEX MODELS

Because the distress manifestations CI , RI , and RDI are considered a very good measure of overall distress, the need to express each as a function of some measurable and well-established material behavior properties, pavement components, load factors, and environment factors in a working model is apparent. If such models are developed, then pavements can be designed rationally and their performance predicted in any arbitrary set of conditions. In the following chapters, such an effort is outlined and the development of the models explained.

The AASHO Road Test is an excellent source of performance data to verify the models developed in Chapters 8, 9, and 10, the measurements for which were obtained under different conditions. Therefore, the AASHO data have been fully utilized to verify the predicted performance curves of the developed models. Because of the extent of the AASHO Road Test, verification and good reproduction of the AASHO data will give confidence in the use of the developed models.

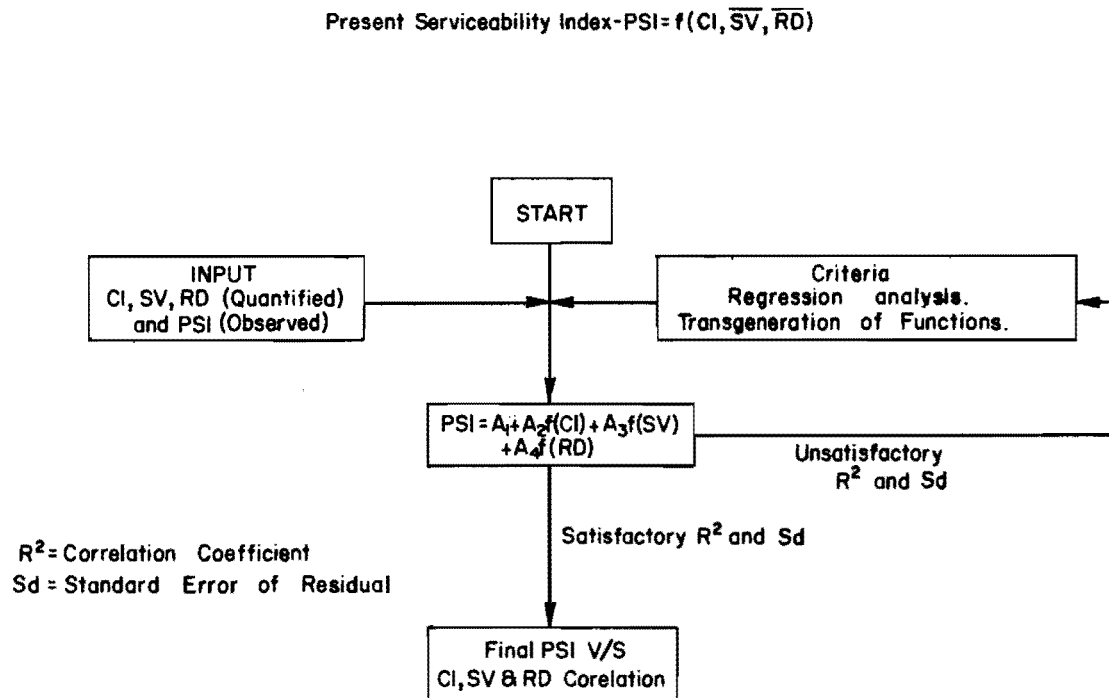


Fig 7.2. Flow chart for development of performance model.

Verification of the distress index models developed in this report is detailed in Chapter 11. A typical pavement performance curve is shown in Fig A7.1 of Appendix 7. An attempt has been made in this research to reproduce the observed performance curves of the AASHO Road Test.

## CHAPTER 8. DEVELOPMENT OF CRACKING INDEX MODEL

In this chapter, a model for predicting the cracking index (CI) is developed, using the fatigue theory and Miner's hypothesis (Chapter 4), and stochastic concepts.

### STOCHASTIC CONCEPTS APPLIED TO CRACKING INDEX IN FLEXIBLE PAVEMENTS

Generally, cracking in a pavement structure is considered to be a deterministic phenomenon occurring when the stress is greater than the strength. Both stress and strength in a pavement are subject to stochastic variations, which can be approximated by a continuous normal distribution (Ref 116). The fatigue phenomenon occurs in a pavement, following a predictable relationship between repetition of a load, stress or strain, and material properties. The modulus of elasticity and Poisson's ratio are subject to variations in both space and time, and generally variations of these properties could be approximated by a normal distribution. Therefore, statistical methods and probability theory are required to predict the amount of cracking or the cracking index. In the present model, stochastic concepts for variations in the material properties are applied only to the space variation. The time variation is considered in terms of monthly variations. Fatigue life tests for asphaltic concrete must be made to determine the distribution of fatigue life. The mean value and standard deviation of fatigue life and its relationship to induced strains must be obtained. As already discussed in Chapter 4, fatigue life variations can be approximated by a log normal distribution. Since in fatigue life (the number of stress repetitions  $N$ ), a log normal distribution is applicable, it is reasonable to assume that the same is applicable to the ratio of actual to theoretical stress applications  $\frac{n}{N}$ .

According to Miner's hypothesis for no distress, the cumulative damage must be less than one, as given by

$$\sum_{j=0}^t \sum_{N_j} \frac{n_j}{N_j} \leq 1.0 \quad (8.1)$$

where

$n_j$  = the actual number of load applications of level  $j$ ,

$N_j$  = the number of load applications of level  $j$  which will cause failure in simple loading.

Using these values in a statistical analysis, the probability  $p$  of distress for cumulative damage being more than 1.0 in a given situation may be computed. In the case of a given area of roadway, it may be said that approximately  $p$  percent of the roadway area would experience cracking distress under the given conditions (Refs 78 and 116). Thus, the cracking index is calculated as the probability of  $\sum_0^t \sum_0^j \frac{n_j}{N_j}$  exceeding one. This probability is represented by the area  $A$  beyond  $\log 1.0$  value of abscissa of a normal distribution curve, as shown in Fig 8.1. The cracking index represents the distress in a pavement at any time, in square feet of cracking per thousand square feet. Thus,

$$CI(x,t) = P\left(\sum_0^t \sum_0^j \frac{n_j}{N_j} > 1.0\right) \times 1000 \quad (8.2)$$

where

$$\begin{aligned} P\left(\sum_0^t \sum_0^j \frac{n_j}{N_j} > 1.0\right) &= \text{probability of total cumulative damage at} \\ &\text{any time for all load groups being more than} \\ &\text{one,} \\ &= \text{area } A \text{ (Fig 8.1).} \end{aligned}$$

Based on a normal distribution curve, this can be determined from the following equations:

$$\log (D)_\alpha = \log (D)_m + K \log \sigma_D \quad (8.3)$$

and



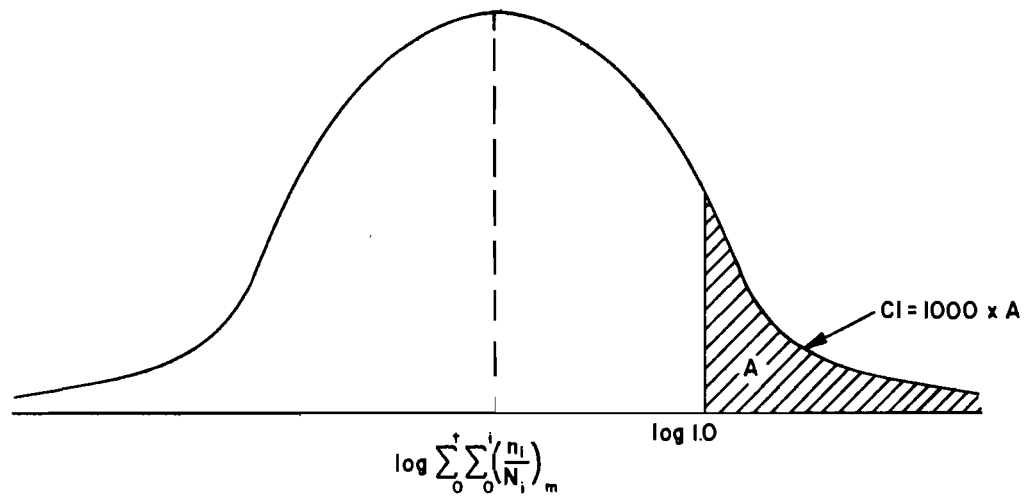


Fig 8.1. Log normal distribution curve for  $\sum_0^{\dagger} \sum_0^i \frac{n_i}{N_i}$

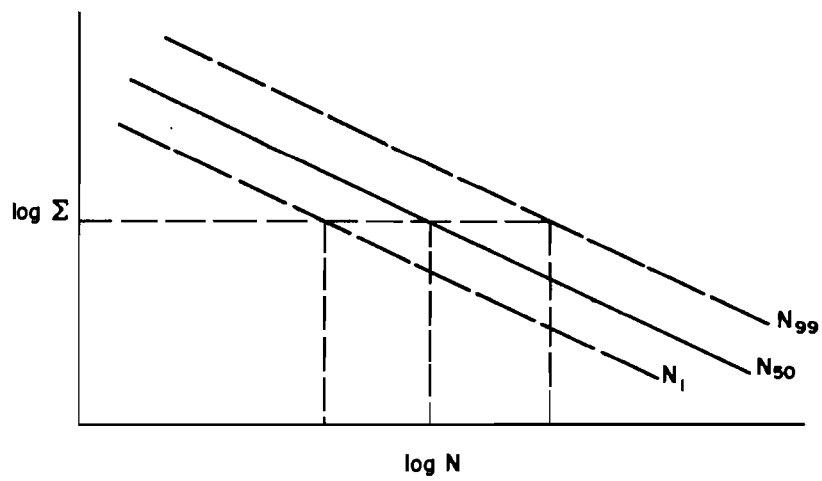


Fig 8.2. Asphaltic concrete fatigue curve.

$$\log (1.0) = \log (D)_m + K_{CI} \log \sigma_D \quad (8.4)$$

where

$$(D)_\alpha = \text{total damage at a confidence level } \alpha = \sum_0^t \sum_0^j \left( \frac{n_j}{N_j} \right)_\alpha,$$

$$(D)_m = \text{mean total damage} = \sum_0^t \sum_0^j \left( \frac{n_j}{N_j} \right)_m,$$

$K$  = normal curve parameter corresponding to confidence level  $\alpha$ ,

$\sigma_D$  = standard deviation for damage,

$K_{CI}$  = normal curve parameter for  $\sigma_D$  corresponding to other parameters in Eq 8.4,

$t$  = time,

$x$  = position vector of a point referred to in a coordinate system.

From the computed values of other parameters, the value of  $\log \sigma_D$  can be calculated from Eq 8.3. Then, from Eq 8.4,  $K_{CI}$  is computed. From the normal tables, the corresponding probability, that is, the area under the normal distribution curve  $A$  in Fig 8.1, is obtained. This area  $A$  when multiplied by 1000 gives the cracking index.

In statistical terms, the modulus values of various layers in a pavement may be considered as random variables and can be treated as independent factors. With the special case of statistical independence, the probability of the modulus values of several layers occurring simultaneously is equal to the product of the probability of each occurring independently. Thus, the overall probability is

$$\alpha = \prod_{i=1}^l \alpha_i \quad (8.5)$$

where

$\alpha$  = the overall confidence level for modulus values in a pavement structure,

$\alpha_i$  = the confidence level of the modulus value in the  $i^{\text{th}}$  layer,

$l$  = number of layers in a pavement structure.

The stress and strain caused by a wheel load in a pavement structure, due to variation in modulus values and variation in the fatigue life of surface layer materials, are considered as mutually exclusive. Thus, the probability of the alternative events is equal to the sum of each occurring alone. The overall probability in this case is given by

$$\alpha_T = \alpha + \alpha_N \quad (8.6)$$

where

$\alpha_T$  = total confidence level for damage or failure load repetitions,

$\alpha$  = overall confidence defined by Eq 8.5,

$\alpha_N$  = confidence level assumed for calculation of fatigue life.

#### QUANTIFICATION OF CRACKING INDEX

The cracking index is calculated for a particular pavement on the basis of its structural components, expected traffic, the period for which the facility will be used, fatigue behavior, and stochastic variations in the material properties. Mathematically, this can be represented as

$$CI(x,t) = f[\sigma_j(x,t), \epsilon_j - N_j(x,t), n_j(t)] \quad (8.7)$$

where

$$\sigma_j(x,t) = f[E_i(x,t), \mu_i(x,t), D_i(x,t), W_j] \quad (8.8)$$

and

$[\epsilon_j - N_j(x,t)]$  = the asphaltic concrete fatigue curve (Fig 8.2).

From the equation of the asphaltic concrete fatigue curve, the mean fatigue life at any time  $t$  is given by

$$N_j(t) = A \left( \frac{1}{\epsilon_j(t)} \right)^\beta \quad (8.9)$$

or

$$\log N_j(t) = \log A + \beta \log \left( \frac{1}{\epsilon_j(t)} \right) \quad (8.10)$$

Assuming that the asphaltic concrete fatigue life variation in space  $x$  at any time  $t$  can be approximated by a log-normal distribution

$$\log N(x,t) = \log N_j(t) \pm K \log \sigma_N \quad (8.11)$$

Combining Eqs 8.10 and 8.11 for both space and time,

$$\log N_j(x,t) = \log A + \beta \log \left( \frac{1}{\epsilon_j(t)} \right) \pm K \log \sigma_N \quad (8.12)$$

where

- CI = cracking index of the surface material, measured in square feet per thousand square foot;
- $\sigma_j$  = stress in the surface material of level  $j$ , in psi units;
- $\epsilon_j$  = flexural tensile strain in the surface material of level  $j$ , in inches per inch;
- $N_j$  = the number of load applications of level  $j$  to cause failure;

- $n_j$  = the number of actual load applications of level  $j$  ;  
 $E_i$  = elastic modulus of the  $i^{\text{th}}$  layer in a pavement structure, in psi;  
 $\mu_i$  = Poisson's ratio of the  $i^{\text{th}}$  layer in a pavement structure;  
 $D_i$  = depth of the  $i^{\text{th}}$  layer in a pavement structure;  
 $W_j$  = applied wheel load of level  $j$  on the pavement structure;  
 $A$  = constant of asphaltic concrete fatigue equation;  
 $\beta$  = constant of asphaltic concrete fatigue equation;  
 $K$  = normal distribution curve parameter;  
 $\sigma_N$  = standard deviation of fatigue curve;  
 $(x,t)$  = function of space and time.

### Modulus of Elasticity

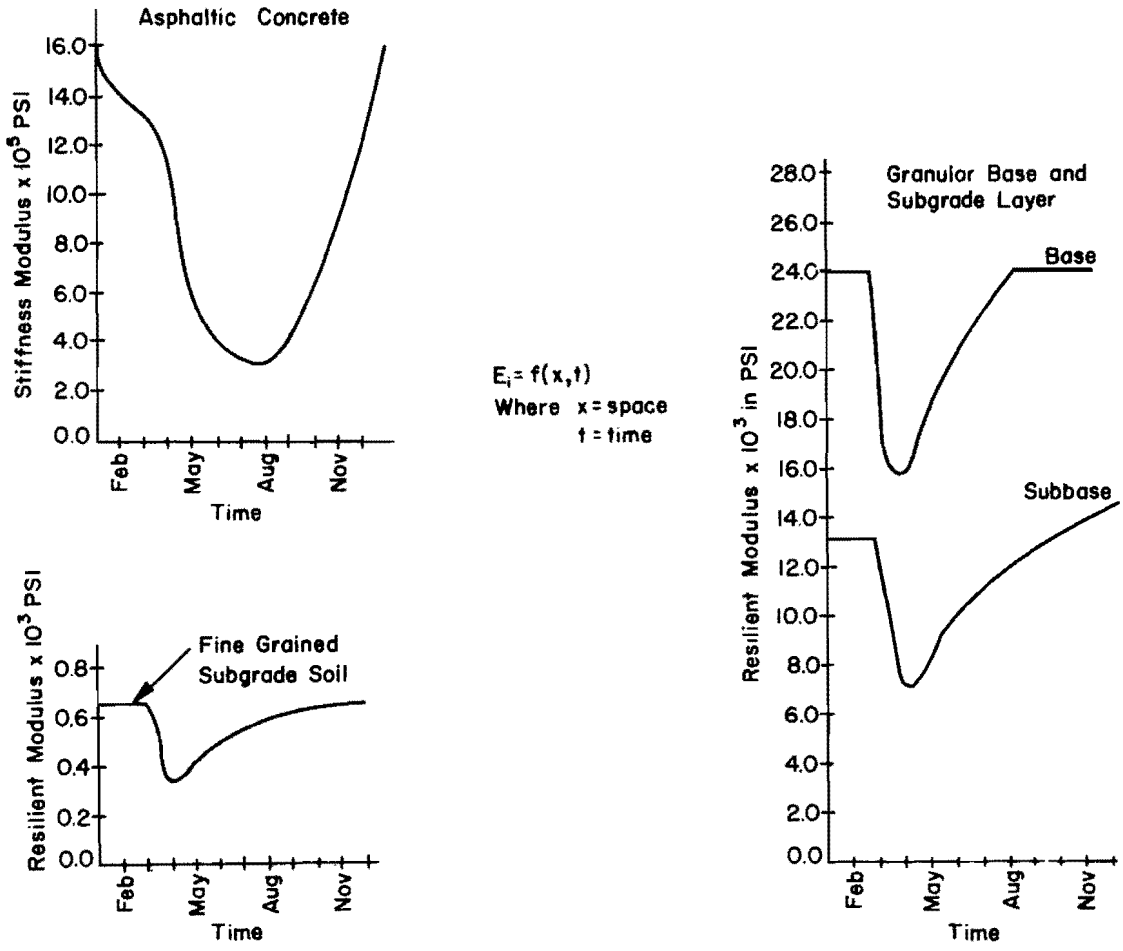
For the elastic modulus values of the materials used in a pavement, the time variation is considered in terms of monthly variations. As explained in Chapter 5, the values of the modulus depend upon many factors and these values vary with time. Actually, the smaller the time interval considered, the better the simulation. However, to limit the computation work, a monthly variation was considered reasonable for the development of the cracking index model

$$E_i(t) = f[E_{i1}, E_{i2}, \dots, \dots, E_{i11}, E_{i12}] \quad (8.13)$$

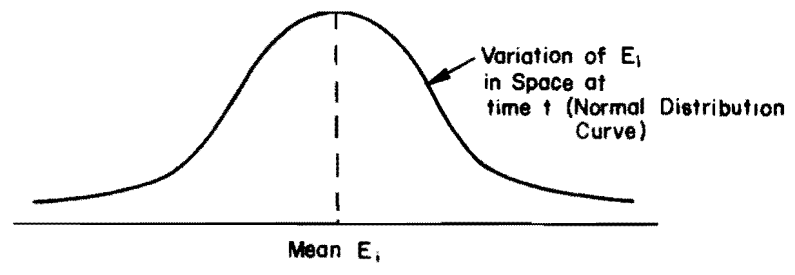
where

- $E_{i1}$  = average elastic modulus value of the  $i^{\text{th}}$  layer material for January,  
 $E_{i2}$  = average elastic modulus value of the  $i^{\text{th}}$  layer material for February, etc.

Typical variations in monthly modulus values for pavement layers of AASHO Road Test sections are included in Chapter 5 and shown in Fig 8.3.



(a) Variation in time.



(b) Variation in space.

Fig 8.3. Conceptual diagram showing variation of modulus  $E_i$  in both space and time.

For the space variation, it is assumed that the modulus values are normally distributed and variation can be represented by a normal distribution curve (Fig 8.3). The density function of such a distribution in space is given by

$$f(E_i) = \frac{1}{\sigma_{Ei} \sqrt{2\pi}} \cdot e^{-\frac{(E_i - \bar{E}_i)^2}{2\sigma_{Ei}^2}} \quad (8.14)$$

and, based on Eq 8.14, the value of elastic modulus in space  $E_i(x)$  is given by

$$E_i(x) = \bar{E}_i \pm K \cdot \sigma_{Ei} \quad (8.15)$$

where

$\bar{E}_i$  = arithmetic mean of the distribution,

$\sigma_{Ei}$  = standard deviation of modulus values,

$\pi$  = constant, and

$e$  = constant.

To combine space and time variations, the final value of the modulus in both space and time is given by

$$E_i(x,t) = \bar{E}_{it} \pm K \cdot \sigma_{Eit} \quad (8.16)$$

### Poisson's Ratio

In a sensitivity study, Buttler (Ref 15) found that magnitudes of the strain values in the surface layer calculated by the layered program are not significantly affected by variations in the Poisson's ratio. Therefore, this parameter is taken as a constant for each material in these developments.

$$\mu_i(x,t) = \mu_i \quad (8.17)$$

### Layer Thickness

In a real-world situation, the thickness of any layer varies in both space and time. With the best construction control, small variations in thicknesses in space cannot be avoided. The load, environment, and variation in material properties can cause variation in this parameter with time. However, the variations in the layer thickness are not considered of much significance in these developments. Therefore,

$$D_i(x,t) = D_i \quad (8.18)$$

where

$D_i$  = thickness of the  $i^{\text{th}}$  layer.

### Applied Load

The effect of different wheel loads is considered by summation of the damage caused by each load group in Miner's hypothesis.

#### PROCEDURE FOR MODELING THE CRACKING INDEX

The flow chart for modeling the cracking index in a typical pavement structure (Fig 8.4) is shown in Fig 8.5. The various steps required are

- (1) From the given monthly values of  $\bar{E}_i$ ,  $\sigma_{Ei}$ , and the assumed confidence level  $\alpha_i$ , calculate the  $E_i(x,t)$  values of materials with Eq 8.16.
- (2) Use the layered program and input  $E_i(x,t)$  from step 1 and  $\mu_i$ , axle load, tire pressure, and layer thicknesses to compute the tangential strains at the bottom of the surface layer.
- (3) Calculate the overall confidence level  $\alpha$  from Eq 8.5.
- (4) From Eqs 8.10 and 8.11, calculate the theoretical values of  $N_j$ , both the mean and at some confidence level, considered.
- (5) Calculate the overall confidence level for damage from Eq 8.6.

- (6) From the given values of  $n_j$ , calculate cumulative damage  $\sum_0^t \sum_0^j \frac{n_j}{N_j}$

for each month and for each load group. In the process, the mean, as well as the value at a certain confidence level, have been calculated.



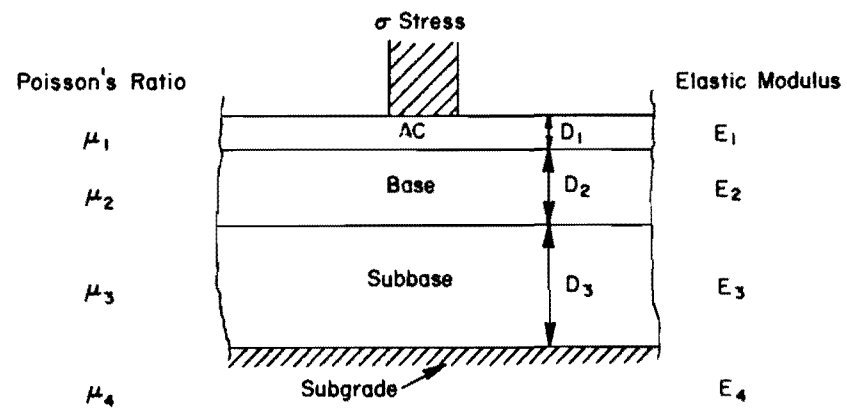


Fig 8.4. Typical flexible pavement section.

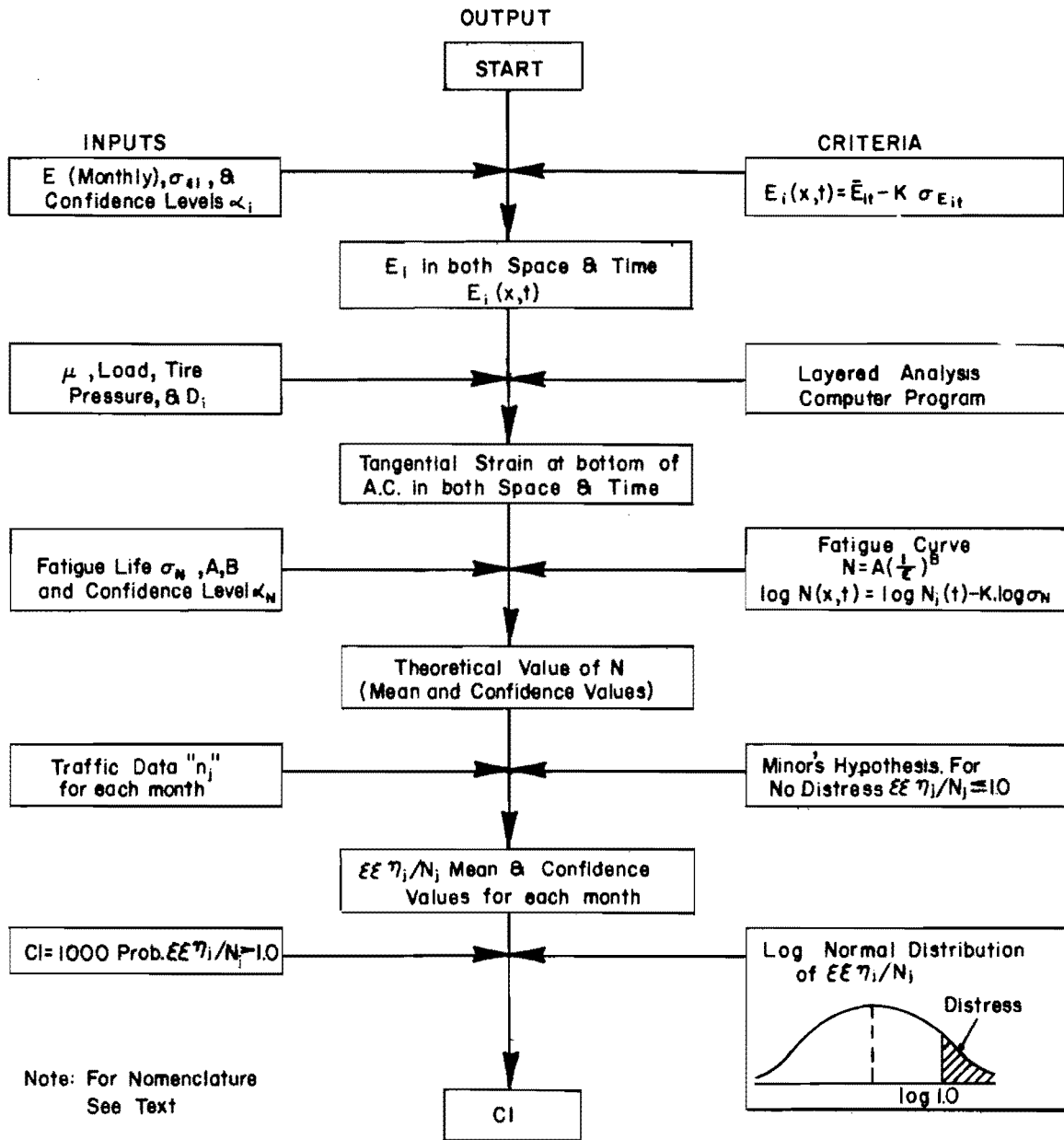


Fig 8.5. Flow chart for modeling the cracking index.

- (7) Calculate the cracking index, as discussed earlier, from Eqs 8.2, 8.3, and 8.4.

#### COMPUTER PROGRAM

As can be seen, it is difficult to make all the necessary calculations by hand. Therefore, a computer program was developed to calculate the final cumulative cracking index values every month. The flow chart of the computer program is shown in Fig A4.1. The computer program follows the steps shown in Fig 8.5 in calculating the cracking index in a pavement structure. The input to the program consists of the parameters listed in the boxes on the left-hand side of this figure. The middle boxes show the sequence of the output based on the criteria in the right-hand boxes and the corresponding input. This computer program is written for the CDC 6600 computer in FORTRAN language. This computer program can be used directly for the design of flexible pavements and can also be included in the pavement system design computer program previously developed for the Texas Highway Department. However, efforts to make this computer program more efficient should continue.

## CHAPTER 9. DEVELOPMENT OF RUT DEPTH INDEX MODEL

In this chapter, a model for predicting the rut depth index is presented. The vertical and radial stresses in a pavement system are computed from the layered analysis. The repeated load-deformation characteristics of the materials under triaxial testing are used. The rut depth may be represented as a permanent portion of the total deformation in a pavement structure due to repetition of loads. The deformation computations are made on the basis of mean values of the parameters, without considering the stochastic variations in space.

### QUANTIFICATION OF RUT DEPTH INDEX MODEL

Figure 9.1 outlines the procedure developed to compute rut depth in a pavement structure. The rut depth is calculated in terms of permanent deformation in different layers due to repeated loading. The vertical deformation in an asphaltic concrete layer is very small relative to other layers and thus is not considered. The total deformation consists of the sum of the deformations in all the layers below the surface layer. Mathematically, rut depth in the pavement is represented as:

$$\text{Rut depth } \overline{RD}(t) = f\left[\sigma_{ij}(t), (\epsilon - n)_i, n_j(t)\right] \quad (9.1)$$

where

$$\sigma_{ij}(t) = f\left[E_i(t), \mu_i(t), D_i(t), W_j\right]; \quad (9.2)$$

$(t)$  = function of time;

$\sigma_{ij}$  = vertical and confining stresses in the  $i^{\text{th}}$  layer due to applied load  $W_j$ , in psi;

$$\overline{RD} = f [\text{applied load } , \eta, E_i, \mu_i, \text{ repeated load v/s deformation curve}]$$

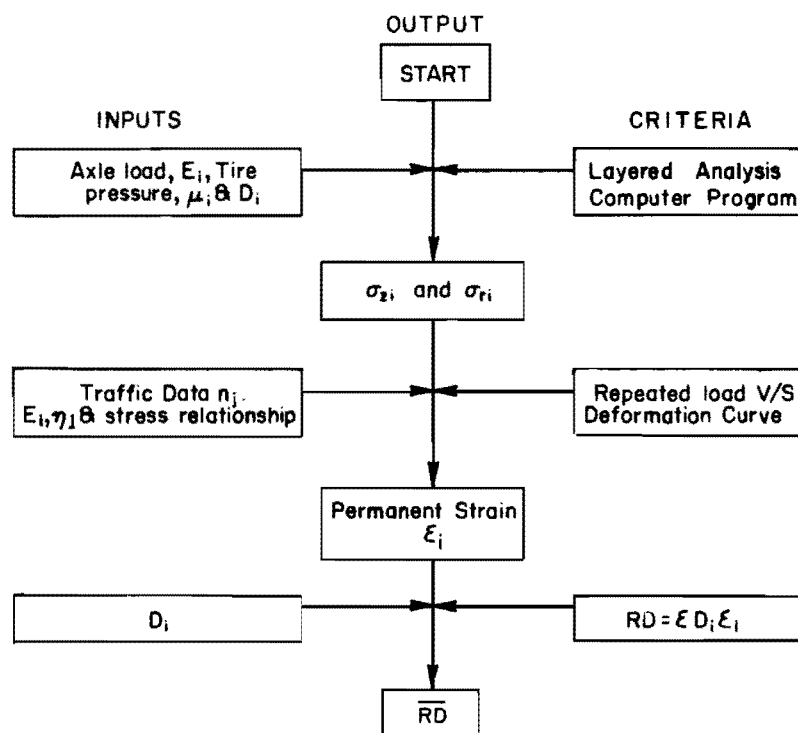


Fig 9.1. Flow chart for quantification of rut depth index.

- $n_j$  = number of load applications of level  $j$  ;  
 $\mu_i$  = Poisson's ratio of the  $i^{\text{th}}$  layer;  
 $E_i$  = elastic modulus of the  $i^{\text{th}}$  layer, in psi;  
 $D_i$  = depths of the  $i^{\text{th}}$  layer, in inches;  
 $(\epsilon - n)_i$  = repeated load deformation curve for the  $i^{\text{th}}$  layer;  
 $\epsilon_i$  = permanent vertical strain in the  $i^{\text{th}}$  layer.

### Load Deformation Curves

In triaxial loading, the permanent deformation of a particular layer depends upon the number of load repetitions and vertical and confining stresses. Load deformation curves and regression equations developed from these curves, required to calculate the permanent deformation in various layers, were discussed in Chapter 5. These curves and regression equations give an estimate of the permanent strain and deformation in each layer in terms of vertical stress, confining stress, and number of stress repetitions.

### Vertical and Confining Stresses

Vertical and confining stresses are considered in two categories:

- (1) Those due to wheel load, for which stresses are calculated from the layered program. The means of the stresses at the bottom and of those at the top of each layer represent the vertical and confining stresses due to wheel load.
- (2) Those due to overburden, for which stresses in each layer are calculated as follows:

Layer	Effective Height of Overburden $h_i$ , inches	Effective Weight of Overburden $\gamma_{di}$ , pci
Base	$D_1 + D_2/2$	$\frac{1}{h_i}(\gamma_{AC} \times D_1 + \gamma_{\beta} \times 0.5D_2)$
Subbase	$D_1 + D_2 + D_3/2$	$\frac{1}{h_i}(\gamma_{AC} \times D_1 + \gamma_{\beta} \times D_2 + \gamma_{S\beta} \times 0.5 \times D_3)$
Subgrade	$D_1 + D_2 + D_3$	$\frac{1}{h_i}(\gamma_{AC} \times D_1 + \gamma_{\beta} \times D_2 + \gamma_{S\beta} \times D_3)$

$$\sigma_{iro\beta} = \gamma_{di} \cdot h_i \cdot \frac{\mu_i}{1 - \mu_i} \quad (9.3)$$

$$\sigma_{izo\beta} = \gamma_{di} \cdot h_i \quad (9.4)$$

where

$\gamma_{AC}, \gamma_{\beta}$  = unit weight of asphalt concrete, base, etc., pci;

$\sigma_{iro\beta}$  = radial stresses due to overburden in the  $i^{\text{th}}$  layer, psi;

$\sigma_{izo\beta}$  = vertical stresses due to overburden in the  $i^{\text{th}}$  layer, psi;

$\mu_i$  = Poisson's ratio of the  $i^{\text{th}}$  layer;

$\gamma_{di}$  = effective weight of overburden, pci.

Final stresses to compute the deformation in each layer are obtained from the following equations:

$$\sigma_{ir} = \sigma_{izo\beta} + \sigma_{irl} \quad (9.5)$$

$$\sigma_{iz} = \sigma_{izo\beta} + \sigma_{izl} \quad (9.6)$$

where

$\sigma_{ir}$  = total radial stress in the  $i^{\text{th}}$  layer, psi;

$\sigma_{iz}$  = total vertical stress in the  $i^{\text{th}}$  layer, psi;

$\sigma_{irl}$  = mean radial stress in the  $i^{\text{th}}$  layer due to wheel load, psi;

$\sigma_{izl}$  = mean vertical stress in the  $i^{\text{th}}$  layer due to wheel load, psi.

### Elastic Modulus

Elastic modulus for each layer is considered monthly; i.e.,

$$E_i(t) = f[E_{i1}, E_{i2}, \dots, E_{i11}, E_{i12}] \quad (9.7)$$

where

$E_{i1}$  = average modulus value of the  $i^{\text{th}}$  layer for January,

$E_{i2}$  = average modulus value of the  $i^{\text{th}}$  layer for February, etc.

### Applied Wheel Load

For various load groups, equivalent repetitions in terms of one single load group can be calculated as portrayed in Fig 9.2. Chan (Ref 20) found a linear relationship for total strain versus the log of the number of repetitions for several sands and gravel. Therefore, a straight-line relationship between the cumulative permanent strain  $\epsilon$  and the logarithm of the number of load repetitions  $\log N$  for materials of various pavement layers, other than the surface asphaltic concrete layer, is assumed. However, similar computations can be made if the straight-line relation is different from the assumed one. The equivalent repetitions are calculated in terms of the heaviest load group to give the least error in this computation. It is also assumed that load group  $h$  is the heaviest load group. For equivalent permanent strain (Fig 9.2),

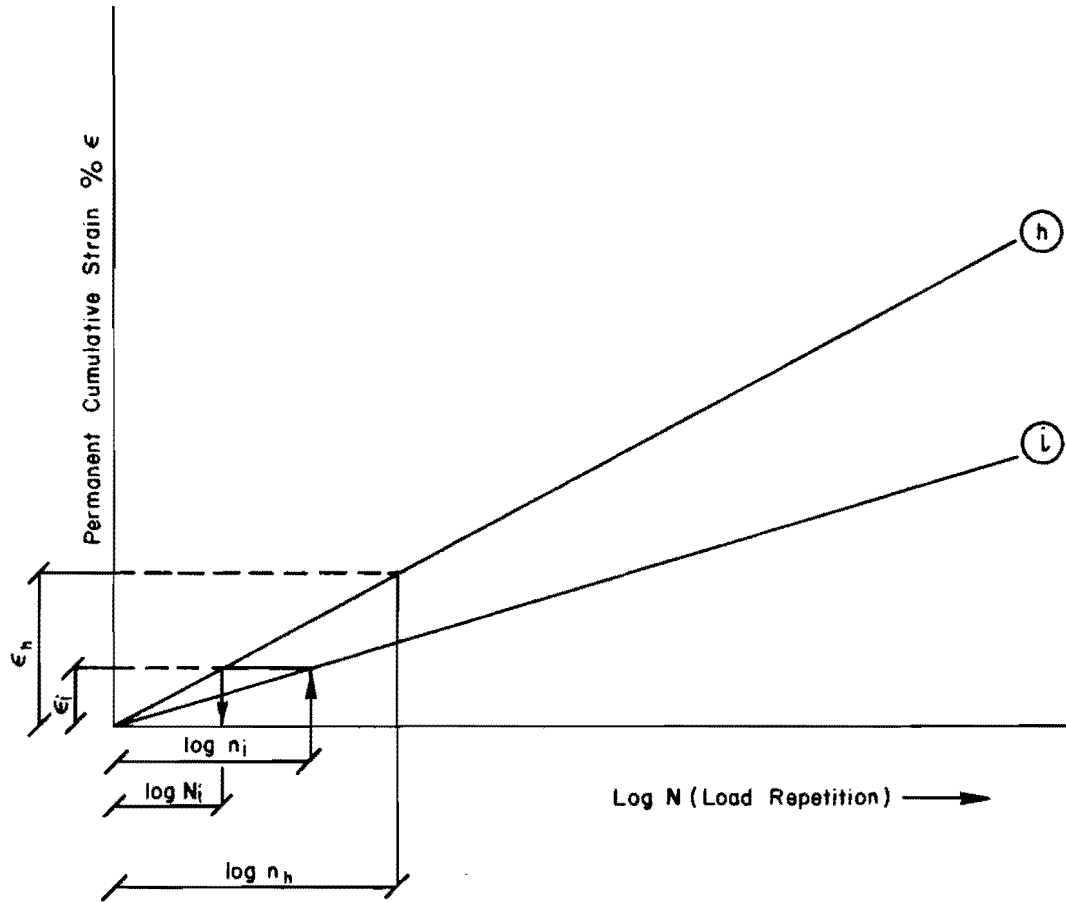
$$\frac{\log N_i}{\epsilon_i} = \frac{\log n_h}{\epsilon_h} \quad (9.8)$$

or

$$N_i = 10.0 \left( \frac{\log n_h}{\epsilon_h} \cdot \epsilon_i \right) \quad (9.9)$$

Total equivalent repetitions in terms of  $h$  load group, say  $N_{ht}$ , is given by





Note: The number on the curves represents the load group.

Fig 9.2. Development of equivalent load repetitions for one load group in terms of other load group.

$$N_{ht} = N_1 + N_2 + \dots + N_h = \sum N_i \quad (9.10)$$

Combining Eqs 9.9 and 9.10,

$$N_{ht} = \sum 10.0 \left( \frac{\log n_h \times \epsilon_i}{\epsilon_h} \right) \quad (9.11)$$

where

$N_i$  = equivalent number of load repetitions of load group of level  $i$  in terms of heaviest load group  $h$ ,

$n_i$  = actual load repetitions of load group of level  $i$ ,

$N_{ht}$  = total equivalent load repetitions in terms of heaviest load group,

$\epsilon_i$  = total permanent strain corresponding to load group  $n_i$ .

#### Permanent Strain in a Particular Month

Due to monthly variation in the material properties, the same load group creates different stress conditions in each layer each month. To find the cumulative deformation in each layer in a particular month, the net permanent strain caused by a particular load group in that month is required. This permanent strain in each layer, in percent inches per inch, is obtained from the difference of the permanent strain corresponding to the number of load repetitions at the beginning and at the end of that month.

$$\epsilon_{ip}(t) = \epsilon_{iE}(t) - \epsilon_{iB}(t) \quad (9.12)$$

where

$\epsilon_{ip}(t)$  = net permanent strain in the  $i^{\text{th}}$  layer for the  $t^{\text{th}}$  month;

$\epsilon_{iB}(t)$  = permanent strain in the  $i^{\text{th}}$  layer for the  $t^{\text{th}}$  month and at the beginning of that month;

$\epsilon_{iE}(t)$  = same as  $\epsilon_{iB}(t)$ , but at the end of the month.

### Permanent Deformation

The permanent deformation for each month in the pavement is calculated as

$$\Delta_i(t) = \epsilon_{ip}(t) \cdot D_i \cdot \frac{1}{100} \quad (9.13)$$

$$\Delta(t) = \sum_{i=1}^{\ell} \Delta_i(t) \quad (9.14)$$

where

$\Delta_i(t)$  = permanent deformation in the  $i^{\text{th}}$  layer and  $t^{\text{th}}$  month, in inches;

$\Delta(t)$  = permanent deformation in the whole pavement structure in the  $t^{\text{th}}$  month, in inches;

$\ell$  = number of layers.

### Cumulative Deformation or Rut Depth

The rut depth in a particular month is represented by the cumulative deformation of the pavement structure from the beginning of the pavement facility to the end of that month. Mathematically, the rut depth is given by:

$$\overline{RD}(t) = \sum_0^t \Delta(t) \quad (9.15)$$

Therefore, knowing the monthly deformations  $\Delta(t)$ , the rut depth is calculated by Eq 9.15.

## PROCEDURE TO COMPUTE THE RUT DEPTH

The steps in the calculation of rut depth, shown in Fig 9.1, are

- (1) From the axle load, modulus of elasticity of various layers, tire pressure, Poisson's ratio, and thickness of layers, compute the vertical and radial confining stresses at the top and bottom of each layer.
- (2) Compute the total radial and vertical stresses in each layer due to overburden and wheel load from Eqs 9.5 and 9.6.
- (3) Input the repeated load deformation curves obtained from the field for each layer except for the asphaltic concrete surface layer. Regression equations used in the computer program are developed in Chapter 5 from the repeated load-deformation data (Eqs 5.3, 5.4, and 5.5). Compute permanent strain corresponding to stress conditions and number of load repetitions at the beginning and end of each month for each load group.
- (4) Calculate the equivalent repetitions in terms of the single heaviest load group, using Eq 9.9.
- (5) Again compute the permanent strain from the regression equations at the beginning and end of each month (as in item 3), but only for the heaviest load group for the equivalent number of repetitions calculated in item 4.
- (6) For each month, the permanent strain in each layer is calculated from the difference of the strain values corresponding to the number of load repetitions at the beginning and end of that month from Eq 9.12.
- (7) From the permanent strain in each layer for each month, the total permanent deformation in the individual layers and for the whole pavement for each month is calculated with Eqs 9.13 and 9.14.
- (8) Finally, cumulative deformation for each month, representing the rut depth in the pavement is calculated by Eq 9.15.

## COMPUTER PROGRAM

The whole procedure for computing the expected rut depth is too lengthy to handle by hand calculations. Therefore, a computer program has been written which solves all the above mentioned steps and computes the values of the expected rut depth. To reduce the work of the designer, this part of the computer program is combined with the program developed for calculation of the cracking index in Chapter 8. The program has the alternative that either or both cracking index and rut depth values can be computed. Because most of the input data for calculation of rut depth and the cracking index are common to the combined program, manual as well as computer time is saved in solving a problem.

Moreover, this one computer program is easier to include in the existing flexible pavement system computer program. At present, the model does not consider the stochastic variations in space. This may be done at the time of detailed stochastic studies of various variables in the present flexible pavement system.

As indicated in Chapter 8, the computer program, flow chart, input guide, and sample input and output are enclosed in Appendix 4.

## CHAPTER 10. DEVELOPMENT OF ROUGHNESS INDEX MODEL

In this chapter, a model for quantification of roughness index is developed. It is hypothesized that the trend in the cracking index is a good indicator of the trend in the roughness index, and a correlation is established between the cracking index and the roughness index. Thus, a model is presented for roughness index in terms of axle load, number of axles, depth of pavement layer, and cracking index.

### THEORY

Cracking in an asphalt surface has long been used as a direct indication of a structural inadequacy somewhere in a pavement system, and cracking was used as the principal criterion of pavement failure at the WASHO Road Test (Ref 74). Cracking and patching were found to be of only minor significance in the performance model of a pavement at the AASHO Road Test (Ref 70), but that does not mean cracking is of minor structural importance. By the time fatigue cracking due to repeated loading has progressed enough to greatly impair the riding quality of a pavement, the pavement becomes very rough in terms of slope variance, and the slope variance or roughness index in the AASHO Road Test represented most of the detrimental effects of cracking (Ref 139). It can be assumed that fatigue cracking due to repeated loading is a good indicator of the roughness caused due to fatigue loading. Thus, a good correlation exists between the cracking index and the roughness index. The following comments of NCHRP Project 39 (Ref 42) support the above hypothesis very well:

"Careful examination of the criterion and the basic measurements tends to indicate that a significant amount of the drop in riding quality must have been due to the longitudinal roughness associated with fatigue cracking."

A mathematical correlation between the roughness index, in terms of the cracking index, and pavement structural elements is hypothesized as:

$$RI(x,t) = f[CI(x,t), D_i, W_j, L_j] \quad (10.1)$$

where

- RI = roughness index,  
 CI = cracking index,  
 (x,t) = function of space and time,  
 $D_i$  = thickness of the  $i^{\text{th}}$  layer in a pavement,  
 $W_j$  = axle load in kips of level  $j$  ,  
 $L_j$  = single or tandem axle of the load  $j$  .

In correlating slope variance and cracking-patching from observed data, it seems that each has a direct relationship with the number of repetitions of a particular load (Ref 139). Figure 10.1 shows a typical example of such a relationship. The relationship between cracking-patching and slope variance, depending upon the various values for structural elements of the pavement and the load, can be represented by the following equation.

$$SV = A + B\sqrt{CP} \quad (10.2)$$

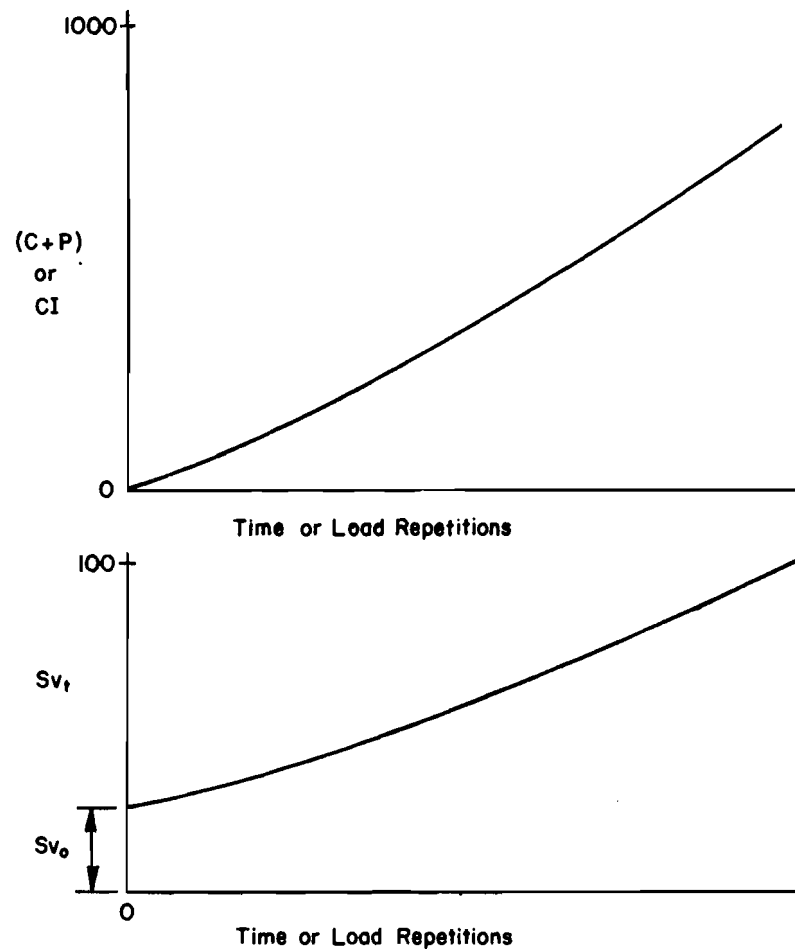
The values of  $A$  and  $B$  will depend upon the pavement structural element and load group, or

$$A = f(C_i, D_i, W_j, L_j) \quad (10.3)$$

$$B = f(C_i, D_i, W_j, L_j) \quad (10.4)$$

where

- SV = slope variance,  
 CP = measured crack-patching,



$Sv_0$  = Initial Value of the Slope Variance  
 $Sv_t$  = Slope Variance at any Time  $t$   
 C+P = Cracking and Patching  
 CI = Cracking Index

Fig 10.1. Typical example of relationship between cracking-patching, slope variance and number of load repetitions of a particular load.



$C_i$  = some coefficient to show the relative importance of various layers in a pavement.

and SV is given by

$$RI = \log (1 + SV) \quad (10.5)$$

Based on the above hypothesis (Eqs 10.1 and 10.2), AASHO Road Test data are analyzed later in this chapter to obtain a correlation between the roughness index based on measured values and cracking-patching. Only the load-associated distress is considered in the work reported here and thus, the AASHO Road Test data are used because they are primarily fatigue load data, with no significant effects of nonload-associated distress. The procedure and steps involved for the above analysis are shown in Fig 10.2. Through the regression analysis (Ref 18), trying various functions, a suitable model is obtained for a correlation between the dependent variable slope variance (to be predicted) and independent variables, cracking-patching, layer thicknesses, etc. (known).

#### QUANTIFICATION OF ROUGHNESS INDEX

Two approaches to quantification of the roughness index are discussed in this section, one based on literature and one on regression analysis.

##### Quantification Based on Literature

A literature review shows that the quantification of a roughness index is possible from the available information on AASHO Road Test results. The results of an analysis of AASHO Road Test data by the Asphalt Institute (Ref 139) include equations of the following forms:

$$\sqrt{SV_t} = \sqrt{SV_o} + bN_t \quad (10.6)$$

$$\overline{RD}^2 = bN_t \quad (10.7)$$

$$\sqrt{CP} = bN_t \quad (10.8)$$

$$RI(x,t) = f[CI(x,t), D_i, W_j, L_j]$$

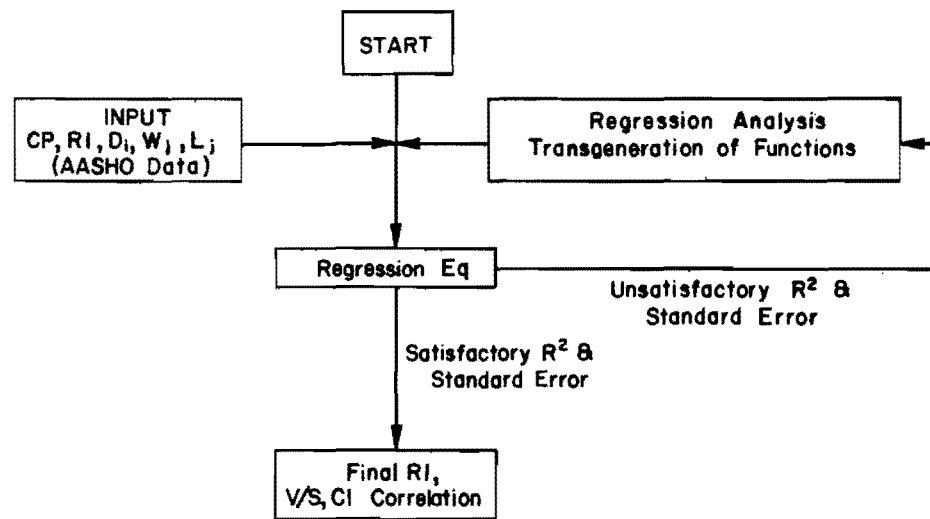


Fig 10.2. Flow chart for quantification of roughness index.

where

$SV_t$  = slope variance at any time  $t$ ,

$SV_0$  = slope variance of the pavement at the time of construction,

$b$  = rate of deterioration,

$N_t$  = accumulated load applications to time  $t$ ,

$\overline{RD}$  = rut depth.

The rate of deterioration  $b$  was shown to depend on the thicknesses and structural coefficients of different layers, subgrade strength, and load parameter:

$$\log b = a_0 + a_1 D_1 + a_2 D_3 + a_4 L + a_6 WP \quad (10.9)$$

where

$a_0, a_1, \text{ etc.}$  = constants representing some structural coefficient and depending upon the strength properties of pavement layers,

$L$  = factor representing axle load group,

$WP$  = factor for outer or inner wheel path.

Painter also obtained some numerical correlations for Eqs 10.6 to 10.9. Based on his work, the correlation between cracking-patching is

$$D_1 + 0.284D_2 + 0.228D_3 = 9.403 + 2.931 \log W_{18} - 1.466 \log CP + \log F \quad (10.10)$$

and slope variance is

$$\begin{aligned}
 D_1 + 0.313D_2 + 0.2D_3 &= 7.412 + 3.477 \log W_{18} \\
 &- 3.477 \log (\sqrt{SV} - \sqrt{SV_0}) + \log F
 \end{aligned}
 \tag{10.11}$$

where

$W_{18}$  = millions of accumulated load applications of 18-kips load,

$F$  = climate factor.

Combining Eqs 10.10 and 10.11 to eliminate  $\log W_{18}$  and combining terms gives

$$\begin{aligned}
 \log (\sqrt{SV} - \sqrt{SV_0}) &= 0.053D_1 + 0.007D_2 + 0.0201D_3 \\
 &- 1.075 - 0.053 \log F + 0.5 \log (CP)
 \end{aligned}
 \tag{10.12}$$

In the Painter's analysis, the average value for  $F$  was found to be 4.0. Eliminating  $\log F$  between Eqs 10.10 and 10.11:

$$\begin{aligned}
 0.029D_2 - 0.028D_3 &= -1.991 + 0.546 \log W_{18} \\
 &- 3.477 \log (\sqrt{SV} - \sqrt{SV_0}) + 1.466 \log (CP)
 \end{aligned}
 \tag{10.13}$$

Thus, Eqs 10.12 and 10.13 can be used for predicting the slope variance from the known value of cracking-patching, thickness of pavement layers, and equivalent 18-kip load applications.

#### Quantification Based on Regression Analysis

In Eqs 10.10 and 10.11, the loads were converted to single-axle 18-kip equivalents (based on values in Ref 139, Table 3, and page 26); hence, no terms for load and number of axles appear in Eq 10.12. However, based on the discussion earlier in this chapter and Painter's analysis, a general relationship between roughness index and cracking-patching would be expected to be

$$\begin{aligned} \log (1 + \sqrt{SV} - \sqrt{SV_0}) &= A_1 + A_2 D_1 + A_3 D_2 + A_4 D_3 \\ &+ A_5 W + A_6 L + A_7 \log (1 + CP) \end{aligned} \quad (10.14)$$

where

$A_1, A_2, \text{ etc.} =$  constants of the regression equation,

$SV =$  slope variance at any value of cracking-patching (CP) ,

$SV_0 =$  initial slope variance of the pavement.

Using data for various AASHO Road Test sections (Table 10.1), a regression analysis was conducted. The data consist of representative sections, constituting various observations for the analysis which could be performed within a reasonable time and efforts under the scope of the project. The data for the analysis represent various loops, load groups, and combinations of thicknesses of various layers of the AASHO Road Test sections. In this analysis, computer program STEP-01 (Ref 18) was used. Input and output of the computer program, used for the regression analysis, are given in Appendix A4.5. Results of this analysis are consolidated in Table 10.2. On the basis of regression analyses alone, one term,  $[\log (1 + CP)]^2$ , i.e., first step, had a correlation coefficient  $R^2$  of 0.9289 and should be considered best for the proposed correlation between cracking-patching and slope variance for interpolation of results within the data analyzed, because the additional terms did not improve the value of the correlation coefficient or standard error of residual. However, from the engineering point of view, based on the earlier discussions (Eq 10.14), and for extrapolation of results from the available information, the inclusion of other terms in the correlation may be considered desirable. During the analyses of data, it was also seen that the first step in the regression analysis produced seven points which are more than twice the standard error away from the desired value, as against four points in the case of the eighth step. Also, the highest error of prediction is lower in step 8 in comparison to step 1. Addition of a few terms in case of computation by computers does not involve any significant difference in time or labor. Moreover, during the

TABLE 10.1 SECTIONS CONSIDERED FOR REGRESSION ANALYSIS FOR RI V/S CP CORRELATION  
(AFTER REF 70)

Loop 1					Loop 2					Loop 3					Loop 4					Loop 5					Loop 6														
Axle Load					Axle Load					Axle Load					Axle Load					Axle Load					Axle Load														
Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2												
None		None			2,000-S		6,000-S			12,000-S		24,000-T			18,000-S		32,000-T			22,400-S		40,000-T			30,000-S		48,000-T												
Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1														
Surface Thickness	Base Thickness	Subbase Thickness	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.													
			Lane 1	Lane 2				Lane 1	Lane 2					Lane 1	Lane 2					Lane 1	Lane 2					Lane 1	Lane 2	Lane 1	Lane 2	Lane 1	Lane 2								
1	0	8	0	857	858	1	0	4	0	721	722	2	0	1	4	0	165	166	3	0	1	4	633	634	4	0	1	4	485	486									
			8	867	868				4	727	728					4	125	126					4	607					608	4	451	452							
		16	833	834	0			743		744	2				143		144	2				571		572				2	415		416								
		8	841	842	4			717	718	8	153				154	3	569	570				3	429	430															
		0	827	828	0			755	756	0	113				114	4	599	600				4	449	450															
	8	847	848	4	719	720	4	135	136	8	573			574	8	419	420																						
	3	0	8	0	839	840	2	3	4	0	771			772	3	3	4	0			159	160	4	3			4	617	618	5	3	4	487	488	6	3	4	253	254
				16	859	860				0	729			730				0			127	128						0	585				586	0				413	414
			8	863	864	4			759	760	4			157			158	4			623	624					4	471	472										
			0	869	870	0			731	732	0			111			112	8			601	602					8	441	442										
16			829	830	4	741			742	4	137	138	12	583			584	12	411	412																			
5	0	8	0	837	838	3	6	4	0	709	710	4	0	4	0	163	164	5	0	4	619	620	6	0	4	481	482	7	0	4	261	262							
			16	825	826				0	775	776				0	109	110				0	603				604	0				443	444							
		8	851	852	4			757	758	4	147			148	4	627	628			4	473	474																	
		0	875	876	0			737	738	0	107			108	8	589	590			8	455	456																	
		16	819	820	4			711	712	4	115			116	12	597	598			12	453	454																	
6	0	8	0	821	822	4	0	4	0	769	770	5	6	4	0	129	130	6	6	4	575	576	7	6	4	425	426	8	6	4	297	298							
			16	823	824				0	739	740				0	117	118				0	595				596	0				437	438							
		8	865	866	4			773	774	4	131			132	4	625	626			4	417	418																	
		0	877	878	0			745	746	0	155			156	8	625	626			8	477	478																	
		16	871	872	4			749	750	4	119			120	12	577	578			12	477	478																	
7	0	8	0	849	850	5	3	4	0	749	750	6	0	4	0	153	154	7	0	4	621	622	8	0	4	439	440	9	0	4	297	298							
			16	879	880				0	763	764				0	141	142				0	587				588	0				421	422							
		8	873	874	4			763	764	4	153			154	4	621	622			4	479	480																	
		0	873	874	0			763	764	0	141			142	8	579	580			8	423	424																	
		16	873	874	4			763	764	4	153			154	12	579	580			12	423	424																	

NOTE:

Shaded sections are replicates  
Dotted sections are considered  
for RI v/s CP correlation.  
Sections marked with squares  
are compared for RI observed  
and computed from proposed  
model.

TABLE 10.2. RESULT OF REGRESSION ANALYSIS FOR CRACKING-PATCHING  
(CP) VERSUS ROUGHNESS INDEX (RI)

Step Number	Variables	Residuals	R	R <sup>2</sup>
1.	Constt. SQLCP	0.0680	0.9638	0.9289
2.	Constt. LCP SQLCP	0.0671	0.9651	0.9315
3.	Constt. LCP D <sub>2</sub> SQLCP	0.0669	0.9657	0.9325
4.	Constt. LCP D <sub>2</sub> L SQLCP	0.0670	0.9660	0.9332
5.	Constt. LCP D <sub>2</sub> W L SQLCP	0.0665	0.9669	0.9348
6.	Constt. LCP D <sub>1</sub> D <sub>2</sub> W L SQLCP	0.0668	0.9669	0.9350

(Continued)

TABLE 10.2. (Continued)

Step Number	Variables	Residuals	R	R <sup>2</sup>
7.	Constt. LCP D <sub>1</sub> D <sub>2</sub> D <sub>3</sub> W L SQLCP	0.0670	0.9671	0.9353
8.	Constt. LCP D <sub>1</sub> D <sub>2</sub> D <sub>3</sub> W L CBLCP SQLCP	0.0674	0.9692	0.9354

INDEPENDENT VARIABLES

- LCP = Log (1 + CP)
- D<sub>1</sub> = Depth of A. concrete
- D<sub>2</sub> = Depth of base
- D<sub>3</sub> = Depth of subbase
- W = Axle load
- L = 1. For single axle  
2. For tandem axle
- SQLCP = [Log (1 + CP)]<sup>2</sup>
- CBLCP = [Log (1 + CP)]<sup>3</sup>

DEPENDENT VARIABLES

- Arctan. Log (1 + √SV<sub>i</sub> - √SV<sub>o</sub>)
- When:
- SV<sub>i</sub> = Slope variance at any time
- SV<sub>o</sub> = Initial slope variance



analysis, it was observed that at the higher values of cracking index, the values of the roughness index often tended to give results relatively lower than the observed values. Therefore, the term  $[\log (1 + CP)]^3$  was also retained in the equation to help in predicting values closer to the actual values. Finally, Eq 10.15, which corresponds to step 8 in the regression program (Table 10.2), is adopted for the present analysis.

$$\begin{aligned} \text{Arctan } \log (1 + \sqrt{SV_i} - \sqrt{SV_o}) &= -0.09136 + 0.09108 \log (1 + CP) \\ &+ 0.02445 [\log (1 + CP)]^2 + 0.00778 [\log (1 + CP)]^3 \\ &+ 0.00837D_1 + 0.00458D_2 + 0.00175D_3 - 0.00386W \\ &+ 0.08325L \end{aligned} \quad (10.15)$$

This regression correlation, developed on the 95 observed points, has nine coefficients with a correlation coefficient  $R^2$  of 0.9354. For a mean value of 0.37 of the dependent variable, the standard error of the residuals is 0.0674 (coefficient of variation 18 percent), in comparison to the standard deviation in the variability of the SD Profilometer measurement of 0.0644 for assumed  $SV_o$  equal to one (Ref 155). Values of dependent variables in the analysis range from 0.0 to 1.0. The comparison of the predicted values from this model and the measured values from the AASHO Road Test is discussed in Chapter 11.

The AASHO Road Test measure of cracking-patching is theoretically obtained from the cracking index model and is defined as the cracking index, as explained in Chapter 8. Thus, for the performance calculation in the present report, the cracking-patching term in Eq 10.15 is replaced by the cracking index CI .

#### SELECTION OF MODEL

Equation 10.13 is given in terms of only equivalent 18-kip load group and is not suitable for the proposed design procedure which considers all load groups individually.

Figure 10.3 compares the actual data points with the predictions made by the equation obtained from Painter's analysis (Eq 10.12) and the regression model (Eq 10.15). Equations derived from literature (Eqs 10.12 and 10.13) are indeterminate at the zero value of CP. The regression model (Eq 10.15) predicts a mean value of  $\log(1 + \sqrt{SV} - \sqrt{SV_0})$  as 0.52, while Eq 10.12 predicts 0.514 against the actual mean value of 0.525. The standard error of residuals for regression model (Eq 10.15) is found to be 0.09 against 0.138 for Eq 10.12. Thus, the regression model is seen to predict the points more accurately and is determinate at all values of cracking-patching.

Verification of the roughness index values predicted by the regression model also shows good agreement with observed values, as further discussed in Chapter 11. Moreover, the equations (Eq 10.12 and 10.13) developed from Painter's analysis are obtained by an indirect relationship (Eqs 10.10 and 10.11) and may not be considered accurate.

The proposed regression model is generalized for various load groups and number of axles, and the correlation between cracking-patching and roughness index is derived directly from AASHO Road Test data. Thus, for the analysis presented here, the regression model represented by Eq 10.15 was adopted.

#### PROCEDURE FOR COMPUTATION OF ROUGHNESS INDEX

The procedure for calculating the roughness index for any pavement section is:

- (1) Calculate the cracking index values as detailed in Chapter 8.
- (2) Based on engineering experience, assume certain values for the initial slope variance ( $SV_0$ ) expected in a planned pavement. Initial values of slope variance, depending on the type of construction, generally vary from 1.0 to 3.0.
- (3) Using the known values of pavement layer thickness, axle load, number of axles, and computed cracking index obtain the slope variance with Eq 10.15.
- (4) Substitute the value of slope variance into Eq 10.5 to obtain the roughness index.

#### COMPUTER PROGRAM

The model selected for the roughness index contains several terms, and making the necessary calculations by hand is very time consuming. Moreover,

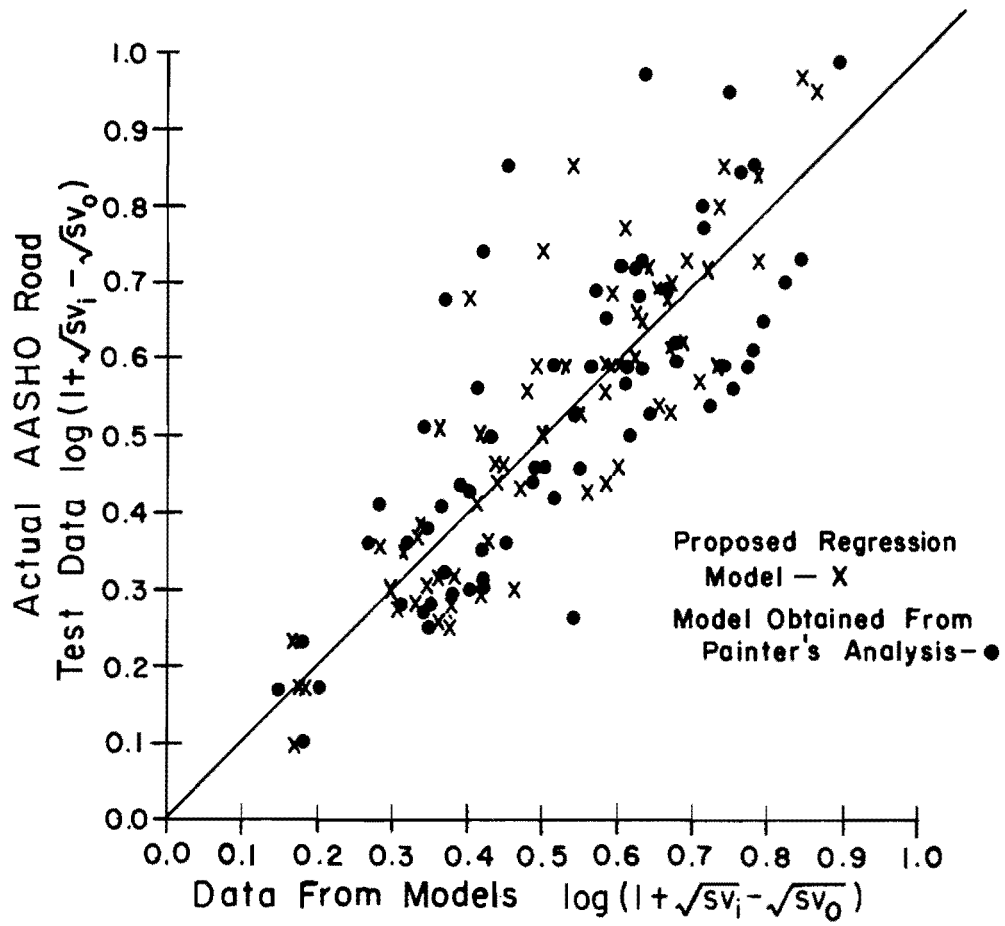


Fig 10.3. Comparison of  $\log(1 + \sqrt{sv_1} - \sqrt{sv_0})$  for various values of  $\log(1 + CP)$  computed from the model of painters analysis with that predicted from the proposed model.

the whole procedure of pavement design developed here is to be included in an existing flexible pavement systems computer program. A computer program for the calculation of the cracking index and rut depth index has been developed, as explained in Chapters 8 and 9. A computer program for calculation of the roughness index has also been developed and is included in Appendix 4. This computer program, for the roughness index also calculates the present serviceability index. The input consists of the cracking index, rut depth index, pavement layer thicknesses, axle load, number of axles, and initial slope variance. A typical output is included in Appendix 4.

## CHAPTER 11. VERIFICATION OF DISTRESS MODELS

The purpose of this chapter is to verify the distress index models which were developed in Chapters 7 through 10. This chapter is divided into four parts, each of which provides details of verification of one of the models developed for cracking index, slope variance, rut depth, and PSI with the AASHO Road Test data. The AASHO Road Test sections which are compared are shown in Fig 11.1. These 28 sections were selected based on the following criteria:

- (1) to represent various load groups;
- (2) to represent various loops;
- (3) to represent various combinations of layer thicknesses;
- (4) to represent some sections without base and some without subbase;
- (5) considering the reasonable time to be spent, scope of the project, computer time involved in solving problems, and money involved consistent with the accuracy desired and obtained.

All 28 sections were carried over for all distress models. Overdesigned sections such as 763 were avoided in this selection because these were not of much value for comparison. Sections of loop 1 were not considered because this loop did not carry any load.

### CRACKING INDEX MODEL

The detailed development of this model was discussed in Chapter 8. Figure 11.2 compares the calculated cracking index and actual measured values of cracking-patching of six AASHO Road Test sections. This comparison for the other 22 sections is included in Appendix 8 (Figs A8.1 through A8.18).

#### Computation of Cracking Index

In computing the cracking index values, the material properties characterized in Chapter 5 are used. Tables 5.2, 5.3, and 5.8 show the adopted monthly values of elastic modulus. The actual monthly traffic data of AASHO Road Test (Ref 70) and the computer program procedure detailed in Chapter 8

Loop 1					Loop 2					Loop 3					Loop 4					Loop 5					Loop 6															
Axle Load					Axle Load					Axle Load					Axle Load					Axle Load																				
Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2			Lane 1		Lane 2													
None		None			2,000-S		6,000-S			12,000-S		24,000-T			18,000-S		32,000-T			22,400-S		40,000-T			30,000-S		48,000-T													
Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1					Main Factorial Design Design 1															
Surface Thickness	Base Thickness	Subbase Thickness	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.		Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.														
			Lane 1	Lane 2				Lane 1	Lane 2					Lane 1	Lane 2					Lane 1	Lane 2					Lane 1	Lane 2	Lane 1	Lane 2	Lane 1	Lane 2									
1	0	8	16	057	058	0	4	721	722	2	0	8	16	165	166	3	0	8	16	633	634	4	0	8	16	485	486													
				867	868	4		727	728					2	607					608	2					451	452													
				833	834	3		743	744					3	571					572	3					415	416													
				841	842	4		717	718					2	569					570	3					429	430													
	6	0	8	16	827	828	3	0	4		8	16	0	4	8		16	113	114	6	0		4	8	16	599	600	9	0	4	8	16	449	450						
					847	848												4	719							720	3						573	574	3	419	420			
					839	840												0	771							772	12						617	618	1	487	488			
					859	860												4	729							730	4						585	586	3	413	414			
	0	8	16	0	863	864		6	0		4	8	16	0	4		8	16	157		158		9	0	4	8	16		623	624	3	0	4	8	16	471	472			
					869	870													8		731								732	12						601	602	2	441	442
					829	830													4		741								742	4						583	584	3	411	412
					837	838													0		709								710	8						619	620	1	481	482
0	8	16	0	825	826	6			0	4	8	16	0	4	8	16	111	112	9		0	4		8	16	603	604		3	0		4	8	16	443	444				
				851	852												8	775								776	4								597	598	1	473	474	
				875	876												0	757								758	0								589	590	2	455	456	
				819	820												8	737								738	2								597	598	2	453	454	
6	0	8	16	875	876		6		0	4	8	16	0	4	8	16	107	108		9	0	4		8	16	575	576	3		0		4	8	16	425	426				
				821	822												6	769								770	2								595	596	4	437	438	
				823	824												0	739								740	0								577	578	8	417	418	
				865	866												4	773								774	12								625	626	12	477	478	
0	8	16	0	877	878			6	0	4	8	16	0	4	8	16	119	120			9	0	4	8	16	605	606			3	0	4	8	16	439	440				
				849	850												0	749								750	8								587	588	8	421	422	
				879	880												4	763								764	12								621	622	12	479	480	
				873	874												8	745								746	4								579	580	4	423	424	
6	0	8	16	871	872	6			0	4	8	16	0	4	8	16	141	142	9			0	4	8	16	631	632		3		0	4	8	16	469	470				
				849	850												4	745								746	12								593	594	12	445	446	
				879	880												8	749								750	4								605	606	4	445	446	
				873	874												16	763								764	8								587	588	8	421	422	

NOTE:

Shaded sections are replicates.  
Dotted sections are used for comparison for the proposed models.  
All thicknesses shown are in inches.

Fig 11.1. AASHTO Road Test sections used for comparison of distress index and performance model (After Ref 70).

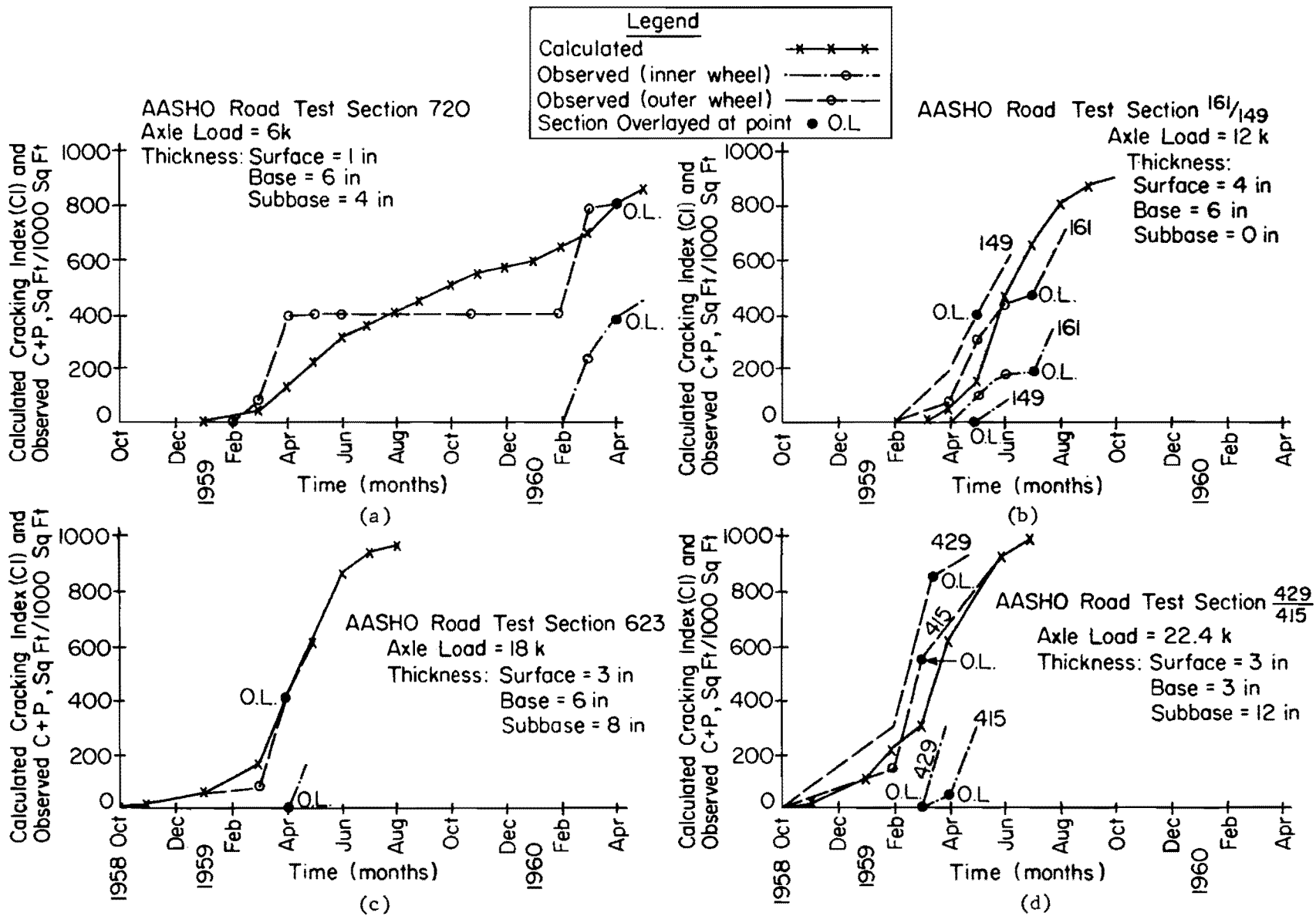


Fig 11.2. Computed cracking index (CI) versus observed cracking-patching (C + P).

to calculate the cracking index are used in these computations. Typical computer computations are shown in Appendix 4. The observed field values of cracking-patching are from the AASHO Road Test performance record for each test section (Ref 70). Typical performance records for a few sections are included in Appendix 5. The observed and computed values thus obtained are plotted and compared. For 2-kip axle loads (Figs A8.1 through A8.3) on loop 2 of the AASHO Road Test sections, the results based on the above mentioned material properties were quite conservative, for the following reasons:

- (1) Small loads and low tire pressures resulted in a lower effective tire radius, which, for the same speed, gives a loading time that is about 50 percent of the average for all axle loads. The lower loading time can give asphalt concrete stiffness values for 2-kip axle loads that are up to 25 percent more than the stiffness values for higher loads. Moreover, a lower time of loading will increase the fatigue life of the pavement.
- (2) The test results for the AASHO Road Test sections indicate that the outer wheel path generally showed more distress than the inner wheel path, but for lane 1 of loop 2 in many cases, and especially the sections under consideration, the inner wheel path showed more distress than the outer wheel path.
- (3) Report 5 of the AASHO Road Test (Ref 70) shows that lane 1 of loop 2 always behaved differently from other loops; in most cases, when other lanes showed good correlation with certain parameters, lane 1 was dropped from consideration. In some cases the correlations were based on a minimum asphalt concrete thickness of 2 inches, which excluded quite a few sections of loop 2 (Ref 70, pp 37, 38, 40, 41, 42, 43, 60, and 66).

In view of the above, the cracking index values for 2-kip axle loads in loop 2 were recomputed with revised stiffness modulus and fatigue characteristics for asphalt concrete. The cracking index values thus calculated are plotted in Figs A8.1 through A8.3.

#### Comparison of Computed and Field Cracking Distress

A study of the comparison of observed and computed cracking index shows that most of the plots agree well, but in a few cases the calculated values are different than the observed values.

The reasons are not known, but even the replicate sections in the AASHO Road Test did not show the same cracking-patching history. Therefore, it can be expected that observed values of cracking-patching will deviate from any calculated with a theoretical model. A model that resulted in residual errors that averaged about the same as the deviations of the replicate observations



would be satisfactory. These replicate sections showed differences of up to 450 square feet per 1000 square feet in the cracking-patching measurements for the same month. Furthermore, for Sections 307 and 305 (Fig A8.16), there was a year's difference in failure, i.e., Section 305 had a failure pattern similar to that for 307, but it was one year sooner.

In comparing results, consideration must be given to the overall trends and quality of the plots, as specific points may differ for various reasons at different times.

Based on this criteria for expected variation and engineering judgment, for comparison and explanation, the plots of observed and computed cracking index (Figs 11.2 and A8.1 through A8.18) were divided into three categories as follows:

- (1) Good fit. Calculated and observed plots match very closely. The plots have similar shapes and differences at any one point are seldom reach more than 300 square feet.
- (2) Medium fit. Calculated and observed plots do not fit well, but the differences can be explained and do not affect the predicted pavement performance significantly. The difference, however, in a few cases may reach 450 square feet or more. This difference of 450 square feet in a particular month is equivalent to a difference of 0.21 in PSI which is not of much significance.
- (3) Poor fit. Calculated and observed plots have considerable differences. Some of them may be explained; some are due to random variations.

The number of sections in each category is

- (1) Good fit - 20
- (2) Medium fit - 5
- (3) Poor fit - 3

Thus, plots for more than 70 percent of the sections under study fit well. Results were good for all the sections in the 12-kip loop 3 group. In addition, other explanations given below show that the cracking index model gives very good results. For extreme conditions of loading and thickness, some deviation can be expected, but this is a normal statistical characteristic and is acceptable.

#### Discussion of Comparison

The comparison of observed and calculated values of cracking index and the above discussion shows that general discrepancies observed in the results for loop 2 may be resolved by proper material characterization. The

following discussion for all loops explains some apparent differences that are due to time phase and measurement discrepancies; some other differences can also be explained and discrepancies resolved as discussed hereafter. Only a few sections actually have notable differences between actual and predicted values.

- (1) If for some reason a failure in the field did not occur one spring season, the whole cycle is likely to be shifted in the actual pavement sections in the field (Fig A8.16). In this case, the shape of the distress envelopes is the same, but the time phase difference will be apparent. Since the theoretical model does not consider this contingency, differences between field and computed values can arise in several cases. Figures A8.4, A8.11, A8.16, and A8.17 are examples of this situation. At first glance it may appear that the observed and calculated values are very different but a careful study shows that if the time phase shift may be considered, the values are in close agreement. Therefore, the sections put in the categories of poor or medium fit in fact had good fit, and confidence in the developed model is improved.
- (2) In several cases not enough points are available to show the actual trend (Figs A8.10, A8.13, and A8.14).
- (3) Monthly average variations in the modulus values were considered, in accordance with the procedure developed, but shorter periods might give closer results.
- (4) Material characterization is a very important factor. Any discrepancy in the characterization of material properties can cause differences in the results. In this study the materials were characterized on an average basis not for specific sections.
- (5) During hot months the rate of crack propagation is slow; the asphaltic concrete becomes softer and the cracks become temporarily invisible. Therefore, during these months inaccuracy in the measurements of observed data could occur and more discrepancy is likely to show up between calculated and observed values. In Section 305 (Fig A8.16) observed values decreased instead of increasing, which was not as expected. Moreover, during hot months, due to lower stiffness values, higher strains are expected; these may appear in the extreme end of the fatigue curve and show more distress, according to the design criteria, than actually happened and was observed. The sections in these periods may tend to behave under constant strain rather than constant stress conditions (assumed), making the calculated values more conservative. These facts are apparent from many sections (Figs A8.4, A8.11, A8.15, A8.16, and A8.17).
- (6) The daily temperature cycling and other environmental effects when taken into account will also tend to give more realistic and better trends in all cases.
- (7) Once the surface has distressed to some extent, impact and regenerated or progressive added distress effects may also influence the pavement condition. This result may be different for different axle loads.

- (8) The slope of the fatigue curve may vary slightly for different axle load groups and temperature conditions, since the response of asphaltic concrete may be affected to some extent by time of loading and temperature. In this study average characteristics are assumed. Thus, the results for load groups other than 12 kips show comparatively more variation.
- (9) From the discussion in Chapter 4, it appears that in thinner sections controlled-strain is more suitable. Therefore, for thinner sections the constant stress assumption is likely to give conservative results (Figs A8.1 through A8.4).
- (10) Some of the unexplainable differences may be due to random statistical behavior of the test sections, especially extremely thin sections with small loads, for which extreme values of strain may show even more variation in fatigue curve and in which statistically more variation is expected.

The following comments from NCHRP Report 35 (Ref 164) support the use of the procedure explained herein.

"It has been shown that tensile strains of a magnitude sufficient to initiate fatigue cracks occur on the road surface and theoretical considerations of a layered system indicate that even greater tensile strains occur on the under side of the top layers. These tensile strains will be a maximum when the overall stiffness of the entire structure is a minimum. The stiffness of bituminous materials is dependent on temperature and the critical condition is therefore likely to arise at high temperatures during the summer months.

However, the fatigue tests at high temperatures show that although cracks initiate under these conditions, they propagate only slowly due to the lower stress, and thus failure will not necessarily be apparent at this time. But once the temperature falls and the stiffness of the bituminous layers increases, there will be an increase in the stress, particularly at the tip of the crack, owing to stress concentration effect. This will result in more rapid propagation of any fatigue cracks under winter conditions, but again it will not necessarily lead to failure owing to the freezing of the subbase and subgrade and the resultant increase in strength. During the thaw period, however, the fact that the surface layers are cracked increases greatly the likelihood of pavement deterioration from penetration of water and consequent local subgrade failure."

#### Summary

From the above discussion, it can be concluded that the difference in calculated and observed values, even in the cases of poor and medium fit plots, can reasonably be explained in all but one or two cases (Fig A8.14), which could not be explained because not enough data points are available. The model developed seems to give acceptable results, is based on a rational recognized approach, and can be used for design of flexible pavements. Any improvements,

as discussed above, in the model will further reduce the gaps between observed and calculated results.

#### RUT DEPTH INDEX MODEL

The details and development of the rut depth index model were discussed in Chapter 9. For the comparison of rut depth index values, the sections used for the cracking index model are considered here. Figure 11.3 shows a comparison of the calculated rut depth index and observed values of six selected AASHO Road Test sections. Other test sections so compared are included in Appendix 8 (Figs A8.19 through A8.36).

##### Computation of Rut Depth Index

In computing the rut depth index values also the material properties were characterized as in Chapter 5. The monthly values of elastic modulus contained in Tables 5.2, 5.3, and 5.8 are used. Equations 5.1, 5.2, and 5.4 are used for repeated load-deformation characteristics of materials. The actual monthly traffic data of the AASHO Road Test (Ref 70) are used. Typical computer computations are included in Appendix 4. The rut depth index computer program, discussed in Chapter 9, computes the total deformation as well as the deformations of the base, subbase, and subgrade layers. When the total calculated deformation was compared with the observed values of the AASHO Road Test sections, it was noted that the calculated values were generally higher initially but were approximately equal at the time of failure. In this regard, a study of the AASHO Road Test (Ref 70) showed that

- (1) Rate of rut development decreased with load applications.
- (2) Although pronounced rutting developed in both wheel paths of the pavement surface, very little was apparent in the embankment soil where the sections were maintained. This and other thickness and trend measurements of the AASHO Road Test sections were considered to be evidence that pavement layers were mainly responsible for rutting observed in the wheel paths of the pavement surface and that the subgrade makes almost no contribution toward rut depth.
- (3) If the sections that were failing at a rapid rate were not maintained rutting or distortion of the pavement in the wheel paths extended into the embankment soil.

Based on the above observations, plots of observed and computed rut depth were made (Figs 11.3 and A8.19 through A8.36) without considering the effect of subgrade deformation unless the condition in item 3 above was encountered.

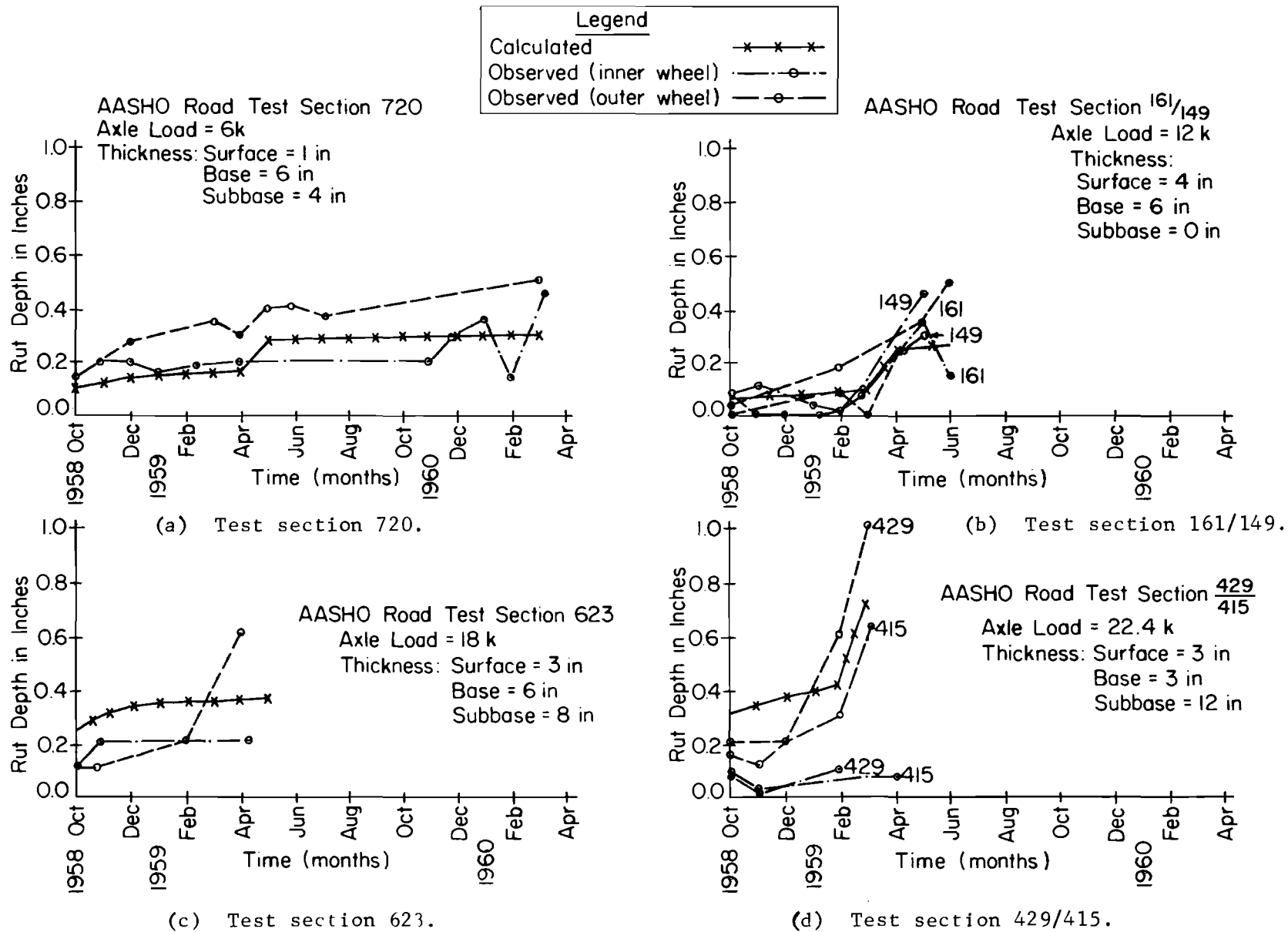


Fig 11.3. Observed versus calculated rut depth index.

In the present design procedure, the computer program computes the deformation in all the pavement layers separately. Therefore, the observation made in item 2 can be accounted for in the procedure easily.

#### Comparison of Computed and Field Rut Depth

Figures 11.3 and A8.19 through A8.36 show that there was close agreement between the observed and calculated values. Of 28 sections which were compared, about 23 showed a difference between the calculated and observed values of less than 0.2 inch and none of the sections has a difference of more than 0.3 inch.

Not even the replicate sections in the AASHO Road Test showed the same performance and rut depths; therefore, it can be expected that observed rut depths data will deviate from values of rut depths calculated from any theoretical model. A satisfactory model should give residual errors that average about the same as the deviations of replicate observations from their own mean. The rut depth differences in the observed values of the replicate sections have been noted as high as 0.3 or even more.

#### Discussion of Comparison

When observed and calculated values of rut depth are compared, the following points must be considered:

- (1) For calculations of rut depth, not all the local climatic and other factors could be or were considered in detail; for example, certain rainfalls and snowfalls of short duration, non-load associated effect, and temperature stresses. Therefore, some differences are bound to occur. Comparison should be made of the general trend of the plots and their qualitative rather than exact quantitative agreements for each month. In some cases the observed values of rut depth decreased with increases in traffic load repetitions, instead of increasing. This is not correct and may be due to observation errors. In many cases corrections close the gap in observed and calculated values.
- (2) Quantitative differences between observed and calculated values may be considered in the light of the effect on the PSI values of the sections. In the PSI and PSR observations of the AASHO Road Test data (Ref 70), a difference as large as 1.1 was observed and the mean value of the difference for 74 observations, on which PSI equation was based, was  $0.3$ . The contribution of the rut depth in the PSI equation is  $1.38 RD^2$  (Ref 70). Assuming that all the error in PSI is due to rut depth and that a correct average rut depth is about 0.25 inch, the reasonable acceptable value of the difference between computed and observed values of rut depth can be computed. It is seen that for a mean correct rut depth of 0.25 inch even a

value of rut depth between 0.0 and 0.53 and similarly at a mean value of 0.6 inch rut depth, values from 0.38 to 0.76 will be within the acceptable difference of 0.3 in the PSI values.

- (3) At the AASHO Road Test a high level of correlation was found between deflections and performance. Performance is affected by degree of rutting. Thus, deflections were correlated with rutting (Ref 70, Fig 95). Dotted lines on the plots of deflection vs rut depth in Fig 95 in Ref 70 were located one standard error of the estimate from the regression line. For a creep speed deflection of 0.04 inch, for example, the corresponding values of rut depth for one standard deviation vary from 0.35 to 0.65. A model predicting a variation to this extent was acceptable for AASHO Road Test data. Therefore, any model of this accuracy should ordinarily be satisfactory.
- (4) In the replicate sections (Ref 70, Appendix C) of the AASHO Road Test reports, the spring creep deflection data show a variation from 0.038 inch to 0.072 inch for loop 3, lane 2. The corresponding rut depths from Fig 95 of Ref 70 are

<u>Axle Applications</u>	<u>Rut Depth in Inches</u>
140,000	0.2 - 0.7
610,000	0.35 - 0.8
1,114,000	0.45 - 1.0

Deflections for 18-kip axle loads in loop 4, lane 1, for replicate sections measured 0.077 inch and 0.056 inch. The corresponding rut depths at 140,000 applications are 0.8 inch and 0.47 inch. These data show that in the replicate sections of the AASHO Road Test data the above order of variation in the rut depth can be expected and any model predicting with this accuracy should be satisfactory.

- (5) For a few sections the difference between observed and calculated rut depth was relatively more in the beginning of the performance period and then evened out. This is not considered significant for the following reasons:
- (a) A discrepancy in rut depth in the beginning affects the present serviceability much less. For example, a discrepancy of 0.2 inch between values of 0.1 and 0.3 will affect the PSI only by 0.1, while toward the end the same difference between rut depth values of 0.5 and 0.7 affect the PSI by 0.33.
- (b) The computed values are in most cases conservative.
- (c) From the design point of view, relatively more correct values for the level of service are required at some time other than the beginning, for example, at the time of overlay, and maintenance, when correct values of PSI are more important for decision criteria.
- (d) In the beginning, when the ruts are not visibly well defined, measurement error in the observed values is likely to be relatively higher.

### Summary

It is seen from the plots that in almost all cases the differences are much less than the expected minimum accuracy discussed above, and in most cases the predicted or calculated values are very close to the mean observed values for outer and inner wheel paths. From the above discussions, it can be concluded that rut depth prediction by the model and the method presented in this report are dependable and can be used in the design of the flexible pavements.

### VERIFICATION OF ROUGHNESS INDEX MODEL

To estimate the value of slope variance, a roughness index model was developed (Chapter 10). This model predicted the roughness index values corresponding to the cracking index. In this section the computed values of roughness index are compared with the observed values of the selected 28 AASHO Road Test sections. These sections are the same as those selected for the cracking index and rut depth index models (Fig 11.1).

#### Computation of Roughness Index

Cracking index values are theoretically calculated equivalent values and represent the cracking-patching in the pavement. Therefore, from the cracking index values calculated from the cracking index model as discussed in the first section of this chapter for various AASHO Road Test sections, corresponding values of roughness index (RI) were calculated by the computer program (Appendix A4.4), as discussed in Chapter 10. In these calculations the regression analysis model (Eq 10.15) developed earlier in Chapter 10 is used.

#### Comparison of Computed and Field Roughness Index

Figure 11.4 shows a comparison with six selected AASHO Road Test sections. This comparison with the other 22 sections is shown in Figs A8.37 to A8.58. The computations were made for 28 AASHO Road Test sections. Careful scrutiny and engineering judgment will show that about 16 sections show very good predictions, six may be termed as showing medium fit, and six show comparatively poor results. However, it must be remembered that the calculated roughness index values were obtained from the calculated cracking index values. Any discrepancy in the observed cracking-patching and calculated cracking index, therefore, will also show up here. Such observed differences were explained



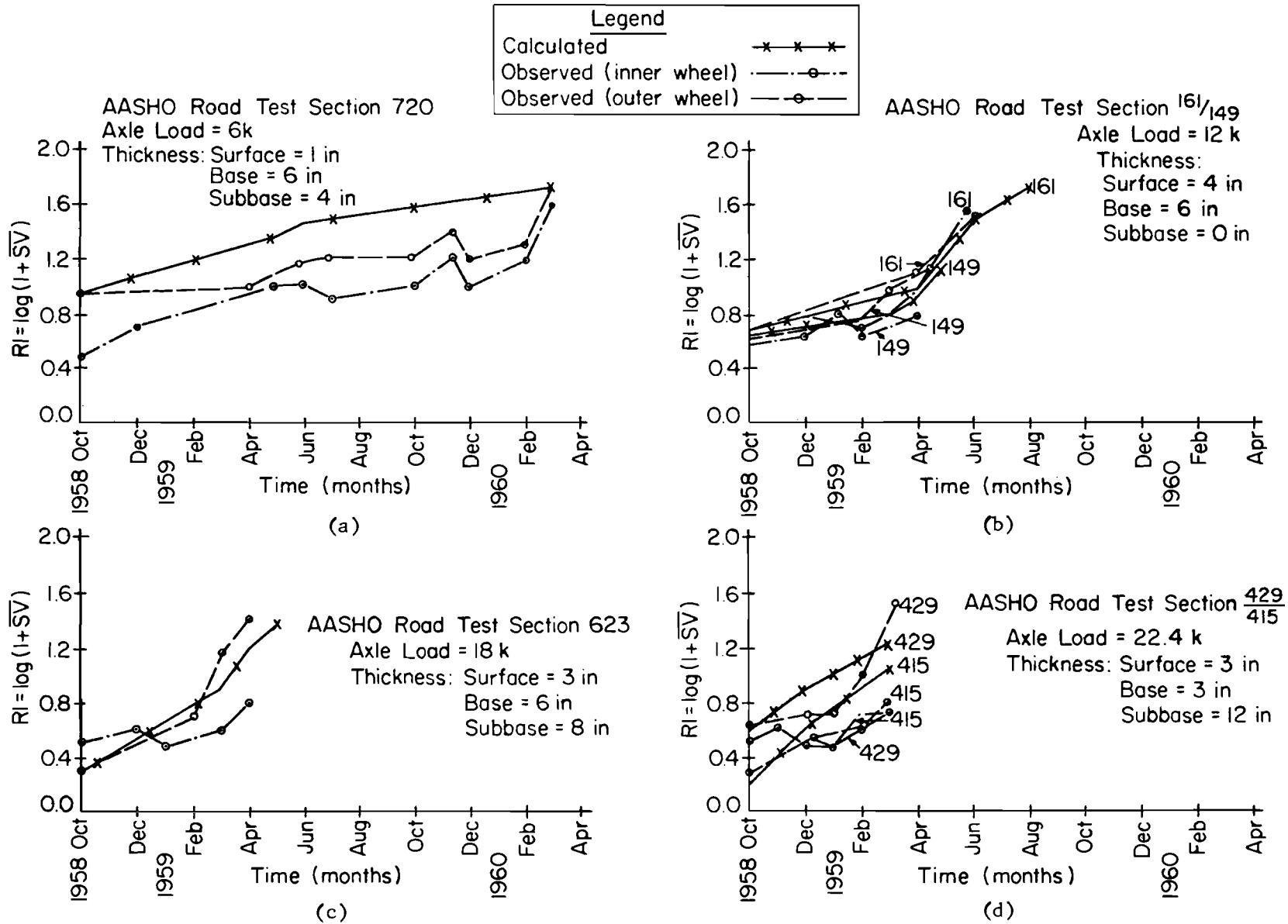


Fig 11.4. Calculated and observed roughness index, RI.

earlier, and it was noted that most discrepancies could be explained, and thus the calculated cracking index values were reliable and satisfactory. Once the cracking index value discrepancy is explained, the estimated roughness index value will also give satisfactory results, and the apparent differences noted above in the calculated and observed roughness index values will be reduced.

For plots with good fit, the differences are very small, with the greatest difference being 0.2. In the medium fit category the difference was occasionally as large as 0.4. For poor fit plots, the differences in calculated and observed roughness index values were sometimes as much as 0.6 to 0.7, although most differences were much less. However, with the explanation already offered for the cracking index these differences will be reduced. Moreover, even in these sections (poor fit), values at the beginning and end of the performance period compare very well, but observed values of roughness index either decreased or stayed the same at some other time resulting in apparent high differences in values during the performance period. The constant or decreased observed values of roughness index during the performance period are considered wrong for theory and/or logic. Proper corrections in the observed data will reduce the differences.

The replicate sections in the AASHO Road Test did not show the same performance and slope variance measurements, and it can be expected that observed performance data will deviate from calculated values from any theoretical model. For the AASHO Road Test sections, replicate differences in roughness index values as much as 0.40 and in some cases 0.7 were observed. These replicate differences in roughness index are equivalent to replicate differences of 0.76 and 1.33, respectively, in PSI. Moreover, even the standard deviation of the variability of the SD Profilometer measurements for roughness index has been observed as 0.37 (Ref 155). The final effect of any discrepancy of roughness index should be compared in terms of present serviceability index, and this is done in the last section of this chapter.

Figures 11.5 through 11.9 show the comparison of calculated and observed roughness index values for some additional AASHO Road Test sections. For these sections, values of roughness index were calculated, from the actual observed values of cracking-patching as well as from the computed cracking index. The calculated and observed values in both cases are found in close agreement for these sections also.

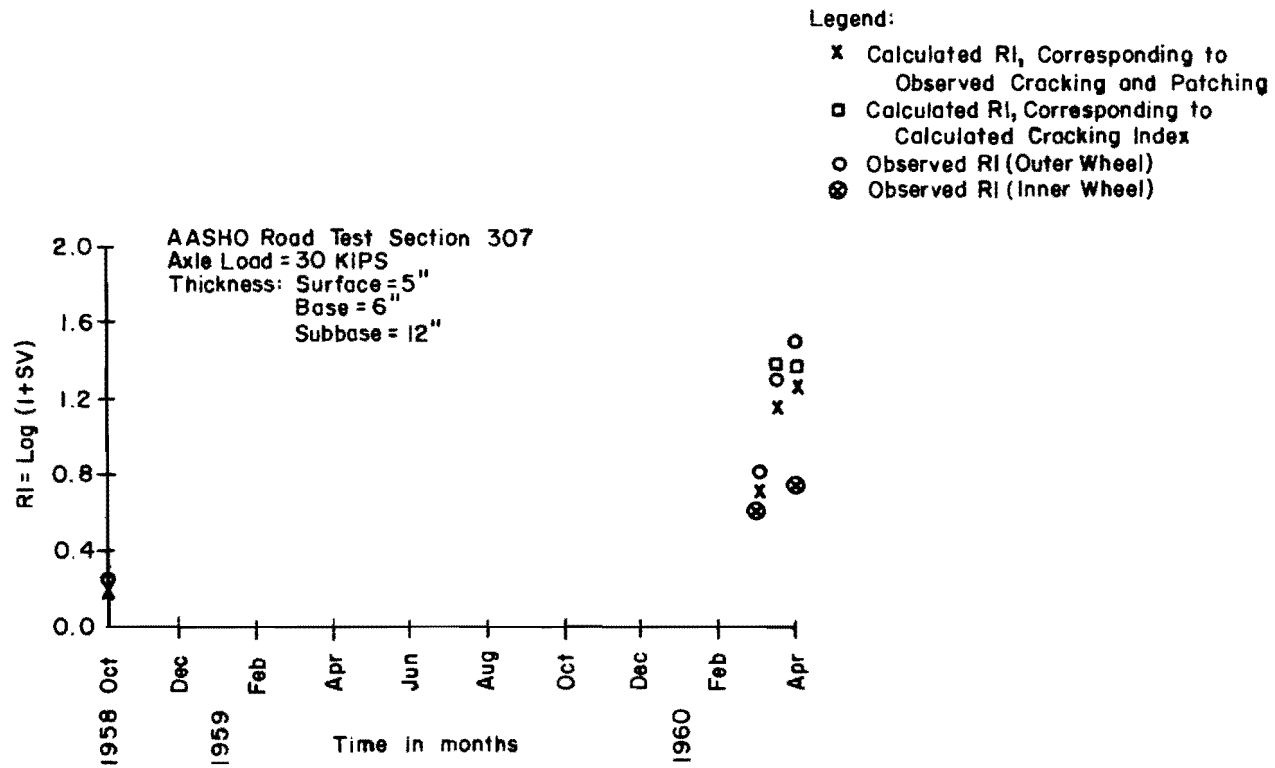


Fig 11.5. Verification of roughness index model.

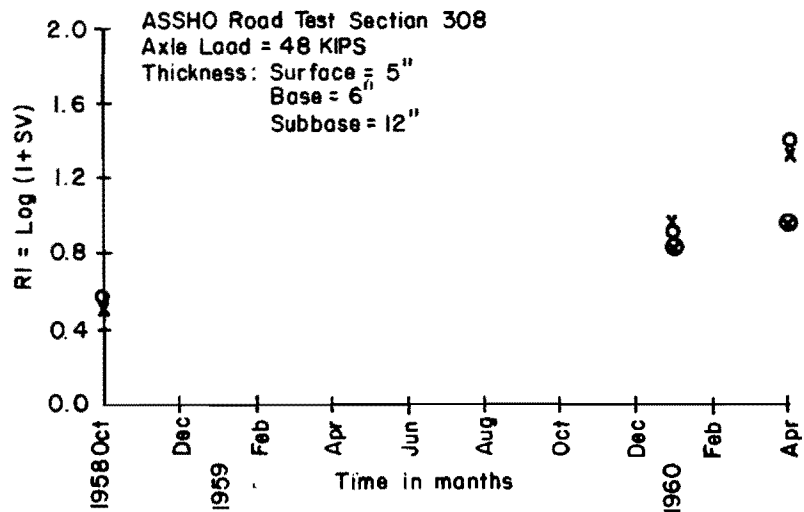


Fig 11.6.

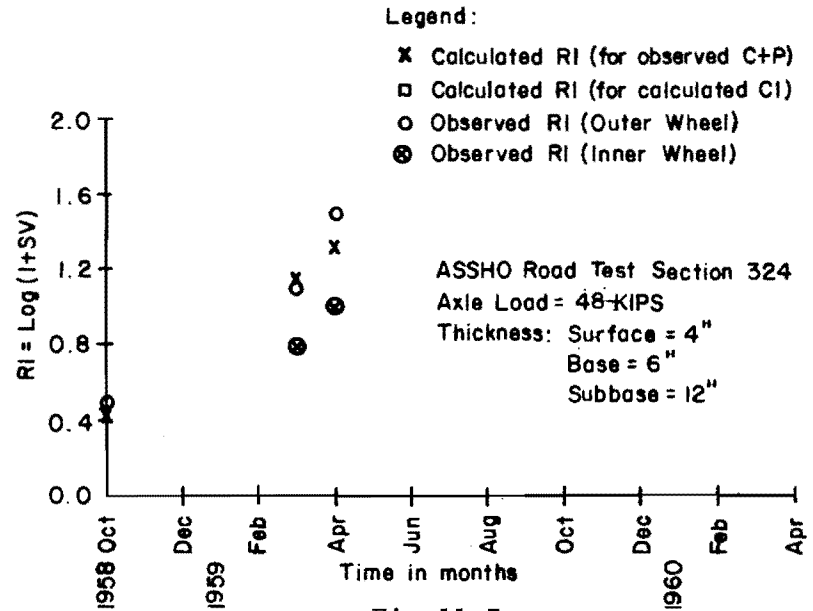


Fig 11.7.

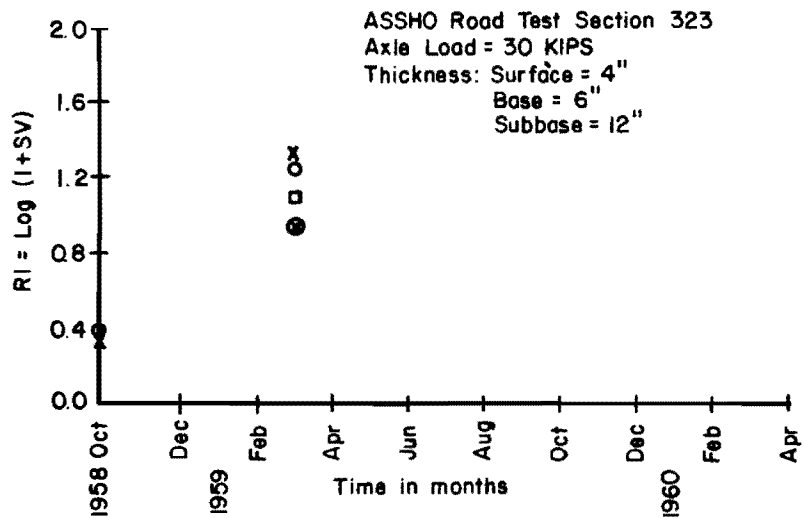


Fig 11.8.

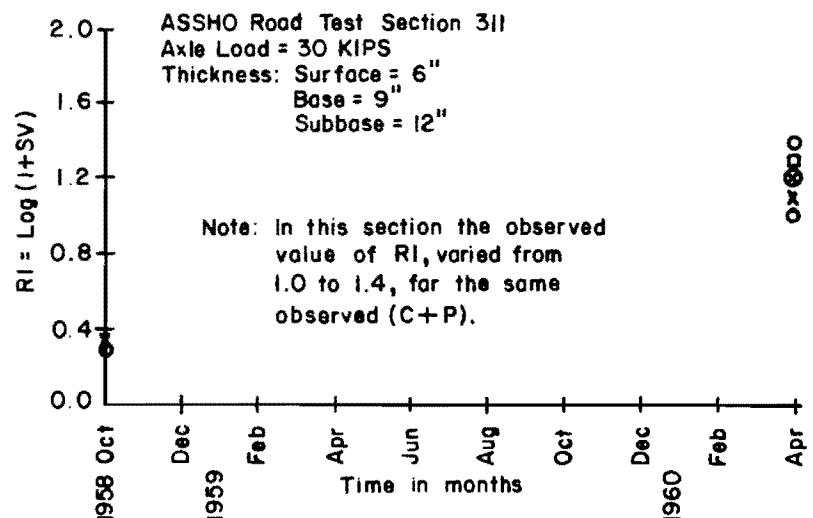


Fig 11.9.

Verification of roughness index model.

### Summary

Generally the differences between calculated and observed values are within acceptable limits and the model presented herein gives satisfactory results and can be relied on in the design of flexible pavements.

### VERIFICATION OF THE PERFORMANCE MODEL

The models for quantification of the cracking index (CI), rut depth (RD), and roughness index (RI) were developed in Chapters 8, 9, and 10, respectively. Verification of these models earlier in this chapter proved their applicability to the design procedure for flexible pavements. For comparison of the observed and calculated values of PSI, 28 sections selected for distress index models comparison are also utilized here.

#### Computation of PSI

For selected AASHO Road Test sections, the values of CI, RI, and RD were computed from the models developed in this report. Then PSI values for these sections were calculated from Eq 7.1. The computer program to calculate the cracking index and rut depth index values is given in Appendix A4.2. Another computer program, which calculates the roughness index and present serviceability index, is included in Appendix A4.4.

#### Comparison of the Performance Model

The calculated and observed values of present serviceability index for AASHO Road Test sections are compared as shown in Figs 11.10 and A8.59 through A8.80. In these figures, it can be observed that

- (1) In general, the calculated and predicted values of PSI at the beginning are very close in almost all cases, as are those at the end.
- (2) Fluctuations and some differences in calculated and observed values occur at times other than close to the beginning and end.
- (3) The calculated values always show a decreasing trend, but in some cases the observed values of PSI increase with time for some periods, which is wrong theoretically as well as conceptually. Other than this discrepancy, the trend of loss in serviceability is computed and observed values is the same.
- (4) Of 28 plots, 17 show a maximum difference in any month in calculated and observed values of PSI on the order of 0.3. This difference is about 0.7 in five cases while it is as much as 1.5 in six cases. The mean difference, however, is only 0.15.

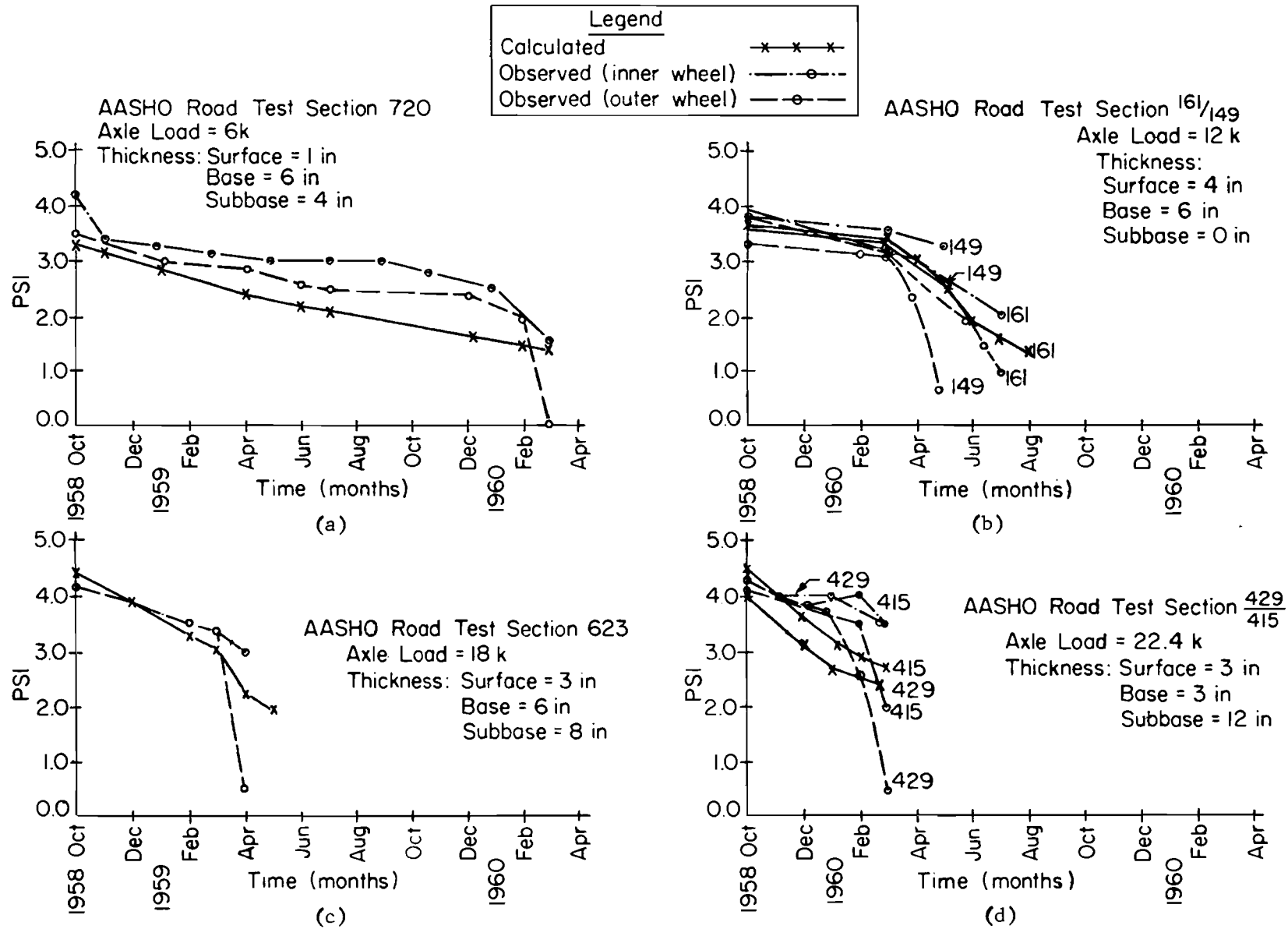


Fig 11.10. Observed versus calculated present serviceability index, psi.

For a comparison of calculated and observed values the following points should be considered:

- (1) At the AASHO Road Test (Ref 70), where the original PSI equation was derived, based on the present serviceability ratings of 74 data points, the difference between PSR and PSI was as large as 1.1, with a mean value of 0.3. Therefore, it seems that any difference between calculated and observed values of  $\pm 0.3$  is not significant, and in some cases a difference as high as 1.1 can be tolerated.
- (2) In the theoretical calculations, not all the local and temporary conditions affecting the PSI can be taken into account, for example, a brief local rainfall or snowfall. Therefore, large fluctuations at times other than at the beginning and end of the performance period are likely to occur.
- (3) The effects of embankment swelling, nonload-associated effects, and temperature stresses were not taken into account. When these effects are included, closer predictions are expected. The work to quantify these effects is already in progress and is the subject of another report.
- (4) The increase in PSI at any time in the performance period cannot be explained theoretically or logically. In observed measurements it increased very often, which may be due to errors in the measurements or, sometimes, to local and short smooth surfaces. When this discrepancy in the observation is removed, better agreement between the calculated and observed values will result.
- (5) During certain times of the year, especially hot months, measurements of cracking are likely to be wrong (Ref 70) because of temporary invisibility of the cracks. This may lead to lower and false observed values of the PSI. During some periods the measured values of roughness index also decreased instead of increasing with load repetitions. Some of the large differences in observed and calculated values of PSI are in fact due to these discrepancies in observed values (Figs A8.63, A8.69, A8.70, A8.72, A8.74, A8.75, A8.78, and A8.79) and the observed values, instead of going down as expected, either went up or stayed the same, causing bigger differences. Correction of this situation will improve the agreement between observed and calculated values.
- (6) The general trend of the performance curve and the values of PSI calculated and observed at the beginning and end are very close in most cases.
- (7) Differences in some cases occurred in the field because of the time phase difference phenomenon, which was explained for the cracking index. Sections showing large differences in PSI are the same as those which showed large differences in cracking index.
- (8) The replicate sections in the AASHO Road Test did not show the same performance (Figs 11.10, A8.63, A8.64, A8.69, A8.72, A8.74, A8.75, A8.77, and A8.78). Therefore, it can be expected that the observed performance data will deviate from the calculated performance values obtained from any theoretical model. However, a model may be

considered satisfactory which results in residual errors that average about the same as the deviations of the replicate observations from their mean. The same type of criteria were adopted in the AASHO Road Test for choosing a satisfactory model (Ref 70, p 43).

The performance model developed in the AASHO Road Test gave a mean prediction error in the performance value of PSI for various loops of 0.39 to 0.75, with an overall mean of 0.53 (Ref 70, Table 11). There is no mention of the extent of the maximum errors of predictions involved in the model. However, when the mean error in a loop is 0.75, the maximum prediction error can be more than 1.0 and up to 1.5.

For the AASHO Road Test, a mean replicate observed difference in PSI was reported as 0.46 (Ref 70, p 43). From the observation of various replicate sections in this report, the PSI values were different by even more than 1.0 at various times. This difference was 2.0 or greater at or close to the time of failure in some test sections.

The replication difference given by the panel in the PSR ranged as high as 0.5 (Ref 70, Table 1.F, pp 295 and 306). It was noted that this replicate difference was observed when ratings were made on successive days and it is possible that replicate PSR's would differ even more over a longer interval of time. The standard deviation of the individual PSR value for each section is 0.5, which shows that 3 ratings out of 10 will be even more than 0.5 rating points from the panel mean PSR.

- (9) In the spring of 1971 a team of graduate students from the Center for Highway Research at The University of Texas at Austin was sent out to measure the performance data of some of the highways in Texas. They were also instructed to assign the rating values to these highways in accordance with the AASHO Road Test procedure. A difference between the ratings of the students as high as 1.1 was noted, and the difference in PSI and PSR was as high as 1.6. A difference in PSI and PSR up to 1.0 was very common.
- (10) Figure 11.11 shows the calculated monthly values of the present serviceability index (PSI) against the observed values for all test sections shown in Fig 11.1. The overall mean values of observed and predicted PSI values are 3.28 and 3.13, respectively. The difference in mean values is only 0.15, as compared to 0.3 mean difference between PSR and PSI (paragraph (1) above) and a mean observed replicate difference of 0.46 (paragraph (8) above). Less than 10 percent of the points fall outside the  $\pm 0.75$  lines (paragraph (8) above). The correlation coefficient between the predicted and observed values is 0.872 and the mean absolute residual is 0.43.

### Summary

Based on the discussions in the previous section of this chapter and the results using the models developed in this report, it may be concluded that the predictions are well within the expected accuracy and discussed criteria.



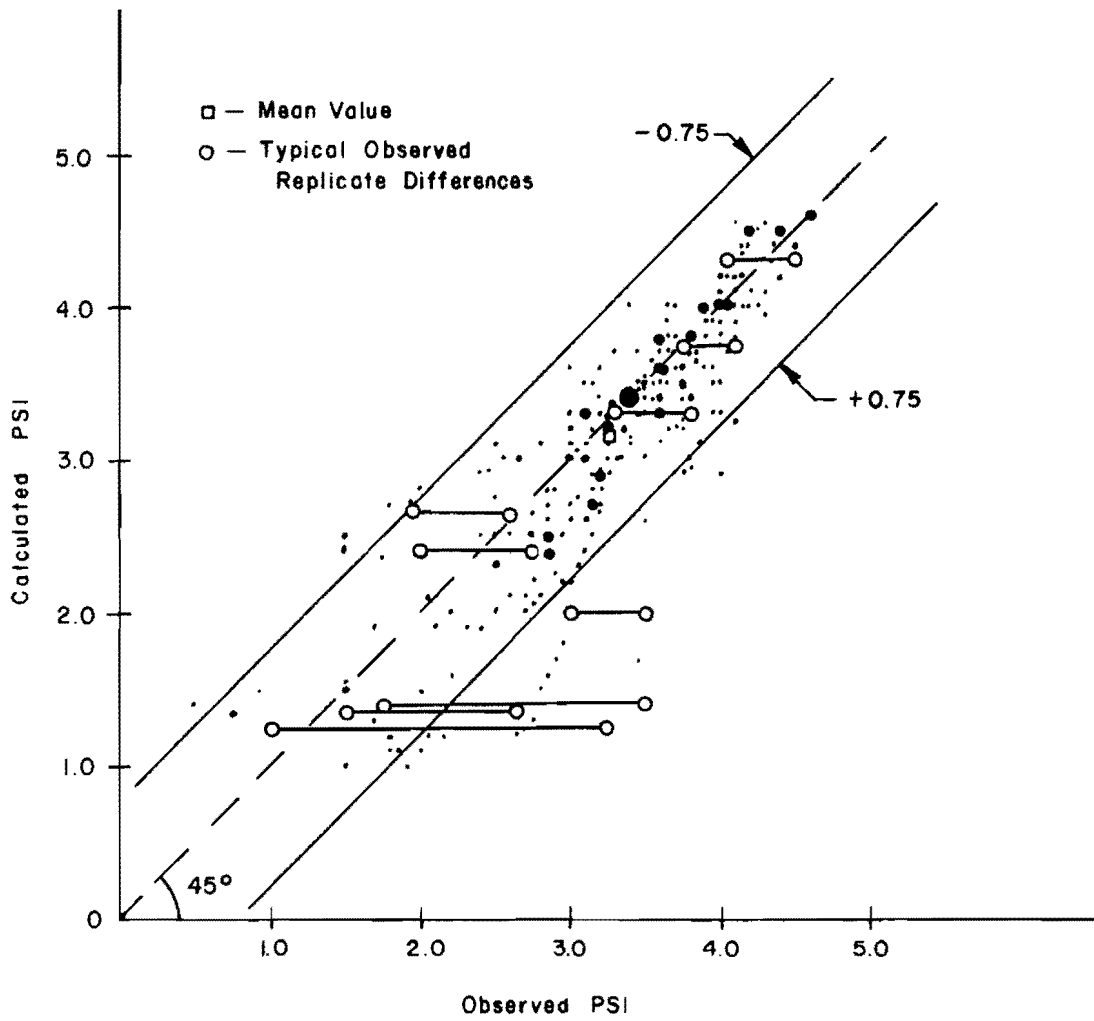


Fig 11.11. Comparison of observed and computed present serviceability index (psi) of various test sections.

Any major differences can be explained in all cases. Therefore, these models should be considered satisfactory for use in the design of the flexible pavements.

PART IV

VERIFICATION AND PROPOSED USE OF MODELS

## CHAPTER 12. SENSITIVITY ANALYSIS

Verification of the developed models in Chapter 11, and predictions which compare well with observed data give confidence in the use of the proposed models. In this chapter, results of a sensitivity analysis for the cracking index model are reported. The purposes of the sensitivity analysis are to

- (1) establish confidence and reliability in the models,
- (2) improve the understanding of the models,
- (3) debug the computer program,
- (4) establish the relative significance of the input variables,
- (5) simplify the computer program by eliminating or fixing variables,  
and
- (6) establish guidelines and precautions for the use of the models.

It is recognized that a designer has only limited time and resources to spend in estimating the large number of inputs needed in the proposed procedure. Therefore, the more important inputs in determining the optimum cost and design should be estimated with greater precision and accuracy than the others. The conclusions in this chapter are based on broad general observation and do not hold in all cases. One effective method of determining the relative significance of the parameters in a complex model is to perform a sensitivity analysis by evaluating the amount of response in a model due to a unit change in the parameters. The interactions of the variables must be studied for a complete sensitivity analysis.

### Description of Analysis

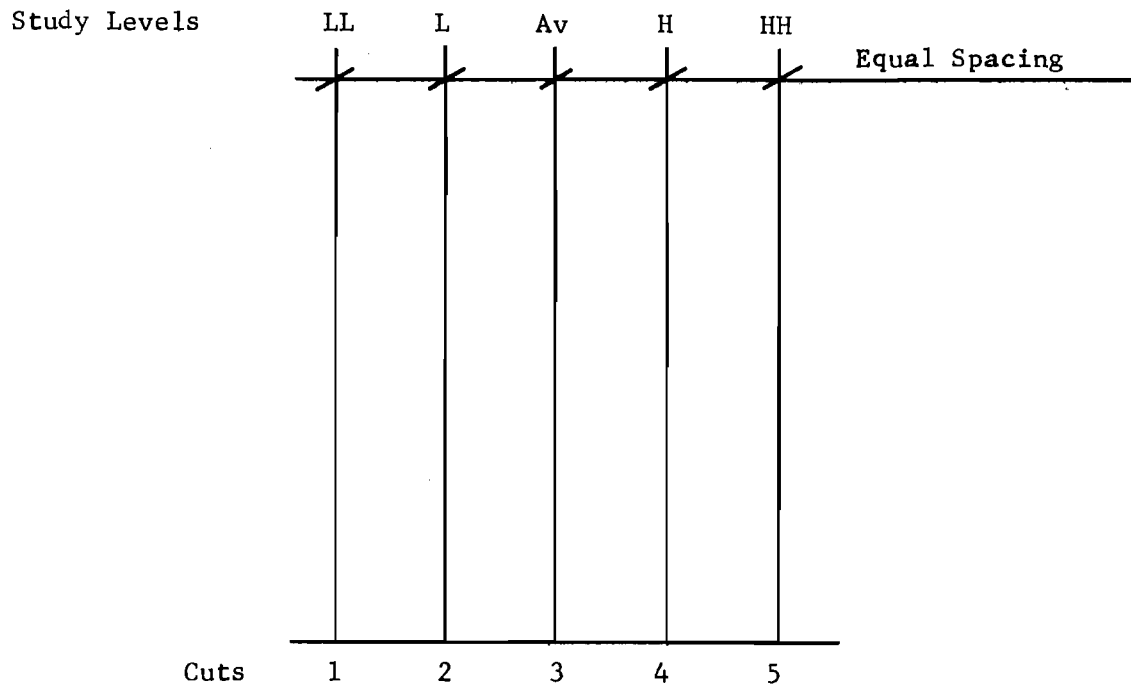
About 35 main parameters are involved in the present fatigue models developed in the report. Complete sensitivity analysis is a major task and is not considered herein, but it is recommended that it be made at the earliest opportunity. The most important and significant mathematical model developed in the present study is the cracking index model. Therefore, a

limited sensitivity analysis was made to study the effect of different parameters on cracking index values and verify the suitability of the model.

Results of detailed sensitivity analyses on FPS models reported in Refs 89, 92, and 93 provide background for such a study.

A complete sensitivity study would require an analysis of designs and costs at all levels of the possible ranges of the variables involved. Such an analysis of variance would have to be performed on a very large scale to cover the effects of individual variations of the variables as well as their variations in groups. To study all the possible interactions of variables, an experiment would have to be set up to solve the number of problems given by the full factorial of 35 variables. Such a large scale experiment is not feasible from either a solution time or a data analysis point of view.

Therefore, an experiment had to be formulated which could be done within reasonable time and with a reasonable amount of effort and would give the maximum information desired to effectively use the developed computer program and to attain the required confidence for using the proposed models for actual field problems. A five-level experiment (Fig 12.1), as discussed in Ref 93, is desirable for a sensitivity study of a model having a large number of variables, the type proposed in this report. However, for the sensitivity study of small magnitude reported herein, a three-level experiment (Fig 12.2) was selected. This experiment would isolate the effects of individual variables by varying one variable while the rest are held constant. The experiment was designed by giving each variable, based on engineering judgments, its low, average, and high magnitude value, as shown in Table 12.1. For example, in the present study the low and high values of the elastic modulus of different materials are varied by 25 percent, i.e., about one standard deviation, either way to study the effect of the variation. These elastic modulus values do not represent the real low and high values of this parameter. A detailed sensitivity study should consider the actual variation from low low values to high high values expected in real situations for all the parameters. One basic solution was then obtained keeping all the variables at the average level. The variations were then studied in the average cut, and two more problems were studied for every variable. These problems involved all the variables at their average levels except that the one under study was given its low and high value for the two problems. In a detailed three-level experiment similar studies should be made for the low



Where

LL = low low values

L = low values

Av = average values

H = high values

HH = high high values

Number of Problem Solutions  $N$  is given by

$$N = C + V \times C (S - 1)$$

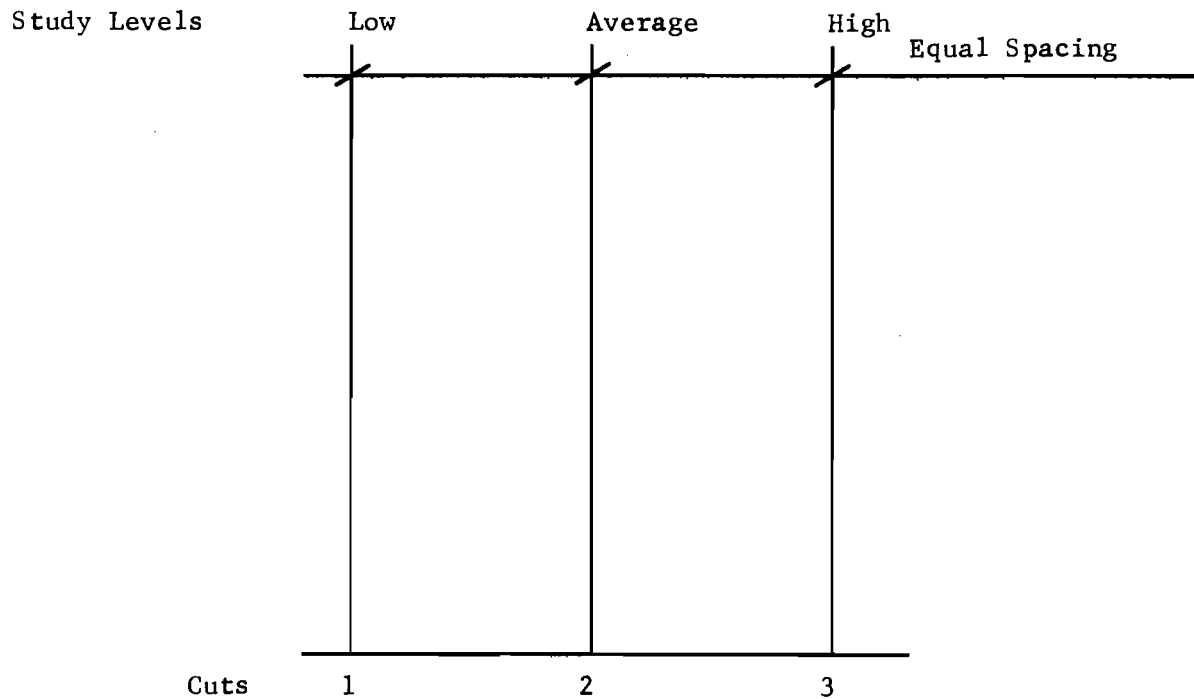
where

$C$  = number of cuts

$V$  = number of variables

$S$  = number of study levels

Fig 12.1. Typical five - level experiment for sensitivity analysis.



Where

L = low value

Av = average value

H = high value

Number of problem solutions  $N$  is given by

$$N = C + V \times C (S - 1)$$

where

$V$  = number of variables

$S$  = number of study levels

$C$  = number of cuts

Note: In the present study the variables were studied at the average cut only ( $C = 1$ ), for which the number of solutions are:

$$1 + V \times 1 \times 2 = 2V + 1$$

Fig 12.2. Typical three-level experiment for sensitivity analysis.

TABLE 12.1. ASSIGNED LOW, AVERAGE AND HIGH MAGNITUDE  
VALUES OF THE PARAMETERS

SR/NO	Variable	Value		
		Low	Average	High
1	Single Axle Load (Kip)	7	12	17
2	Tire Pressure (lbs/in <sup>2</sup> )	42.3	65.7	67.5
3	Thickness (inches)			
	(a) A. Concrete	3.0	5.0	7.0
	(b) Base	3.0	6.0	9.0
	(c) Subbase	9.0	12.0	15.0
4	Mean Elastic Modulus E values (lbs/in <sup>2</sup> )			
	(a) Concrete			
	Jan.	12.00×10 <sup>5</sup>	16.00×10 <sup>5</sup>	20.00×10 <sup>5</sup>
	Feb.	10.60×10 <sup>5</sup>	14.20×10 <sup>5</sup>	17.60×10 <sup>5</sup>
	Mar.	10.28×10 <sup>5</sup>	13.70×10 <sup>5</sup>	17.12×10 <sup>5</sup>
	Apr.	6.75×10 <sup>5</sup>	9.00×10 <sup>5</sup>	11.25×10 <sup>5</sup>
	May	4.50×10 <sup>5</sup>	6.00×10 <sup>5</sup>	7.50×10 <sup>5</sup>
	June	3.00×10 <sup>5</sup>	4.00×10 <sup>5</sup>	5.00×10 <sup>5</sup>
	July	2.62×10 <sup>5</sup>	3.50×10 <sup>5</sup>	4.37×10 <sup>5</sup>
	Aug.	2.25×10 <sup>5</sup>	3.00×10 <sup>5</sup>	3.70×10 <sup>5</sup>
	Sep.	3.15×10 <sup>5</sup>	4.20×10 <sup>5</sup>	5.25×10 <sup>5</sup>
	Oct.	4.92×10 <sup>5</sup>	6.50×10 <sup>5</sup>	8.12×10 <sup>5</sup>
	Nov.	6.27×10 <sup>5</sup>	8.30×10 <sup>5</sup>	10.37×10 <sup>5</sup>
	Dec.	9.82×10 <sup>5</sup>	13.10×10 <sup>5</sup>	16.37×10 <sup>5</sup>
	(b) Base			
	Jan.	18000	24000	30000
	Feb.	18000	24000	30000
	Mar.	18000	24000	30000

(Continued)



TABLE 12.1. (Continued)

SR/NO	Variable	Value		
		Low	Average	High
	Apr.	11700	15600	14500
	May	11000	18000	22500
	June	14700	19600	24500
	July	16200	21600	27000
	Aug.	17400	23200	29000
	Sep.	18000	24000	30000
	Oct.	18000	24000	30000
	Nov.	18000	24000	30000
	Dec.	18000	24000	30000
(c)	Subbase			
	Jan.	9900	13200	16500
	Feb.	9900	13200	16500
	Mar.	9900	13200	16500
	Apr.	5400	7200	9000
	May	6450	8600	10750
	June	7100	9800	12250
	July	8100	10800	13500
	Aug.	8700	11 00	15500
	Sep.	9150	12200	15250
	Oct.	9400	12400	15600
	Nov.	9600	12800	16000
	Dec.	9900	13200	16500
(d)	Subgrade			
	Jan.	4950	6600	8250
	Feb.	4950	6600	8250
	Mar.	4950	6600	8250
	Apr.	2700	3600	4500
	May	3225	4300	5375

(Continued)

TABLE 12.1. (Continued)

SR/NO	Variable	Value		
		Low	Average	High
	June	3675	4900	6125
	July	4050	5400	6750
	Aug.	4350	5800	7250
	Sep.	4575	6100	7625
	Oct.	4650	6200	7750
	Nov.	4800	6400	8000
	Dec.	4950	6600	8250
5	Poisson's Ratio ( $\mu$ ) held constant			
	(a) A. Concrete	0.3	0.3	0.3
	(b) Base	0.4	0.4	0.4
	(c) Subbase	0.45	0.45	0.45
	(d) Subgrade	0.5	0.5	0.5
6	Confidence $\alpha$ for E-values ( $E_\alpha$ )			
	(a) A. Concrete	.095	.18	.265
	(b) Base	.095	.18	.265
	(c) Subbase	.095	.18	.265
	(d) Subgrade	.095	.18	.265
7	Coefficient of variation "V <sub>v</sub> " of E values			
	$\left( V_v = \frac{\sigma_E \times 100}{\bar{E}_v} \right)^*$			
	(a) A. Concrete	12.5	20.0	27.5
	(b) Base	12.5	20.0	27.5
	(c) Subbase	12.5	20.0	27.5
	(d) Subgrade	12.5	20.0	27.5

(Continued)

TABLE 12.1. (Continued)

SR/NO	Variable	Value		
		Low	Average	High
8	Values for parameters in fatigue equations:  $N = A \left( \frac{1}{\epsilon} \right)^\beta **$ and $N_\alpha = N_{50} - \log SD.K$			
	(a) "A" values	$10^{-9}$	$10^{-8}$	$10^{-7}$
	(b) "B" values	3.35	3.9	4.45
	(c) Log SD	0.25	0.3	0.35
	(d) Confidence $\alpha$ for "N"	0.095	0.18	0.265
9	Monthly load applications "N" Actual	20750	40500	60250

\*  $\bar{E}_V$  = Mean value of elastic modulus

$\sigma_{E_V}$  = Standard deviations in elastic modulus

\*\* See Chapters 4 and 9.

cut with the variable being studied given its average and high level and all the other variables kept at low levels. A similar procedure should be used for high cut, in which the variables are studied at their average and low values. A five-level experiment needs similar study at five cuts (Fig 12.1). In the present experiment the procedure required, in all, that 31 problems be solved for fifteen variables. One additional solution for the fatigue parameter B, considered to be most significant, was run for the value as 2.9. Two more solutions for all variables at their high and low values were also run. Thus the total number of solutions was 34. In Table 12.2 the cumulative final value of the cracking index (CI) of each problem after 36 months for the given monthly traffic is expressed as a percentage of the corresponding CI value of the average problem, with all parameters at the average level (Ref 171). These values of the cracking index were obtained using the computer program for cracking index, included in Appendix 4.

## RESULTS

Figure 12.3 shows the plot, for each variable, of cracking index versus the percentage of the variable in terms of its average value (Table 12.2). The relative slopes of these plots indicate the comparative significance of each variable. The plots also help suggest the comparative qualitative significance of different variables and give an initial indication of the sensitivity or rating of the variables. For example, because the slope of the curve for fatigue parameter B (Curve No. 23-24) is steepest on both sides of the average value (100), this parameter is considered most significant (significance 1) at both levels. Similarly thickness of subbase (Curve No. 7-8) is considered 14 in significance, one level above the least significant variable.

A more quantitative approach to assigning the significance of various parameters is shown in Table 12.3. For many plots in Fig 12.3, it is difficult to define any regular trend of slopes from the low side to the high side, i.e., from one end of the plot to the other. Therefore, it seems reasonable at this stage to define a constant variation in two parts, i.e., from low to average and from average to high. The plots are not always uniform and straight but in most cases, based on this subdivision, reasonable indication of the relative significance of the various variables is expected. Cracking index values for a one percent increase or decrease in average value of a particular parameter are calculated (col 4, Table 12.3) representing the slope of the

TABLE 12.2. CRACKING INDEX VALUES FOR VARIOUS PROBLEMS

No.	Variable	Final Value of CI in sq ft per 1000 sq ft with Variable at		CI Expressed as Percent of Average Value = $\frac{(2) \text{ or } (3)}{40.7} \times 100$		Curve Number Figs 12.3 & 12.4
		Low	High	Low	High	
	1	2	3	4	5	6
1	Axle Load	0.322	107.0	0.7	260.0	1-2
2	Thickness of Asphalt Concrete	251.0	1.22	610.0	2.0	3-4
3	Thickness of Base	56.3	24.0	138.0	58.0	5-6
4	Thickness of Subbase	47.2	35.9	115.0	88.0	7-8
5	Elastic Modulus E of Concrete	88.3	19.7	216.0	48.0	9-10
6	Elastic Modulus E of Base	60.9	27.9	149.0	68.0	11-12
7	Elastic Modulus E of Subbase	48.5	34.9	119.0	85.0	13-14
8	Elastic Modulus E of Subgrade	42.9	14.3	105.0	35.0	15-16
9	Confidence Value for Elastic Modulus E $\alpha$	85.7	17.5	210.0	42.0	17-18
10	Coefficient of Variation for Elastic Modulus E $\alpha$	12.6	79.4	30.0	195.0	19-20
11	Fatigue Parameter "A"	397.0	0.633	975.0	1.5	21-22

(Continued)

TABLE 12.2. (Continued)

No.	Variable	Final Value of CI in sq ft per 1000 sq ft with Variable at		CI Expressed as Percent of Average Value = $\frac{(2) \text{ or } (3)}{40.7} \times 100$		Curve Number Figs 12.3 & 12.4
		Low	High	Low	High	
	1	2	3	4	5	6
12	Fatigue Parameter "B"*	905.0	0.00686	2220.0	0.01	23-24
13	Log Standard Deviation of Fatigue Curve	29.9	52.4	70.0	120.0	25-26
14	Confidence Level for Fatigue Curve $N \alpha$	18.3	82.3	40.0	202.2	27-28
15	Actual Number of Load Repetitions "N"	14.9	68.5	36.6	168.3	29-30

## Notes:

1. Value of cracking index (CI) for the case when
  - a. all variables are at their average value is 407,
  - b. all variables are at their low values is 1000,
  - c. all variables are at their high values is  $1.46 \times 10^{-5}$ .
2. For typical pavement under consideration see Fig 8.4.

\* Final value of CI for value of B as 2.9 is 1000.

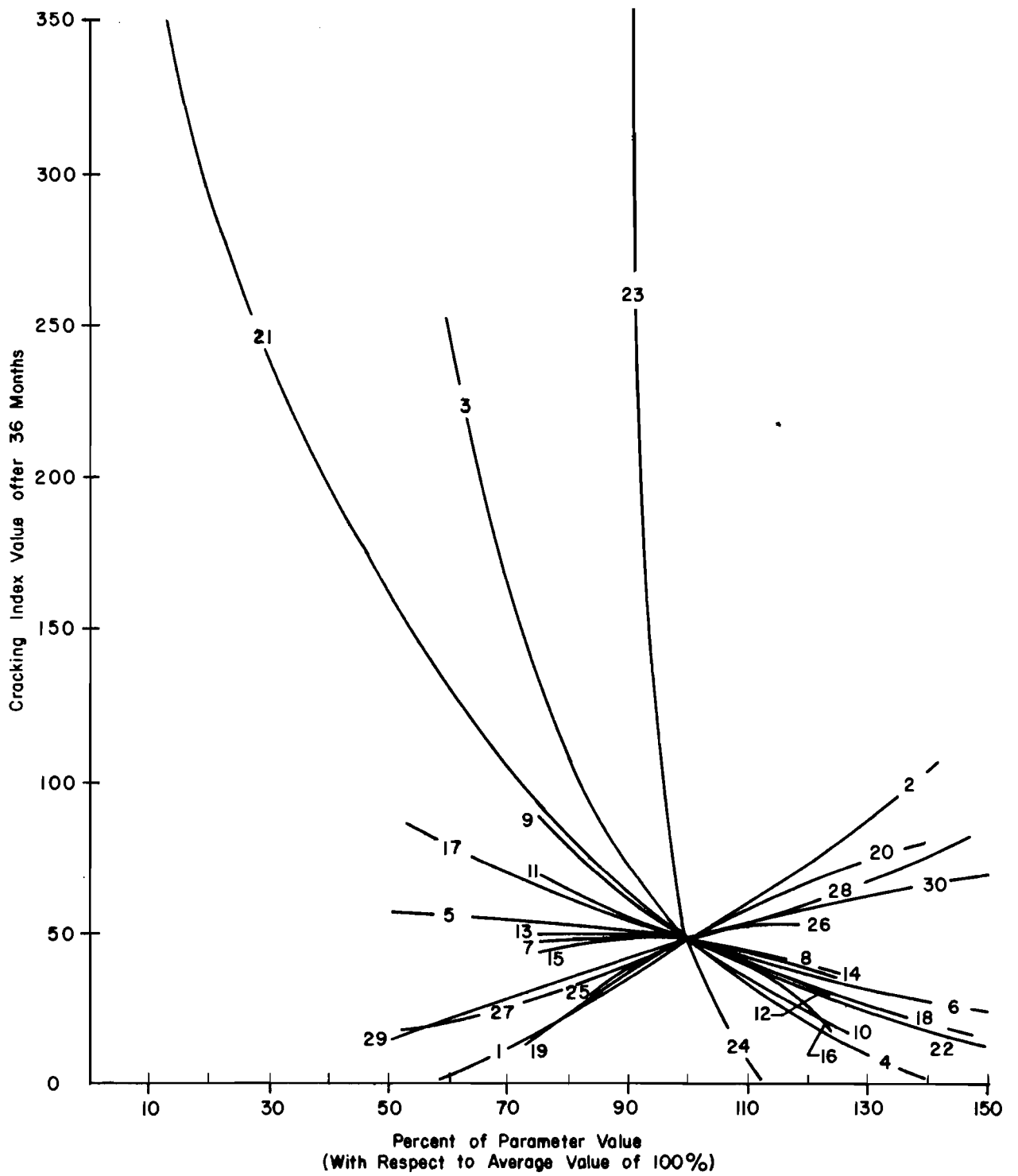


Fig 12.3. Plots showing relative significance of various parameters.

TABLE 12.3. PROCEDURE TO GET THE RELATIVE SIGNIFICANCE OF VARIABLES

Parameter's Name	X = % Increase or Decrease in Parameter Value From Average 100.0	Y = Final CI of the Problem- Final CI of Average (CI = 40.7)	CI Value per 1% of Increase or Decrease in Average Parameter Value Slope = Y/X	Sequence Nos. in Order of Decreasing Slope		Curve or Problem No. (Fig 12.3)
				Lowside of Average Value of Para.	Highside of Average Value of Para.	
1	2	3	4	5	6	7
Axle Load (L)	-41.8	-40.378	0.970	5		1
Axle Load (H)	+41.8	+66.3	1.586		2	2
Thickness AC (L)	-40.0	+210.3	-5.260	2		3
Thickness AC (H)	+40.0	-39.48	-0.987		5	4
Thickness Base (L)	-50.0	+15.60	-0.312	12		5
Thickness Base (H)	+50.0	-16.70	-0.334		12	6
Thickness Subbase (L)	-25.0	+6.50	-0.260	14		7
Thickness Subbase (H)	+25.0	-4.80	-0.192		14	8
Elastic Modulus $E_{AC}$ (L)	-25.0	+47.60	-1.905	4		9
Elastic Modulus $E_{AC}$ (H)	+25.0	-21.00	-0.840		7	10
Elastic Modulus $E_{Base}$ (L)	-25.0	+20.20	-0.808	7		11
Elastic Modulus $E_{Base}$ (H)	+25.0	-12.80	-0.5115		10	12

(Continued)



TABLE 12.3. (Continued)

Parameter's Name	X = % Increase or Decrease in Parameter Value From Average 100.0	Y = Final CI of the Problem-Final CI of Average (CI = 40.7)	CI Value per 1% of Increase or Decrease in Average Parameter Value Slope = Y/X	Sequence Nos. in Order of Decreasing Slope		Curve or Problem No. (Fig 12.3)
				Lowside of Average Value of Para.	Highside of Average Value of Para.	
1	2	3	4	5	6	7
Elastic Modulus $E_{\text{Subbase}}$ (L)	-25.0	+7.80	-0.312	13		13
Elastic Modulus $E_{\text{Subbase}}$ (H)	+25.0	-5.80	-0.232		13	14
Elastic Modulus $E_{\text{Subgrade}}$ (L)	-25.0	+2.20	-0.0879	15		15
Elastic Modulus $E_{\text{Subgrade}}$ (N)	+25.0	-26.40	-1.056		3	16
Confidence Value for Elastic Modulus $E_{\alpha}$ (L)	-47.3	+45.00	-0.9515	6		17
Confidence Value for Elastic Modulus $E_{\alpha}$ (H)	+47.3	-23.20	-0.4910		11	18
Coefficient of Variance (L)	-37.5	-28.10	+0.07490	8		19

(Continued)

TABLE 12.3. (Continued)

Parameter's Name	X = % Increase or Decrease in Parameter Value From Average 100.0	Y = Final CI of the Problem- Final CI of Average (CI = 40.7)	CI Value per 1% of Increase or Decrease in Average Parameter Value Slope = Y/X	Sequence Nos. in Order of Decreasing Slope		Curve or Problem No. (Fig 12.3)
				Lowside of Average Value of Para.	Highside of Average Value of Para.	
1	2	3	4	5	6	7
Coefficient of Variance (H)	+37.5	+38.70	+1.033		4	20
Fatigue Parameter A-Value (L)	-90.0	+356.3	-3.962	3		21
Fatigue Parameter A-Value (H)	+90.0	-40.067	-0.044519		15	22
Fatigue Parameter B-Value (L)	-14	+864.30	-61.700	1		23
Fatigue Parameter B-Value (H)	+14	-40.693	-2.908		1	24
Standard Deviation of Fatigue Curve Log SD (L)	-16.6	-10.80	+0.650	9		25
Standard Deviation of Fatigue Curve Log SD (H)	+16.6	+11.70	+0.705		8	26

(Continued)

TABLE 12.3. (Continued)

Parameter's Name	X = % Increase or Decrease in Parameter Value From Average 100.0	Y = Final CI of the Problem-Final CI of Average (CI = 40.7)	CI Value per 1% of Increase or Decrease in Average Parameter Value Slope = Y/X	Sequence Nos. in Order of Decreasing Slope		Curve or Problem No. (Fig 12.3)
				Lowside of Average Value of Para.	Highside of Average Value of Para.	
Confidence Level for Fatigue Curve Number of Load Rep.: $N_{\alpha}$ (L)	-47.3	-22.40	+0.474	11		27
Confidence Level for Fatigue Curve Number of Load Rep.: $N_{\alpha}$ (H)	+47.3	+41.60	+0.8805		6	28
Actual Number of Load Repetitions "N" (L)	-48.5	-25.80	-0.528	10		29
Actual Number of Load Repetitions "N" (H)	+48.5	+27.80	+0.5725		9	30
Fatigue Parameter B-Value (LL)	-28.2	+959.3	-30.00			31

curves in Fig 12.3. The relative values of this slope then represent the relative significance of each parameter as entered in cols 5 and 6. The variables are arranged in order of decreasing significance in Table 12.4.

These procedures not only give the qualitative and quantitative importance of each variable, but also give the relative order of importance of the variables. However, these results are considered to be limited since they do not consider all the interactions between the different variables. A complete factorial experiment for all interactions is not possible because of the large number of variables. However, the detailed sensitivity study recommended earlier is expected to give more dependable results in a wide variety of situations. In some cases it may be advisable to run a sensitivity analysis for each design problem. It may further be advisable to compare the variation in the output due to one standard deviation in each variable rather than on the basis of 1% increase or decrease and assign the significance on this basis. However, before this can be done a study of the expected variations to calculate the standard deviation for each variable is required and is a field opened for future research.

Figure 12.4 shows curves for all parameters at various levels against the cracking index expressed as a percent of average value. For true high and low values the ranges shown would represent 100 percent variations and the actual relative significance of the various parameters would be shown. However, extreme high and low values were not considered in this study and the relative significance of the parameters shown is only for the specific values of the parameters considered in this experiment. In the detailed sensitivity study of the five-level experiment, low low and high high values represent true extreme variations of each parameter.

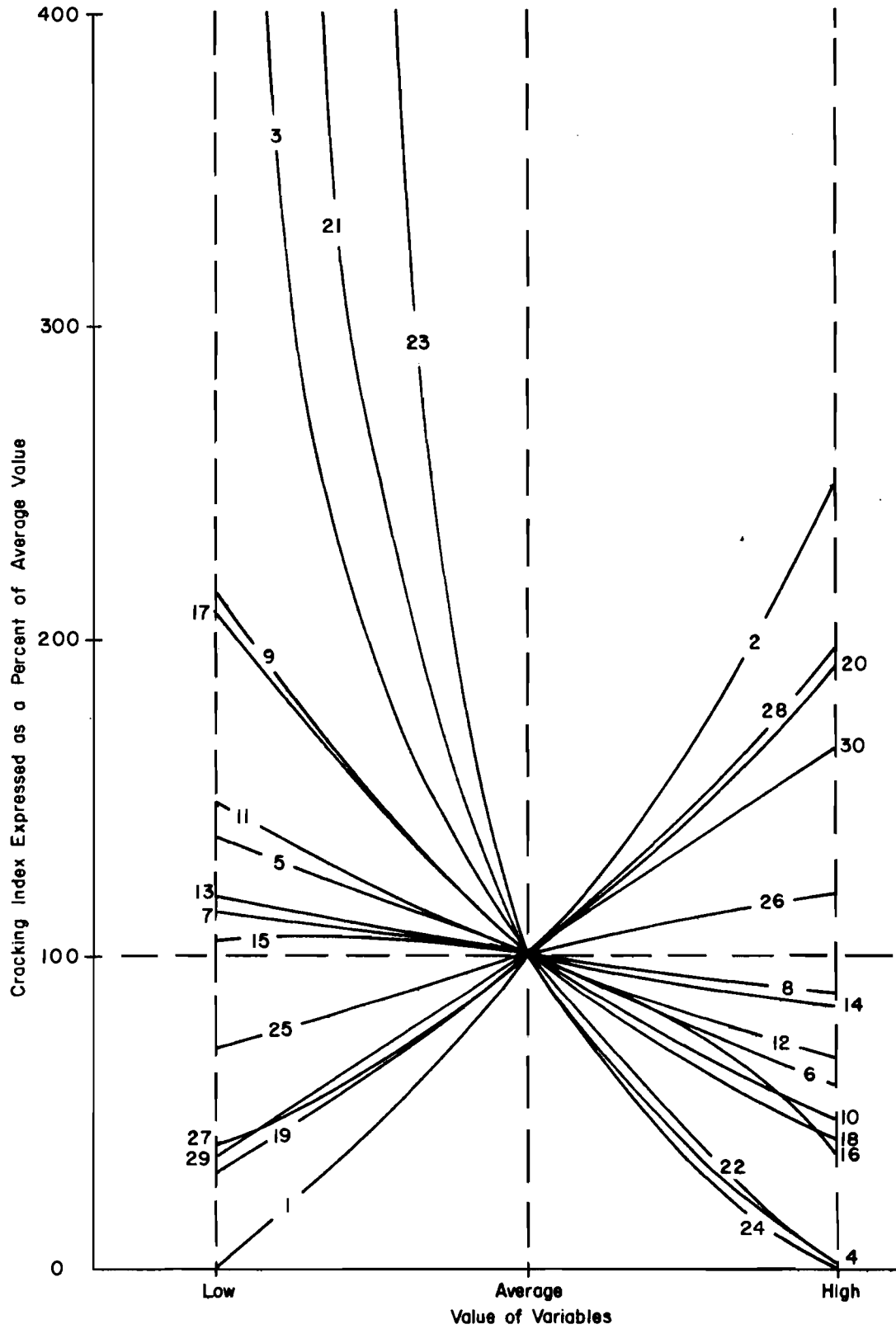
#### OBSERVATIONS

The following tentative observations are made from this short sensitivity study:

- (1) Fatigue parameter B in the fatigue equation 4.1, is the most significant variable and has the maximum effect on the CI values. Other important parameters are thickness of asphaltic concrete fatigue parameter A, axle load, and modulus of subgrade.
- (2) The effect of the resilient modulus of subgrade is least on the low side and is quite significant on the high side.

TABLE 12.4. RELATIVE SIGNIFICANCE OF THE VARIABLES

Sequence Number	Parameters Arranged in the Order of Effect	
	Low Side (L)	High Side (H)
1	Fatigue Parameter "B"	Fatigue Parameter "B"
2	Thickness of Asphalt Concrete	Axle Load
3	Fatigue Parameter "A"	Elastic Modulus Subgrade
4	Elastic Modulus Asphaltic Concrete	Coefficient of Variation in Modulus Values
5	Axle Load	Thickness - A.C.
6	Confidence Value for Elastic Modulus	Confidence for Fatigue Curve $N_{\alpha}$
7	Elastic Modulus for Base	Elastic Modulus Asphaltic Concrete
8	Coefficient of Variation in Modulus Values	Log SD for Fatigue Curve
9	Log SD for Fatigue Curve	Actual Traffic Repetitions $N_{\alpha}$
10	Actual Traffic Repetitions "N"	Elastic Modulus for Base
11	Confidence for Fatigue Curve $N_{\alpha}$	Confidence Value for Elastic Modulus
12	Thickness - Base	Thickness - Base
13	Elastic Modulus for Subbase	Elastic Modulus for Subgrade
14	Thickness - Subbase	Thickness - Subbase
15	Elastic Modulus for Subgrade	Fatigue Parameter "A"



Note: Number on the curves correspond to the numbers given in Table 12.2.

Fig 12.4. Variation of parameters versus cracking index expressed as a percent of average value.

- (3) The thickness of asphalt concrete, fatigue parameter A in Eq 4.1, the stiffness modulus of concrete, the confidence level of modulus values, the thickness of base, and the resilient modulus of base have a more pronounced effect on the low side than on the high side.
- (4) Axle load, coefficient of variation for modulus values, confidence value for fatigue curve, standard deviation of fatigue life log SD, and number of actual load repetitions have a more pronounced effect on the high side than on the low side.

The present analysis, which was conducted using only the average values, may not give a true picture of the actual significance of the parameters, and problems with all low values and high values should also be run. For complete analysis, a five-level experiment is recommended. To examine the behavior of the models, a complete sensitivity analysis of all the models should be conducted at the earliest opportunity.

The large number of solutions run during this study not only gave a better understanding of the cracking index model, but also more confidence in the use of the model. The various runs also helped in debugging the computer program at various stages.

The magnitude of this small sensitivity analysis makes it difficult to draw any definite conclusions regarding fixing or eliminating the less important variables. From this study, all the parameters considered variable contributed significantly and none can be fixed or eliminated at this stage. However, this study establishes a criterion of relative significance which can be used to determine the precision which should be applied in estimating each variable. Fatigue parameter B is relatively significant and should be estimated very accurately. The relative significance of various parameters based on the range of values for each variables (Table 12.1) and relative effect on the output for one percent change in the average value of the parameter established on the basis of this study is shown in Table 12.4. Such relative significance is liable to change in certain cases because this type of analysis does not consider complete interaction between parameters.

Detailed criteria for using the cracking index models in the most economical way can be established only on the basis of the complete sensitivity study. In the meantime, several alternate solutions should be run to obtain the most economical design based on the procedure established in the next chapter.

## SUMMARY

In this chapter, a general format for a complete sensitivity analysis was discussed. The results of a sensitivity analysis of the cracking index model were discussed and the relative significance of the variables established, as shown in Table 12.4. Based on this study it is noted that fatigue parameter B is very sensitive and should be estimated very accurately. It is recommended that a detailed sensitivity study of all the models developed in this report should be made as soon as possible.

Verification of the proposed models in Chapter 11 and the sensitivity study in this chapter prove the applicability of these models for design of flexible pavements. The design procedure based on the proposed fatigue models is prescribed in Chapter 13. This design procedure should be used until some parameters can be fixed or eliminated and the amount of computation time reduced, based on a detailed sensitivity study.



## CHAPTER 13. SUMMARY OF PROPOSED FATIGUE MODEL

The theoretical background of the proposed distress index models was explained in earlier chapters, and these models were developed in Chapters 7 to 10. The computer program to calculate the distress indices and pavement performance is included in Appendix 4. The verification of these models with AASHO data that was made in Chapter 11 proved that these models predict the distress indices and performance of a pavement satisfactorily and that the procedure can be used for the design of flexible pavements. In this chapter a summary of the proposed design procedure is presented. Revision of the FPS model for a second generation model is discussed. A comparison of the existing flexible pavement systems computer program with the present design procedure is made by solving example problems.

### PROPOSED FATIGUE MODEL

This section describes direct use of the proposed procedure for the design of flexible pavements. Use of the procedure in the existing FPS is discussed in the next section. The steps required to solve a design problem by the proposed design procedure (Fig 13.1) are:

- (1) Collect data on traffic, materials, and environment. A list of all the input data required is included in Appendix 4. The traffic volume and design period are decided from the traffic record and project planning.
- (2) Characterize Materials: Material characterization is an important part of the whole design process. Material parameters should be ascertained on the basis of laboratory test results, as explained in Chapter 5 and Appendix 2.
- (3) Initial Pavement Condition: The design and performance of the pavement require assumption of the initial slope variance  $SV$  and initial and final values of the pavement PSI. Through an engineering judgment an evaluation is made of these parameters in advance. For example, just after the construction of pavements, the initial value of the roughness index,  $\log(1 + SV)$ , generally varies from 0.3 to 0.6 and the PSI from 4.5 to about 3.8, depending upon the importance of the highway. The terminal PSI varies from 3.0 to 1.5, at which stage either an overlay or a reconstruction will be required.

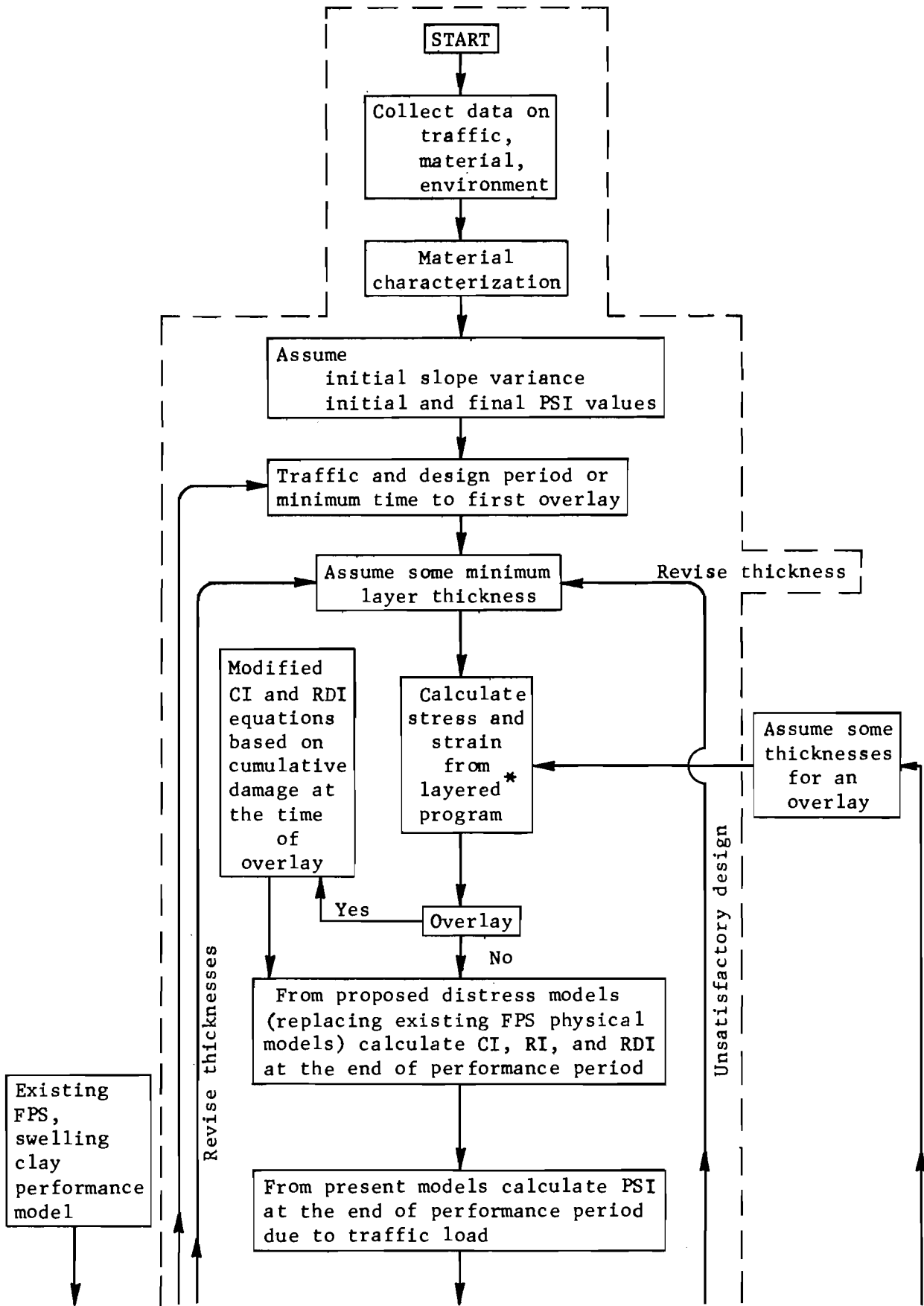
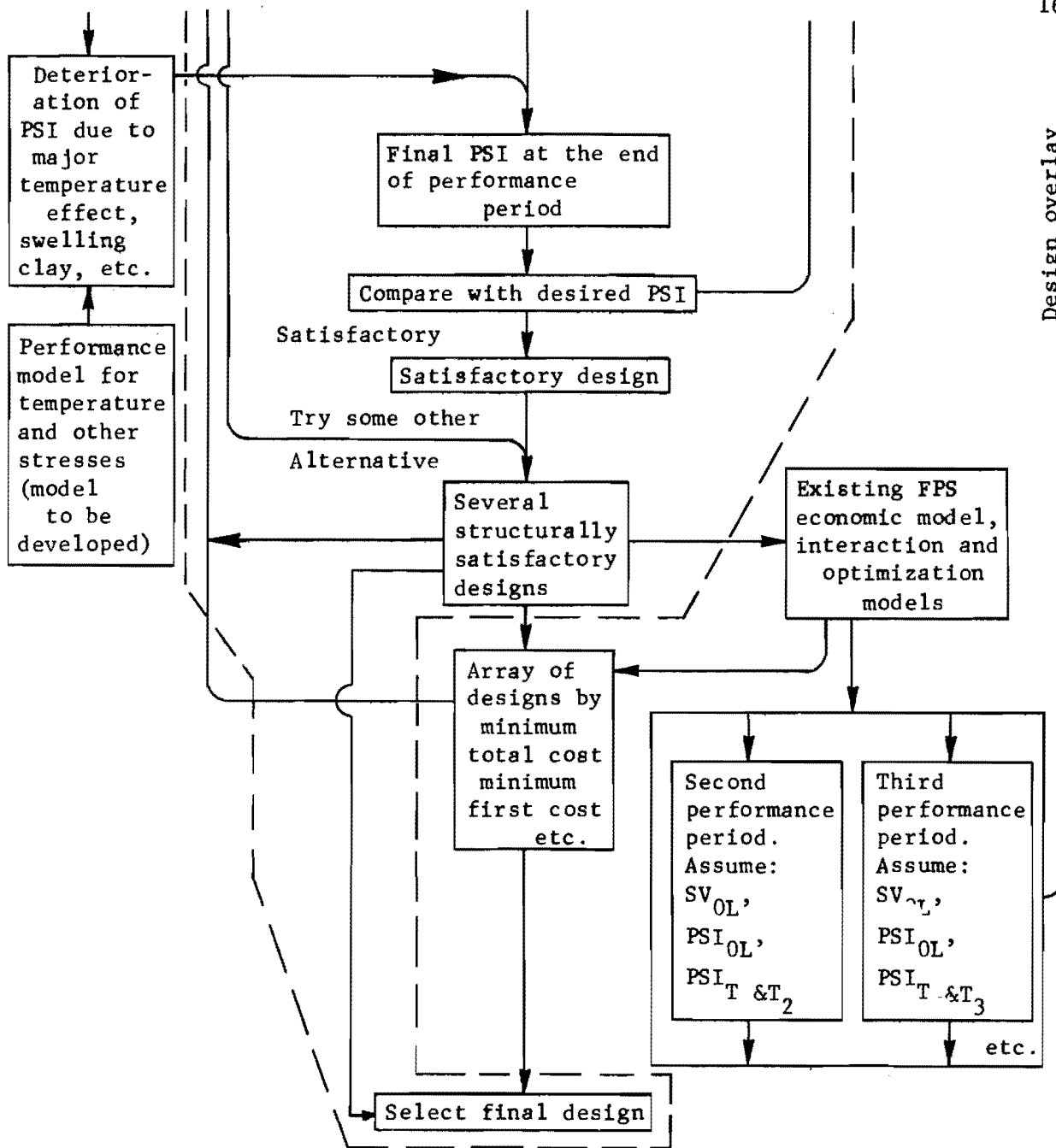


Fig 13.1. Flow diagram for present design procedure and FPS second generation. (Continued)



CI - Cracking index  
 RI - Roughness index  
 RDI - Rut depth index  
 PSI - Present serviceability index  
 SV<sub>OL</sub> - Slope variance just after an overlay  
 PSI<sub>OL</sub> - Present serviceability index just after an overlay  
 PSI<sub>T</sub> - Present serviceability index at the end of performance period

T<sub>i</sub> - Performance period

\* To be replaced later by a more efficient technique for stress-strain calculations

[ ] Shows steps for direct use of the present procedure

Fig 13.1. (Continued).

- (4) Assume Minimum Layer Thickness: Based on engineering judgment, trial layer thicknesses for the pavement section are assumed.
- (5) Calculate Distress Index: Based on the above data and information, the distress index values (CI, RDI, and RI) and pavement performance PSI for the trial traffic and design period are calculated with the distress models and computer program developed in this report (Appendix 4).
- (6) Compare Final PSI: The computed terminal PSI is compared with the desired value. If the values compare within the desired accuracy, the assumed design is satisfactory; otherwise, the layer thickness assumed is revised and another trial comparison is made, until a satisfactory solution is reached. Layer thickness combinations may have to be tried also to find the most economical design. Although several designs, with different layer thickness combinations, may be structurally equal, only one is economically best.

#### PROPOSED FPS SECOND GENERATION

The flow diagram of the proposed FPS second generation (Fig 13.1) shows the proposed procedure included in the existing FPS programs. Mainly, it is a question of replacement of the physical models of the existing FPS. From the new distress index models, PSI due to traffic load is computed at the end of an assumed performance period. Adjustments are made to this PSI for the existing swelling clay model of the FPS and for other nontraffic associated PSI models to be added later. This adjusted PSI is compared with the desired PSI and a satisfactory design is obtained. Several other structurally satisfactory designs can be obtained, and these designs are optimized by the existing FPS economic and other models to get the array of designs for final selection for the no-overlay case. The computations for overlays are made for several performance periods, as shown in Fig 13.1 and all satisfactory designs are again optimized by the existing FPS models to give the final array of design from which final selection is made.

Figure 13.2 shows the existing and proposed generation of working pavement systems. The modified and replaced items are marked. The deflection coefficients have been replaced by elastic constants, fatigue properties, and stochastic variations in these parameters. The deflection term is replaced by stress and strain computations. Instead of a direct empirical deflection versus performance equations in the existing procedure, the distress indices are computed and from the PSI versus distress indices correlation the performance of the pavement is obtained. The overlay computations are made as shown in

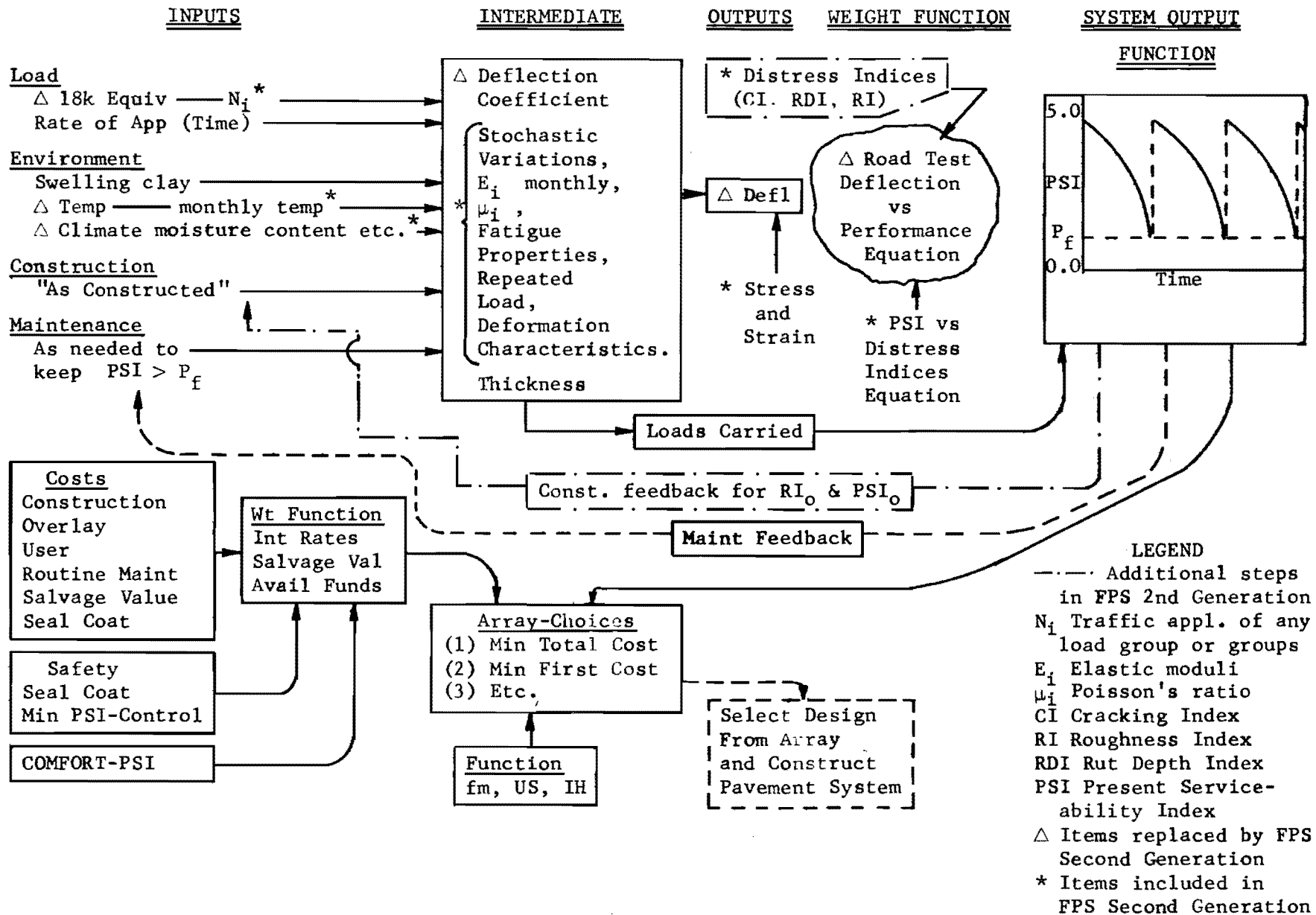


Fig 13.2. Existing and Second Generation FPS (after Ref 81).

Fig 13.1, with proper consideration of fatigue damage at the time of the overlay. The cost models, which compute the array-choices, are not changed.

#### Computation of Stress and Strain

At present, the computer program developed in this report takes about 60 seconds to solve one problem. In solving one problem the computation for strain is made 24 times while the computation for stresses goes through the layered subroutine 12 times. Therefore, most of the computer time is spent on these computations. To make the computer program more efficient, it is necessary to find an efficient way to calculate stresses and strains and to replace the layered subroutine. The simplest way would be to solve some factorial problems for various combinations of important parameters ( $E_i$ ,  $D_i$ , and  $W_i$ ) in the layered program, and to get regression equations to determine stresses and strains. It is seen that a direct factorial of the important parameters involved gives an unfeasible number of problems to be solved. Therefore, some simplified procedure needs to be adopted. During a study of the analysis of layered program it was found that stress in the layers is a function of the modular ratios of the layers rather than their absolute values (Ref 131). This is verified and reported by Shahin (Ref 166). Furthermore, Heukelom and Klomp (Ref 60) found from field observation that the modular ratios of untreated pavement layers do not exceed 1.5 to 2.5. For a fixed modular ratio, the number of variables in the layered program is reduced, thereby considerably reducing the number of factorial solutions. This or some other suitable approach should simplify the stress and strain computation in a layered system. Scrivner, at the Texas Transportation Institute, is also working to simplify this problem. Once efficiency in stress and strain calculation is achieved, the problem of computer time in the present procedure is solved.

#### Application of Damage Theory for Overlay Design

In the existing FPS models, it is assumed that after an overlay the resulting total thickness of asphalt concrete behaves as one layer, with the original material properties of all layers. Although in reality some allowance is required to take into account the change in material properties due to traffic loads and time, no such allowance is made in the existing FPS. In the proposed procedure, for layers other than the surface layer, it is planned to account for this change in material properties in the calculations of the rut depth index. The cumulative net rut depth index after an overlay is calculated by subtracting the rut depth index at the time of the overlay from the cumulative

rut depth index at any time after an overlay. After damage to the surface layer, a new layer is added but the old surface layer is cracked. At present there is no rational method available to take this damage into account. An advantage of the proposed procedure, which is based on fatigue and damage hypotheses, is that this damage to the pavement can be taken into account. In a pavement layer, according to Minor's hypothesis, failure is considered to occur when the cumulative damage exceeds 1.0 because initial damage for a new facility is zero. Thus, for a new pavement the cracking index is computed based on the probability of cumulative damage exceeding one. However, when a pavement is overlaid it has already experienced some cumulative damage  $\left( \sum_0^t \sum_0^j \frac{n_i}{N_j} \right)$ . Thus, for an overlaid pavement, an estimate of the mean initial damage  $x$  due to traffic already experienced by the pavement should be made and subtracted from 1.0 (Ref 116). After an overlay, the cracking index should be computed based on the probability that cumulative damage due to new traffic will exceed  $(1.0 - x)$ . This procedure should be adopted in the second generation FPS. An overlay example problem based on this criteria is shown in Fig 13.3. A pavement with layer thicknesses of 3.5, 9, and 8 inches is considered. For a terminal PSI of 2.5, this pavement lasts for 5.5 months. After an overlay of 1-1/2 inches, for the same constant traffic, the pavement lasts for 13 months as compared to 16 months based on the criteria of the existing FPS.

#### EXAMPLE PROBLEM - COMPARISON OF THE PRESENT FPS AND PROPOSED FATIGUE MODELS

To develop confidence in the procedure presented in this report, example problems were run comparing the proposed model with the existing FPS method. Since the comparisons were carried out with AASHO Road Test data, in these example problems high traffic values corresponding to the AASHO Road Test data are adopted. Thus, short time periods for the pavement performance are the result due to high traffic. However, in actual problems, the lower traffic values will result in corresponding increase in the actual performance time periods.

In the first example problem, AASHO Road Test Section 623, which carried 18-kip single axle applications, is designed using the AASHO FPS model. The strength coefficient values assumed are the same as developed in the AASHO Road Test. The input and output are shown in Tables A9.1 and A9.2, respectively.

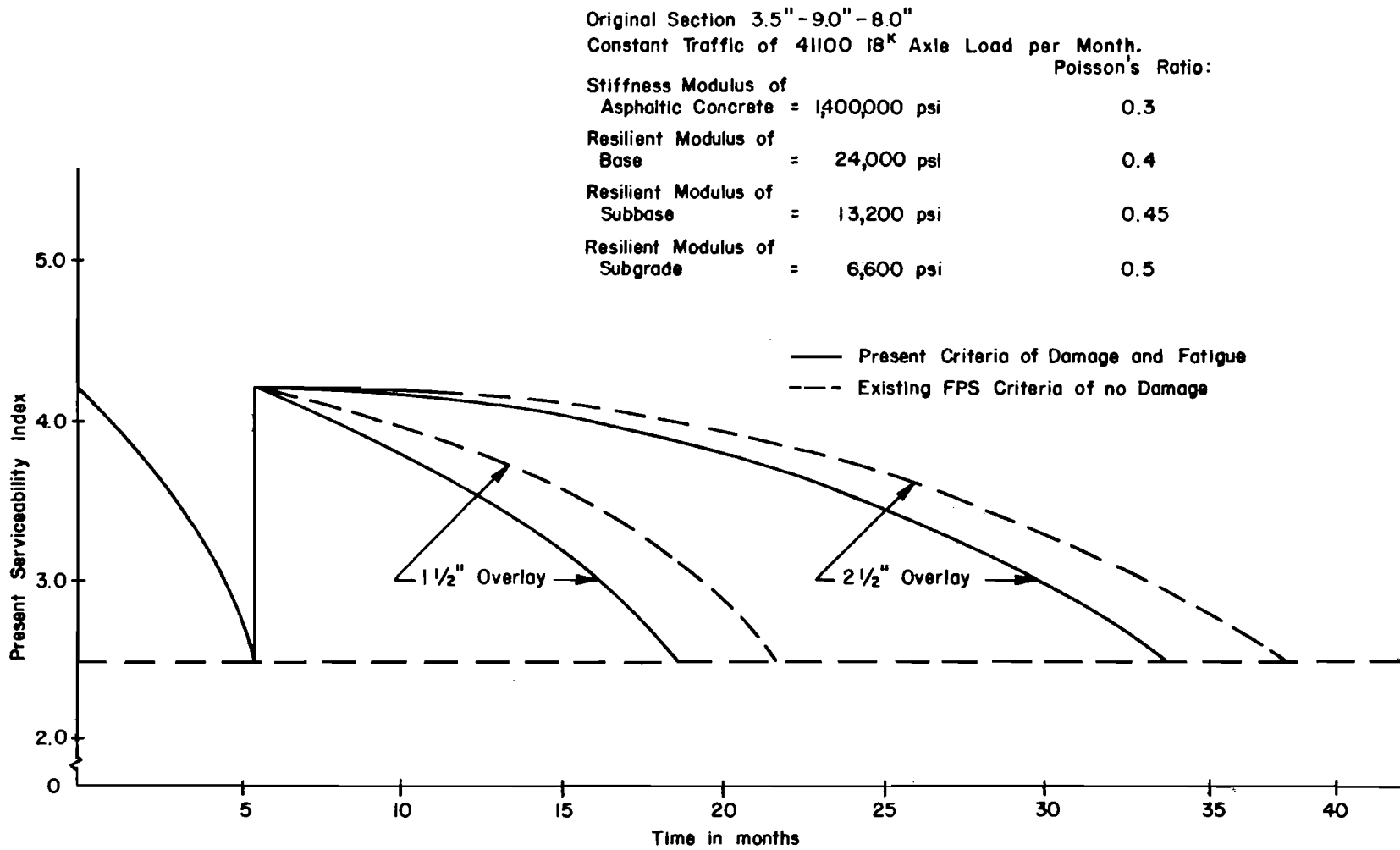


Fig 13.3. Typical overlay design based on fatigue and damage criteria.



To simulate the design for the AASHO Road Test section, no swelling effect was assumed. The initial present serviceability index was assumed as 4.2, which corresponds to the initial observed value of the AASHO Road Test Section 623. The average daily traffic values at the beginning and end including accumulated 18-kip axle applications are based on average values of traffic used on the above AASHO Road Test section. The maximum and minimum layer thicknesses are restricted to 3, 6, and 8 inches for surface, base, and subbase, respectively, and minimum and maximum time for the overlay are restricted to obtain the design life for this thickness combination without an overlay and for an assumed terminal PSI of 1.5.

A few trial solutions established the stiffness coefficient of the FPS deflection model giving the same performance as that predicted by the FPS AASHO model. Figure 13.4 shows performance curves for the pavement section under consideration from various methods, as detailed in the figure. The performance curves are quite close and give confidence in the present procedure.

In the second example, with 18-kip axle load repetitions of the AASHO Road Test data, problems are run with FPS, AASHO and deflection model computer program to obtain a set of structurally equal designs. The input to the program for the FPS AASHO model is shown in Table A9.3 while output is shown in Table A9.4. The input and output for the deflection model are similar. The stiffness coefficients for deflection models are the same as those used in the first example.

The data for swelling clay, traffic, and PSI are the same as taken for the first example. However, the maximum and minimum layer thicknesses are input to get some feasible designs. The minimum and maximum times of overlay are made equal to the performance period to get structural designs to last for approximately the time of the AASHO Road Test period, 2.2 years without an overlay.

These computations have given several designs with different layer thickness combinations, but with the same performance, i.e., the same structural number in the AASHO model and the same surface curvature index (SCI) in the deflection model of FPS. Therefore, as far as FPS is concerned all these combinations have the same structural performance. For comparison the following combinations are considered:

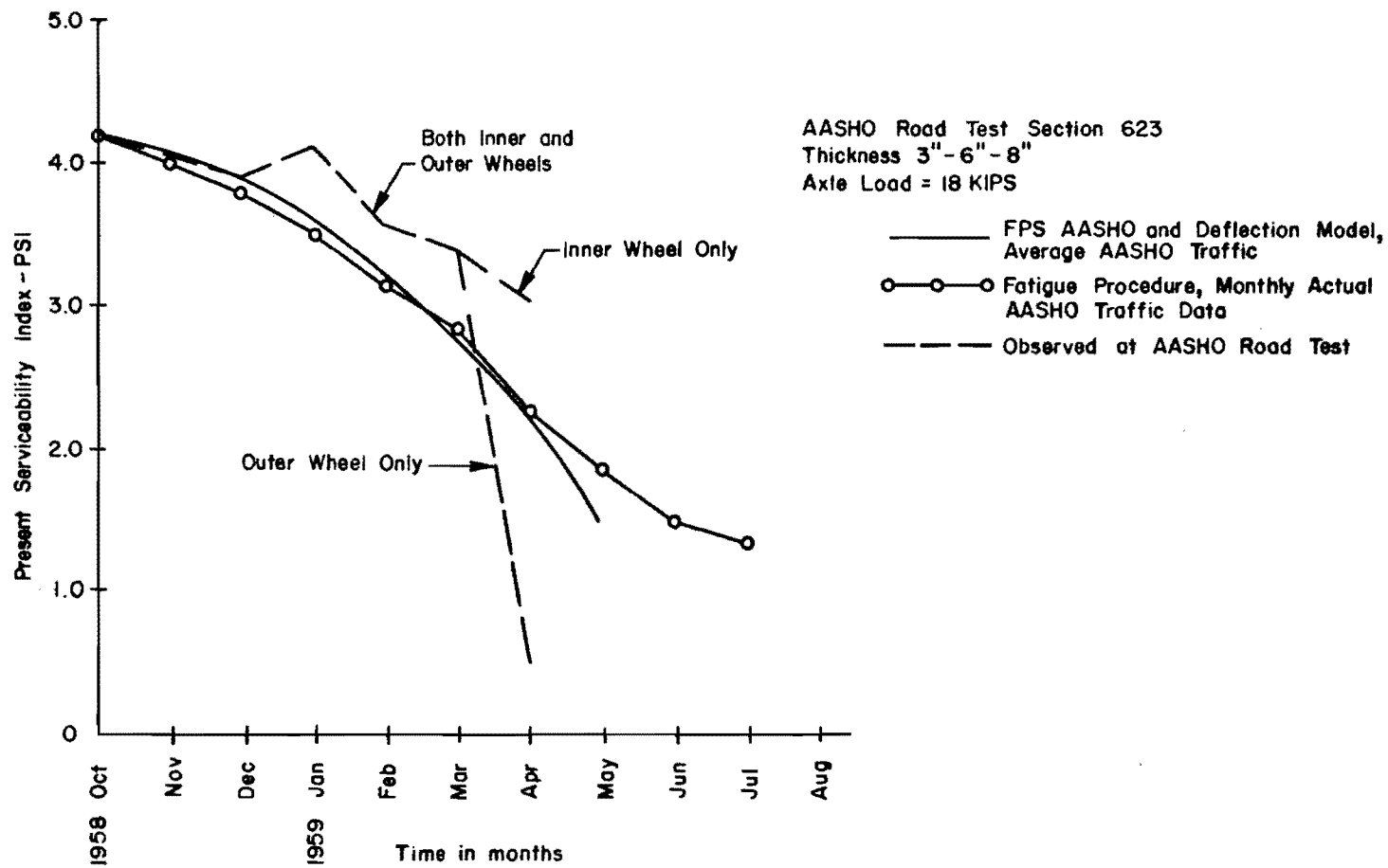


Fig 13.4. Comparison of fatigue method with FPS.

<u>Concrete</u>	<u>Base</u>	<u>Subbase</u>
2.75"	9"	11"
3.5"	9"	8"
4.5"	9"	4"

Figure 13.5 shows that, though the FPS predicts the same performance, the different thickness combinations do influence the performance of a pavement. It is shown that the 3.5, 9, and 8-inch thickness combination has the same performance in the FPS and fatigue procedures, but the performance of the other combinations is significantly different in the fatigue procedure.

The third example (Fig 13.6) shows the effect of support variations. In the FPS, a constant support condition throughout the year is assumed, which is not the real world situation. The performance curves for the FPS deflection model for 50 percent and 80 percent confidence are also shown. The fatigue procedure with some material properties assumed constant throughout the year shows the same performance as FPS. The effect of change in the material properties is shown. The figure also shows the performance when account is taken of monthly variations in the material properties. The figure shows that the assumption of a constant support condition in FPS can give designs which fail much earlier than predicted.

For FPS, the month when the pavement is opened for traffic and monthly traffic distributions are immaterial when no monthly variations in the material properties are taken into account. However, for the proposed fatigue procedure the opening month and monthly traffic distributions and material properties variations are important because the performance and the deterioration in the PSI of the pavement at particular time depend upon the material properties at that time. The effect of monthly traffic distribution and of opening month, for short design period, is shown in Fig 13.7. This is a more realistic approach since, generally, in practice failures have been observed in the spring months.

Figure 13.8 shows the performance curves, for the three combinations of layer thicknesses, as computed by FPS and the proposed fatigue procedure. The proposed procedure is more realistic because, as expected, the performance curves for the three thickness combinations are different in the case of the proposed procedure as compared to the one and same performance curve in case of existing FPS. In this particular example, the design life for different combinations varied by 50 percent. It is also observed that increasing the

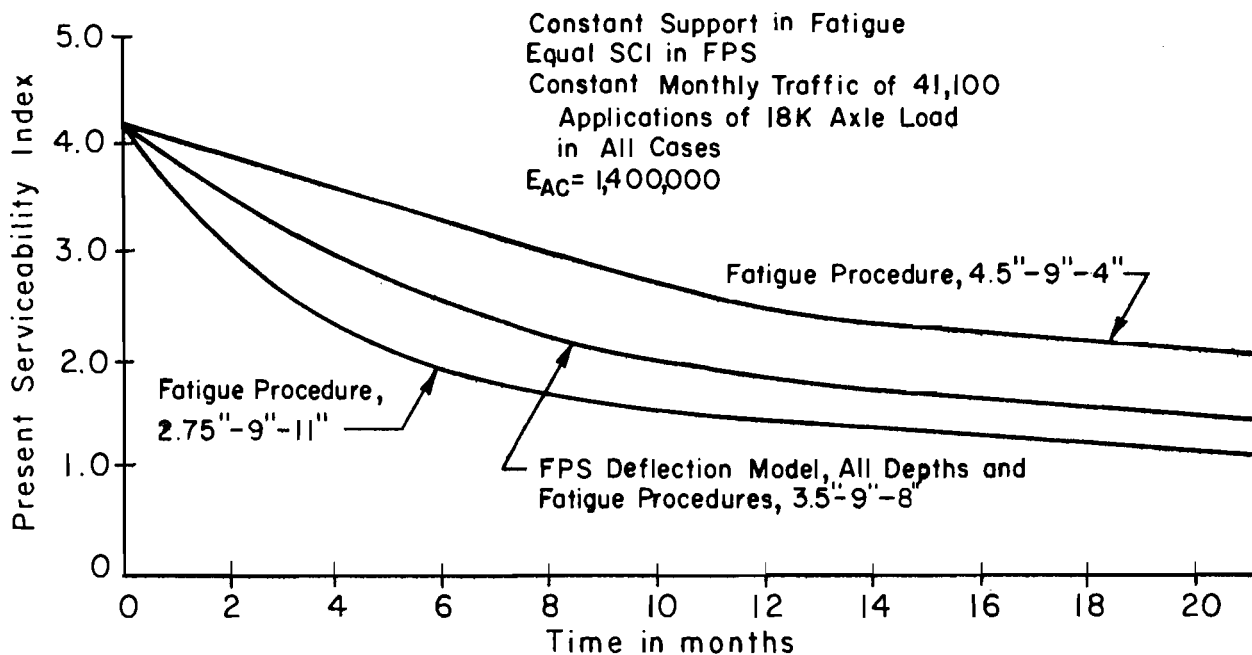


Fig 13.5. Thickness combinations influence performance.

Note: For typical pavement section see Fig 8.4.

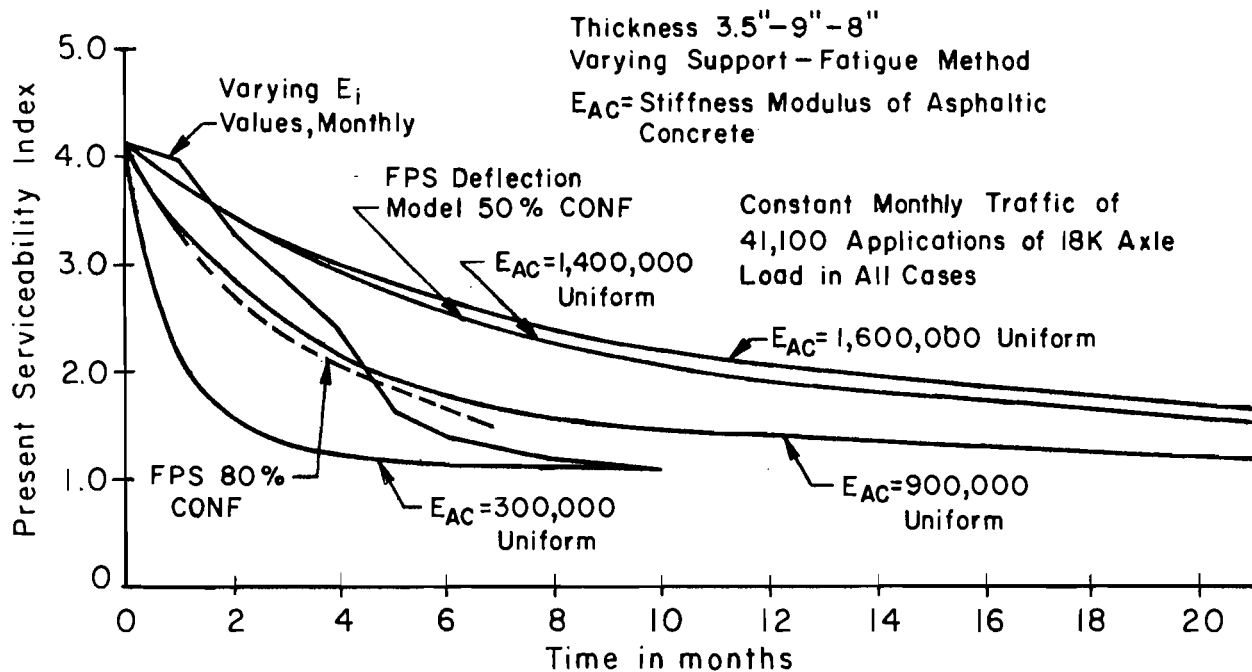


Fig 13.6. Effect of support variation in fatigue procedure.

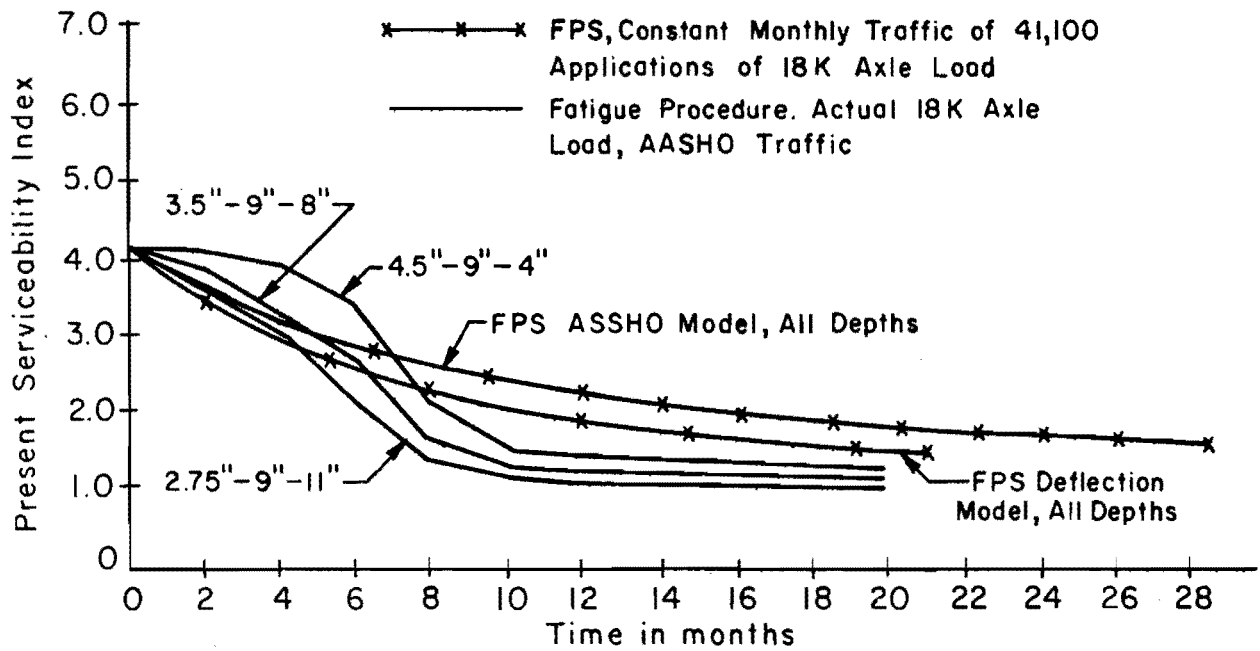


Fig 13.7. Performance curve - fatigue procedure for different starting time.

Note: For typical pavement section see Fig 8.4.

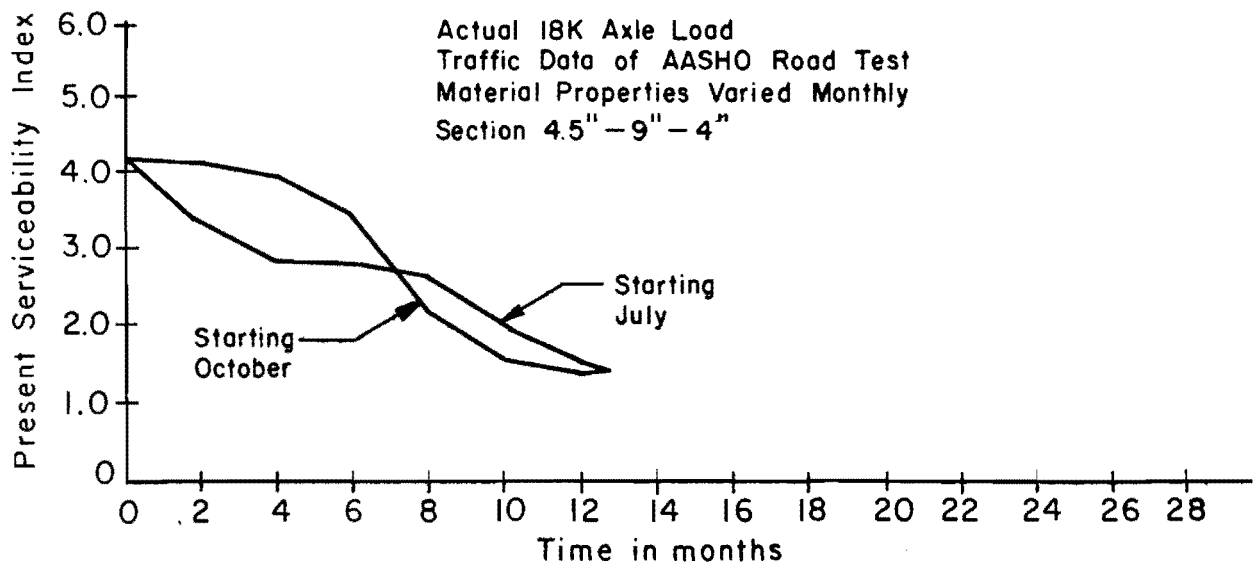


Fig 13.8. Performance curve - FPS vs fatigue procedure.

layer depths from 3, 6, and 8 inches to 3.5 - 9 - 8 inches increased (Figs 13.4 and 13.8) the life of the pavement in FPS from 7 to 21 and 29 months for deflection and AASHO models, respectively, which does not seem reasonable. However, for the proposed fatigue procedure the life increased by 16 percent.

From the results discussed herein the fatigue design procedure seems to give better and more realistic designs of flexible pavements than based on the existing FPS models.

## CHAPTER 14. IMPLEMENTATION

The verification of the models developed in this report and their accurate predictions of the observed data in Chapter 11 along with the results of the sensitivity study in Chapter 12 give confidence in using the proposed procedure for the design of flexible pavements and in including this procedure in the existing FPS. In Chapter 13, revision of the existing FPS was discussed. Including the present design procedure and making the revision to the existing FPS led to the second generation FPS. In its implementation, new inputs are required (see Appendix 4). Implementation of the proposed procedure is discussed in this chapter.

### STRESS AND STRAIN COMPUTATIONS

In the proposed procedure stresses and strains are calculated from the layered program. The inputs for this analysis include the elastic modulus, Poisson's ratio, and stochastic variations in modulus for each material. These properties of the materials are characterized as in Chapter 5. In the previous chapter it was noted that stress and strain calculations in the present analysis, by the direct use of the layered subroutine, should be improved and replaced by a more efficient approach. An alternate approach was discussed and further research to make the present procedure more efficient was recommended. The work on distress due to major temperature cyclic effects is also to be included in FPS second generation and is expected to take additional time at the Center for Highway Research. During this time the research efforts may also be continued to reduce the computation time in the proposed procedure and revision of the present FPS may be made to include the present procedure as well as the major temperature effects. To help with the problem of long computation time only the limited number of designs predicted as the most optimum by the existing FPS should be checked by the present design procedure for correct performance. This will avoid the time used for computations on infeasible and unacceptable designs. Once an array of most suitable designs from the existing FPS is known they are checked by the proposed procedure and

final selection is made. It may be necessary once more to run the cost analysis for these revised designs through the FPS cost models. This tentative design procedure, shown in Fig 14.1, is expected to reduce the computation time considerably.

#### OVERLAY DESIGN

Revision of the existing FPS overlay design procedure is proposed, following the procedure discussed in the previous chapter, which is based on the fatigue theory and cumulative damage hypothesis. The computer program developed for the present design procedure needs a small change for overlay designs. For computation of the cracking index after an overlay, the log 1.0 term in Eq 8.5 needs to be changed to  $\log(1.0 - x)$ , where  $x$  is the cumulative damage  $\left( \sum_0^t \sum_0^j \frac{n_j}{N_j} \right)$  up to the time of overlay. For the rut depth index

the change for an overlay design requires that the rut depth index at the time of an overlay  $RDI_{oL}$  should be subtracted from the rut depth index at any time after an overlay  $RDI_L$  to get the correct net rut depth index after an overlay  $RDI_C$ . The effect of swelling clay is already considered by the existing FPS.

#### REPEATED LOAD-DEFORMATION DATA

The regression models for computing the permanent strain in a pavement (Eqs 5.1, 5.2, and 5.4) are based on the typical characteristics of coarse-grained base and subbase materials and fine-grained subgrade materials of the AASHO Road Test. Characteristics of the coarse-grained materials of the AASHO Road Test are similar to those of materials tested at the Texas Transportation Institute (Ref 35), as mentioned in Chapter 5. These regression models are only used for computations of the rut depth index. Any small variation in the rut depth index is not comparatively important in the performance equation (Chapters 7 and 11) and does not affect the PSI significantly. In addition, any change in the above typical regression models based on actual materials used in a particular pavement probably does not affect the rut depth appreciably. Thus, though these regression models should be revised for accurate computations, based on actual repeated load-deformation characteristics of a



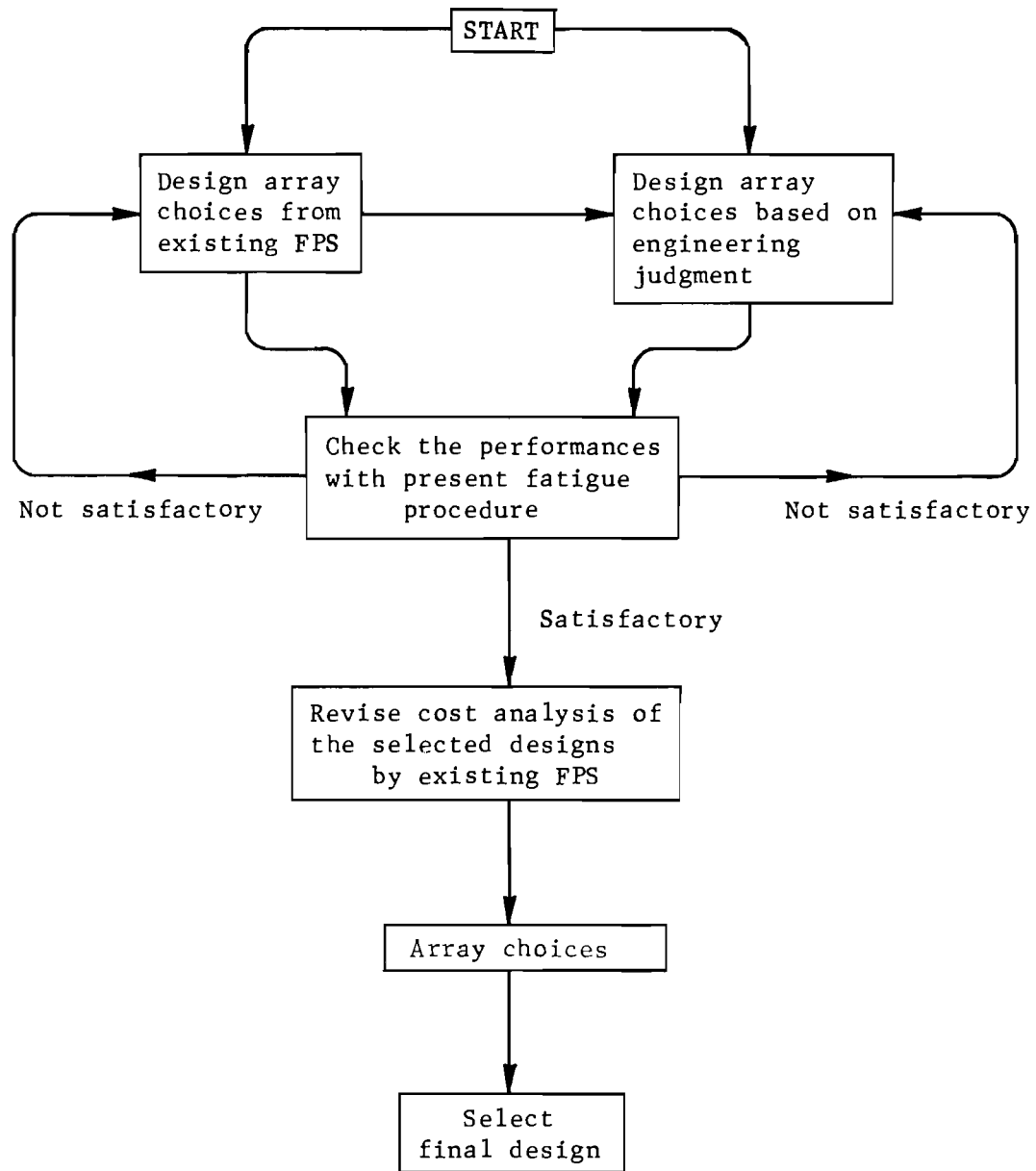


Fig 14.1. Tentative design procedure utilizing existing FPS.

particular material used in a particular pavement as discussed in Chapter 5, even these models are expected to give results which do not affect the final PSI significantly.

#### EQUIVALENCIES

The present procedure has the capability to compute the distress for various single axle load groups of any load intensity, and it is not necessary to change into equivalent 18-kip axle applications as in the existing FPS. However, it is seen that separate computations for various load groups consume much computer time. Thus, to save computation time it is suggested that equivalencies to convert the various load groups into one should be used. The present procedure was developed based on the verification of the AASHO Road Test data. Moreover, in the HRB Asphalt Concrete Structural Design Workshop, 1970 (Ref 63 and Appendix 1), it was suggested that the load equivalency factors developed from the AASHO Road Test equations be used for the present. These equivalencies are based on fatigue and damage criteria. Thus, it is recommended that the equivalencies based on the AASHO Road Test for different load groups and axle combinations be utilized in the design procedure to reduce computation time.

#### PRESENT SERVICEABILITY INDEX

The computation of present serviceability index with the proposed procedure and the modifications needed in the existing FPS performance equation were discussed in Chapter 3. With slight modifications in Eq 3.2, the proposed procedure can be included in the FPS.

#### TRAFFIC COMPUTATIONS

In the existing FPS, Eq 3.3 is utilized for traffic computations. In the proposed procedure the actual load repetitions for each load group or Eq 3.4 is utilized for traffic instead of 18-kip equivalent repetitions. However, existing Eq 3.3 can also be utilized in the second generation FPS with slight modifications in the proposed procedure, by including this equation in addition to Eq 3.4 or replacing Eq 3.4.

#### TIME SUBROUTINE

The time subroutine of the existing FPS can be utilized in a modified form for the convergence process of performance time for the desired PSI, traffic, layer thicknesses, and distresses based on material properties.

#### LABORATORY INVESTIGATIONS

Test procedures required to characterize the material properties which are used in the present design procedure are discussed in Chapters 4 and 5. Laboratory testing programs should be initiated at the earliest opportunity to characterize the fatigue material properties properly and to include the determination of the parameters needed in this development, so that reliable data may be created for implementation of this procedure for use of specific materials in a particular pavement. In the meantime, the design may have to rely on data in the literature or from other sources to obtain values for all variables used in the present method. The fatigue parameters B and A are very sensitive variables, and their values need accurate determination. Thus an immediate testing program to create accurate data is important.

#### LIMITATIONS FOR SURFACE TREATMENT AND THIN ASPHALTIC CONCRETE SURFACES

The proposed models for cracking index and roughness index are derived based on the following hypotheses:

- (1) The cracking index in a pavement system is caused by the repeated flexural tensile strain developed in the asphaltic concrete surface layer due to applied wheel loads.
- (2) The constant stress mode of loading conditions is most logical to determine the fatigue response of asphalt concrete for flexible pavement design.
- (3) The roughness index represents most of the detrimental effects of cracking and that cracking is a good indicator of roughness in a pavement.

Under the above logical hypotheses, the proposed cracking index and roughness index models are not applicable in case of the surface treatments. For thin asphalt concrete surfaces less than 1 inch thick the models are not expected to give satisfactory results, and therefore the models may not be directly used for these cases. Further research work is needed to modify the models for their use for thin surfaces.

#### PREDICTION ERRORS IN THE MODELS

The prediction errors in the proposed models were discussed in Chapter 11 and results of a short sensitivity analysis were discussed in Chapter 12 to create a confidence in use of the proposed models. However, for the variations in the performance predictions at various confidence levels under different combinations of input variables and for determining the relative significance of these variables a detailed sensitivity analysis shall have to be performed.

PART V

CLOSURE

## CHAPTER 15. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

In Chapter 1, the general study objective of development of new design models for flexible pavement system second generation, based on best available technology, to update the existing FPS model, was mentioned. The author feels, in his opinion, that this objective has been achieved successfully herein by development of the various distress models and quantifying the present serviceability index value from these models. The inclusion of this procedure in FPS requires revisions in only the structural design portion and corresponding material characterization of the systems, while the user's cost and other economic models will continue to remain the same.

### SUMMARY

In the first few chapters, the existing design methods and theories were discussed and need for a new systems design procedure, considering the fatigue theory, linear elastic layered theory, and probability theory, was established. The concept of distress and failure in the pavement, along with the AASHO concept of performance and present serviceability index, was discussed. Based on these concepts, the distress models for cracking index, roughness index, rut depth index, and overall present serviceability index were developed in terms of elastic and fatigue material properties (which can be predicted in the laboratory) and their stochastic variations with space and time, loading, environmental conditions, and load-deformation characteristics. These distress models are proposed to replace the empirical relationship used at present to simulate the transformation between the input variables and performance of a pavement.

Similarity and accurate predictions of the distresses actually observed for the AASHO Road Test sections and sensitivity analyses performed for the cracking index models give the confidence in the use of the models and procedure developed in this report. The development and the use of the computer program makes it easy to handle the calculations involved in the systems.

The proposed models can be used directly and can also be included in the existing FPS models leading to the flexible pavement system second generation. To reduce the computer time, the proposed models may be used to provide an independent check on current design procedures. The present method of overlay design is proposed to be replaced by the method which is based on fatigue theory and damage criteria. The example problems are solved which show the improvements of the proposed method and add to the confidence in use of the proposed models for the design of flexible pavements.

The concept of fatigue and probability theory in pavement design presented herein, though recent, is well recognized now. Their proper use in the design of flexible pavements for the first time, in the present form, adds a new dimension to the pavement design field.

The need of (1) proper relationship between distress mechanism, performance, and serviceability; (2) considerations of stochastics in pavement design; (3) distress due to fatigue in the pavement; and (4) applicability of linear theories to predict stress and strains in the pavement was recognized in the first few priority items for research by the HRB Workshop in January 1970, held at Austin, Texas (Appendix 1). The author feels that this report is a first successful attempt in this direction.

In summary, a comparison of various field observations with the predicted distress values gives a large degree of initial confidence in the design models and, in the author's opinion, the method is ready for immediate practical application, although it is only long-term observation and feedback process that will truly verify the models.

## CONCLUSIONS

The flexible pavement design models presented in this report are based on sound fundamentals using the best state-of-the-art information available. The author feels that the specific objective of this study, detailed in Chapter 1, has been well accomplished.

Following are the specific conclusions for this study.

- (1) A study of the development of the design methods of flexible pavement shows the need of a rational method of design which can predict the performance of a pavement under various sets of conditions to update the existing FPS.

- (2) The review of current procedures and methods of flexible pavement design reveals that only a few bona-fide procedures exist and those in practice now certainly need improvements in various ways. The proposed design method will go a long way to fulfill these needs.
- (3) The characterization of materials is a very important part of the whole design procedure. Proper laboratory techniques need to be extended for material testing. In the meantime, the engineer may have to rely on data in the literature or from other sources to obtain values for all variables used in the present method.
- (4) The use of linear layered elastic theory may be considered as the most appropriate method for the calculations of stress and strains in various layers, although some more efficient approach to make these computations is needed.
- (5) The development of the proposed method in the present form gives a new added dimension in the pavement design field and gives the realization of the importance of stochastic principles over the deterministic models.
- (6) The proposed distress models have been verified successfully with the AASHO Road Test data which gives the confidence in the use of these methods. Example problems show the improvements of the proposed method over the existing FPS.
- (7) This method shows some combinations to be unacceptable, which are acceptable with the present design.
- (8) The new design models are more realistic and are based on sound and latest state-of-the-art. The models can be easily included in the existing flexible pavement systems model without many changes except in the structural design portion of the systems program. The proposed method can also be used directly for design of flexible pavements.
- (9) Since the conventional hand solutions are a physical impossibility in solving the problems with these new models, the necessary computer programs have been developed to aid the design process.
- (10) The developed design models are considered to be ready for an immediate application in the field.
- (11) Deteriorated condition of the pavement should be adequately considered at the time of the overlay construction, based on fatigue principal. The proposed method of overlay design, based on fatigue theory, damage hypotheses, and stochastic concepts, presented in this report, adds a new dimension to the overlay design.

## RECOMMENDATIONS

The author recommends that

- (1) The models developed in this report may be put to immediate application for the design of flexible pavements and procedure included in the existing systems model to create FPS second generation.



- (2) Research efforts should be continued to include the fatigue effects of the nontraffic-associated temperature cycles and foundation movements.
- (3) The existing FPS model may be revised to include the developed models for traffic-associated distress and for nontraffic-associated distress when such models are ready.
- (4) Laboratory testing programs should be initiated to characterize the material properties properly and to include the determinations of the parameters needed in this development.
- (5) Necessary feedback data banks should be created and kept up-to-date to update the method as and when required.
- (6) It is also important that necessary efforts are continued to make the developed computer program more efficient.
- (7) A sensitivity study of the parameters involved should be made and their significance in the program should be evaluated.
- (8) Consideration of the variability and probability may be extended for other parameters in the systems design not considered in these developments.
- (9) Maintenance and other models in the FPS may be updated.
- (10) The swelling clay effect needs to be considered in some more rational way in the systems design model.
- (11) Cracking index distress model, presented herein, has been based on the constant stress conditions in the pavement. This is a conservative situation in some cases in comparison to the constant strain conditions, especially for thin pavements. Further research efforts are needed as to the application of these two cases in different situations.
- (12) The proposed cracking index and roughness index models are not applicable in case of the surface treatments. For thin asphalt concrete surface of less than 1-inch thickness, the models are not expected to give satisfactory results. Further research efforts may be extended to modify the models for their use in these cases.
- (13) The principles of design and development of models discussed in this report should be extended for the existing rigid pavement system.
- (14) Efforts should be continued to make the present method of layered computer program to calculate stress and strain more efficient and/or replaced by a more efficient approach.
- (15) The proposed models could be used to give better evaluation of some of current blackbases being proposed for pavements by the Texas Highway Department.

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APPENDIX 1

SUMMARY OF RESEARCH NEEDS, ADVISORY COMMITTEE, HRB WORKSHOP  
ON STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENT SYSTEMS  
HELD IN AUSTIN, TEXAS, DECEMBER 7-10, 1970 (REF 63)

APPENDIX 1. SUMMARY OF RESEARCH NEEDS, ADVISORY COMMITTEE, HRB WORKSHOP  
ON STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENT SYSTEMS  
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To make engineers more effective in bringing developments to the profession quickly and in helping to direct research efforts by improving the interaction between engineers and researchers, a workshop was held at The University of Texas during the period December 7-10, 1970, under the auspices of the Highway Research Board and sponsored by the Federal Highway Administration.

The following list represents ten major research items required to develop additional methodology for problems currently not solvable. These items have been obtained from the deliberations of the nine discussion groups and have been ranked by the Advisory Committee (Ref 63). This committee has prepared statements which reflect the extent of these research areas.

It should be noted that there are many items included in the discussion group reports which have not been included in this listing.

1. Relationship Between Pavement Distress and a Performance or Failure Function

There is no mechanistic way to relate pavement distress to pavement failure except for specific conditions (e.g., excess of rutting levels related to safety).

2. Determine Applicability of Linear Theories to Predict Stress, Strain, Deflections and Fatigue Distress in Pavements

This research is intended to determine how accurately the linear theories of elasticity and viscoelasticity (applied to layered systems) can predict the stress and strain states, and surface deflections. The predicted stress and/or strain state in conjunction with fatigue data is to be used to estimate the cracking of pavements subjected to repeated loads. In addition the viscoelasticity theory is to be used to predict surface rutting. In all cases the predictions will be compared with closely controlled and thoroughly instrumented laboratory and field experiments.

### 3. Mechanical Characterization of Granular Materials

Although unbonded granular materials have been used as components of pavements for many years, there are as yet no generally accepted constitutive equations by which they may be represented in the stress analysis or which will reflect their cumulative deformation under repeated loadings. Sensitivity to confining pressure, the modification of response due to various degrees of saturation, the "conditioning" which occurs under early applications of load, and the cumulative densification or distortion which is produced by many cycles of load well below failure levels, must be considered. Relationships which approximate these effects under the three dimensional states of stress typical of those occurring in pavement systems are required. Since rigorous representation is not immediately attainable, the emphasis should be placed on the permissible deviations from linear viscoelastic systems which are tractable in analysis. After acceptable parameters are selected to characterize such materials, test procedures must be developed for use by engineers on a production basis which permit measurement of these parameters on granular materials in a state representative of their in-situ condition in the pavement system.

### 4. Effect of Environment on Pavement System Condition and Response

To provide the ability to predict the equilibrium conditions which will prevail in a given pavement system under local moisture and temperature environments and the effects of these conditions on materials' properties, differential surface deformations, and pavement performance.

### 5. Treating Pavement Design as a Stochastic Process

A procedure needs to be developed which will predict variations in the system response due to statistical variations in load, environment, geometry, and material properties. In addition, an error analysis is required to estimate the variations in the predictions arising from inaccuracy of the analytical model and inconsistency in testing procedures.

This would provide the designer with the ability to evaluate the risk involved in arriving at a particular design value by the selection of various assumed values for parameters based on statistical considerations.

### 6. Fracture Mechanisms

The mechanistic approach to fatigue-crack prediction utilizes fracture-mechanics principles to explain the initiation, propagation, and accumulation

of cracks. It offers many potential advantages as compared with the phenomenological approach primarily in terms of its ability to handle both mode of loading and areal cracking as well as in terms of its ability to explicitly treat the stochastic nature of the process. Following the successful completion of current research programs, additional research that is anticipated includes the effects of random loading, the phenomenon of localized plastic flow due to occasional heavy loadings, and continuing field verification.

#### 7. Mechanical Characterization of Pavement Materials (Other than Granular)

While considerable progress has been made in the identification and measurement of properties of asphaltic concrete required for insertion into the linear viscoelastic and other procedures of stress analysis, there still remain important questions in the characterization of these materials and of asphalt-treated base materials, cement-treated base materials, and cohesive soils. In all cases, the degree of departure of these materials from the linear response model must be determined to identify any deviations large enough to require special analysis. Further, the deformation and fracture response of these materials to repeated loading under states of stress representative of critical states in pavement systems must be determined. The effects of the environmental variables of temperature and moisture, where appropriate, must be evaluated. After appropriate characterizations are obtained, production type tests capable of use by highway engineers must be developed, and typical ranges of values determined.

#### 8. Identification of Loading

(a) Determine accuracy of weight and volume data presently being obtained and reported in the W-4 loadometer tables by extending the studies.

(b) Gather data to check accuracy of past predictions of design loadings.

#### 9. Reflection Cracking - Method of Prediction

Current systems of overlay design do not provide adequate guidance in designing overlays to prevent reflection cracking. This is particularly true in the case of large random or thermal cracks found in older Portland Cement Concrete or cement-treated base structures. In addition, current design methods do not recognize that cracking can initiate in the base course due to

shrinkage or other environmental changes. Such a crack can then reflect through the surface layer leading to distress.

It is believed that the possibility of developing a mechanistic model should be explored with the purpose of providing a rational approach to these design problems.

With portions of the Interstate as well as other Federal and State Highways approaching the end of their structural design life, it is important that work on the problem be started at an early date so that it will be available to help in the designs which will be facing the states in the next few years.

#### 10. Information Data Base for the Pavement System

Development of rational pavement design methods is an iterative process which involves observation and subsequent improvement based on analysis of observed data. Validation or modification of system and sub-system models lends emphasis to the need for selecting of proper variables and compatible ways of measurement. The numbers of the possible candidates for inclusion in the system requires that effective information management techniques be applied to the data handling process. This involves; selection of parameters to be stored, sampling plan (i.e., how, when, where to take data), data processing, input, storage, and output techniques. The pilot model of such a system probably involves selected pavement sections rather than an entire pavement inventory.

Service-Performance Measurements, or subjective ratings, must be used to bridge this gap and thus to establish a way of defining pavement failure and unserviceability for combined levels of distress in terms of the pavement function and the user.

Studies must define important distress factors involved in pavement failure (including weighting functions for these factors) in terms of time, traffic, or other usable factors. Concepts of "value", such as Utility Theory, should be studied to see if such work can be applied to this function.

A crude estimate of man-years necessary to accomplish bringing research needs to the true implementation state was compiled by the Advisory Committee. There was not time to make any considered judgments, therefore these should only be viewed as an expression of magnitude.

1. 10-50
2. 10
3. 12
4. 10
5. 5
6. 10
7. 12
8. 2
9. 5
10. 5-50

APPENDIX 2

DETAILS OF MATERIAL CHARACTERIZATION

## APPENDIX 2. DETAILS OF MATERIAL CHARACTERIZATION

### Introduction

The need for and technical aspect of the material characterization along with the details of the material properties required for the present flexible pavement design procedure were discussed in Chapter 5. Consolidated statements on the properties of the AASHO Road Test materials which were adopted for the present analysis were also included in Chapter 5. Details of the procedure to characterize these material properties which were not included in Chapter 5 are given in this appendix.

### Elastic Modulus

The elastic modulus is one of the most important material properties to be considered in the proposed fatigue model. Its determination should be based on a close simulation of expected field conditions. The computations required for such a determination are given in the following paragraphs.

Asphalt Concrete. Various parameters and mix properties required for determination of the stiffness modulus of asphalt concrete are listed in Chapter 5. The procedure to determine these parameters is detailed below.

Monthly Temperature. For the present design procedure, to determine the monthly values of stiffness modulus of asphalt concrete, a temperature representative of each month is required. This can be obtained from weather data. For the present analysis, this information (Table A2.1) was obtained from the AASHO Road Test Report 5 (Ref 70).

Time of Loading. Several axle loads were used in the AASHO Road Test. The axle load as well as the tire pressures in each case was different. However, a constant speed of 35 miles an hour was maintained for the test traffic (Ref 70). Time of loading is required to calculate suitable values of the stiffness modulus for asphalt concrete. For this report, an average time of loading was calculated as shown in Table A2.2, which is self-explanatory. On this basis, a mean value of 0.02 seconds was adopted for all calculations.



TABLE A2.1. MONTHLY TEMPERATURES

Month	Temperature, ° <u>F</u>
January	21
February	25
March	27
April	41
May	54
June	66
July	70
August	75
September	65
October	51
November	43
December	28

Mix Properties. The following average values were adopted from the AASHO Road Test Report 2, SR61B (Ref 67):

Ring and ball temperature	117 <sup>o</sup> F
Penetration at 77 <sup>o</sup> F	91
Voids	3-5%
Asphalt content	5.4% based on total weight of mixture
Density of surface course mix	146.8 lbs per cu ft

From the above data,

$$\text{Volume concentration } C_v = \frac{\text{Volume of compacted aggregate}}{\text{Volume of aggregate + asphalt}}$$

$$\text{Volume of asphalt per cu ft} = \frac{146.8 \times 5.4}{100 \times 62.4} = 0.127 \text{ cu ft}$$

(assuming specific gravity of asphalt as unity).

$$C_v = \frac{1}{1 + 0.127} = 0.89$$

Making Van Draat and Sommer corrections for voids of more than 3 percent (Ref 38),

$$\text{Corrected volume concentration } C'_v = \frac{C_v}{1 + \Delta H}$$

$\Delta H$  = difference between actual air void content and the value of 3 percent (expressed in decimal form)

$$C'_v = \frac{C_v}{1 + \Delta H} = \frac{0.89}{1 + \frac{5 - 3}{100}} = 0.87$$

$$\frac{C'_v}{1 - C'_v} = 6.7$$

From PI charts (Ref 121), assume PI = 0 .

Time of loading from Table A2.2 = 0.02 seconds.

Values of Stiffness Modulus. Values of the stiffness modulus of asphaltic concrete were calculated by the Heukelom and Klomp (Table A2.3), and the Nijboer method (Table A2.4).

Table A2.5 gives a consolidated statement of the stiffness values calculated by the above two methods at various temperatures. Columns 5 to 9 in this table give the stiffness values of asphalt concrete adopted in various references. Column 10 gives the practical observation values of stiffness in a pavement at various temperatures from a plot reported in Highway Research Record No. 71, pp 70-73. This plot was developed by use of the results of subgrade stress measurements at different pavement temperature. It may be observed that the values of stiffness calculated by Nijboer are very low as compared to the Heukelom and Klomp method. The literature review shows that the Heukelom and Klomp method has been given enough recognition to make the results based on this method more reliable, although it has been observed (Ref 42) that this method tends to give higher values. The Nijboer method has not been used much. To get a reasonable value of stiffness consistent with the values in Column 10, the following criteria were adopted for the present analysis.

- (1) A weighted average stiffness value was calculated at all temperatures by weighting the Heukelom and Klomp method, twice as compared to the Nijboer method.
- (2) After the stiffness values were recalculated by the method in the preceding paragraph, it was found that values agreed reasonably well with Column 10 except at high temperatures. At a temperature of 77° F the values given by the Nijboer method as well as by the Heukelom and Klomp method are lower than the values of Columns 5 to 10 obtained at this temperature from the indicated references. Therefore, to get a consistent value, an average of all the values in Cols 5 to 10 was taken and this value was assumed reasonable.

Based on paragraphs (1) and (2) above, a plot was made as shown in Fig A2.1 to represent the stiffness values of asphalt concrete of the AASHO Road Test. The monthly temperature values (Table A2.1) for the present analysis are taken from AASHO Road Test Report 5 (Ref 70). The monthly temperature and stiffness modulus thus obtained are tabulated in Table A2.6.

Untreated Granular Base and Subbase Materials. Monthly values of the resilient modulus for these AASHO Road Test materials are not available. However, proper analysis of the existing data could give the desired information.

TABLE A2.2. TIME OF LOADING

Wheel Load, lb	Tire Pressure, lb/in <sup>2</sup>	Contact Area, in <sup>2</sup>	Diameter of Contact Area, in.	Time of Loading, seconds
1,000	29.1	34.4	6.6	.0107
3,000	42.3	71.0	9.5	.0162
6,000	65.7	91.5	10.8	.0175
6,000	65.7	91.5	10.8	.0175
9,000	67.5	133.3	13.0	.0210
8,000	69.5	115.0	12.1	.0196
11,200	66.4	169.9	14.7	.0240
10,000	66.4	151.0	13.9	.0225
15,000	69.7	216.0	16.6	.0270
12,000	69.8	172.0	14.8	.0240
MEAN				0.02

Average Adopted Time of Loading = 0.02 seconds

Speed of Vehicles at AASHO Road Test = 35 mph  
= 51.3 fps

TABLE A2.3. CALCULATION OF MONTHLY STIFFNESS MODULUS OF AASHO ASPHALT CONCRETE BY HEUKELUM & KLUMP METHOD REF NCHRP REPORT 39 (Ref 42)

Temp °F	Temp Below R&B		Stiffness of Bitumen (S <sub>b</sub> ) kg/cm <sup>2</sup>	n= 0.83 × log $\frac{4 \times 10^5}{S_b}$	$\frac{S_m}{S_b} = \left(1 + \frac{2.5}{n} \cdot \frac{cv'}{1-cv'}\right)^n = X^n$		Stiffness of mix (S <sub>m</sub> ) kg/cm <sup>2</sup>	S <sub>m</sub> psi × 10 <sup>5</sup>
	° F	° C			X	X <sup>n</sup>		
1	2	3	4	5	6	7	8	9
20	97	54	2000	1.91	9.75	77.5	155000	22.1
23	94	52	1600	1.95	9.6	82.3	131680	18.8
29	88	49	1200	2.08	9.0	96.7	116040	16.6
41	76	42	500	2.41	7.9	146.0	73000	10.5
52	65	36	300	2.59	7.5	184.0	55200	8.0
64	53	29	100	2.95	6.6	285.0	28500	4.0
67	50	28	90	3.03	6.5	290.0	26100	3.7
71	46	26	60	3.18	6.3	348.0	20880	3.0
74	43	24	50	3.24	6.2	370.0	18500	2.6
75	42	23	40	3.32	6.0	383.0	15320	2.2

TABLE A2.4. CALCULATION OF MONTHLY STIFFNESS MODULUS VALUES OF AASHO ASPHALT CONCRETE BY THE NIJBOER METHOD (Ref 173)

$$E_{60} = 16 \frac{p}{f}$$

where

$E_{60}$  = modulus of deformation ( $\text{kg}/\text{cu}^2$ ) at  $60^\circ \text{C}$  and time of loading of 5 sec,

$p$  = Marshall stability (kg) of mix,

$f$  = flow value (unit 1/100 cm).

For AASHO asphalt concrete  $p = 900 \text{ kg}$

$f = 28$

$$E_{60} = 16 \frac{p}{f} = 16 \times \frac{900}{28} = 514 \text{ kg}/\text{cu}^2$$

Coefficient for 0.02 time of loading = 4.0

Hence  $E_{60}^t = 0.02 = 514 \times 4.0 = 2056 \text{ kg}/\text{cu}^2 = 29362 \text{ lbs}/\text{sq in.}$

Temp. $^{\circ}\text{F}$	Temp. $^{\circ}\text{C}$	Coefficient	E lbs/sq in. $\times 10^5$
1	2	3	4
20	-6.7	19	5.6
23	-5.0	18	5.3
29	-1.7	16	4.7
41	5.0	12.5	3.7
52	11.1	8	2.4
64	17.7	6.5	1.9
67	19.4	6.0	1.8
71	21.7	5.8	1.7
74	23.3	5.0	1.6
75	23.9	5.0	1.5

TABLE A2.5. STIFFNESS MODULUS OF ASPHALTIC CONCRETE OF AASHO ROAD TEST BY VARIOUS METHODS

Temp. ° F	Values of Stiffness Modulus by Various Methods psi × 10 <sup>5</sup>								
	Heukelom & Klomp Method	Nijoboer Method	Adopted Values*	Coffman et al AAPT 1964 pp 87-89 Dynamic Test	NCHRP 1-10	Finn in Ref (169) pp 418			HRR 71 pp 70
						Based on Vanderpoel Method	Baker & Papazian	Adopted by Finn	
1	2	3	4	5	6	7	8	9	10
20°	22.1	5.6	16.6						15.0
23°	18.8	5.3	14.3						14.0
29°	16.6	4.7	12.6						13.0
41°	10.5	3.7	8.2	17.0 at 40° F					10.0
52°	8.0	2.4	6.1						8.0
64°	4.0	1.9	3.3						6.0
67°	3.7	1.8	3.1						6.4
71°	3.0	1.7	2.6						5.0
74°	2.6	1.6	2.3						4.5
75°	2.2	1.5	2.0	6.0 at 77°F	1.5	3.4 at 77°F	4.6 at 77°F	1.5	4.3

\*Adopted Values Calculated by  $\frac{\text{Col (2)} \times 2 + \text{Col (3)}}{3}$

TABLE A2.6. MONTHLY TEMPERATURES AND VALUES OF STIFFNESS

Month	Temp. ° F	Stiffness Modulus psi × 10 <sup>5</sup>
January	21°	16.0
February	25°	14.2
March	27°	13.7
April	41°	9.0
May	54°	6.0
June	66°	4.0
July	70°	3.5
August	75°	3.0
September	65°	4.2
October	51°	6.5
November	43°	8.3
December	28°	13.1



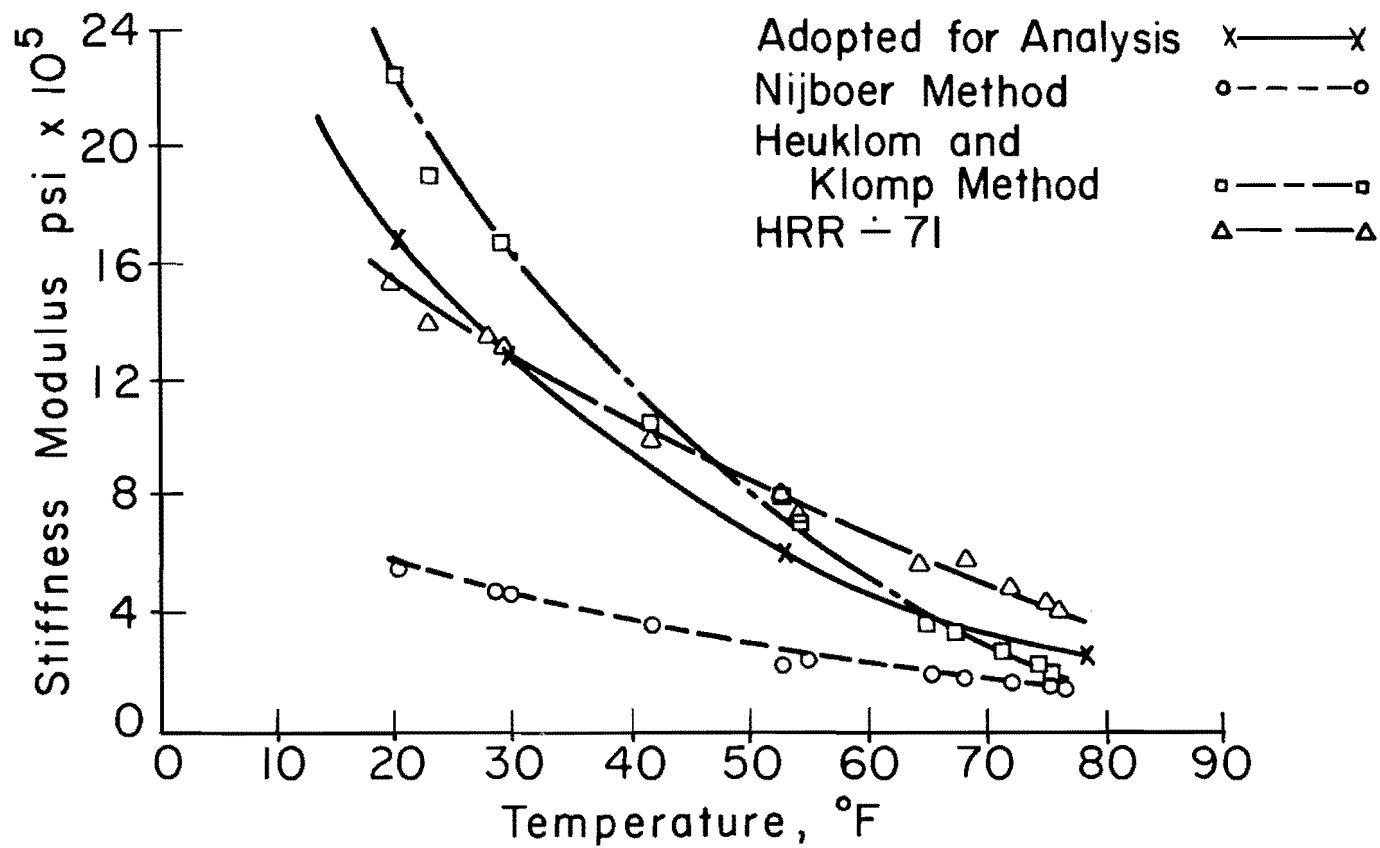


Fig A2.1. Plot of temperature versus stiffness modulus.

Figures 96 and 97 of AASHO Road Test Report 5 (Ref 70) show the moisture content and CBR values of the pavement components during various months. These values are entered in Columns 1 to 3 of Tables A2.7 and A2.8. NCHRP Report 1-11 (Ref 117) gives an approximate correlation between CBR and resilient modulus for these materials. Estimated values of resilient modulus based on this criterion are entered in Column 4. Special Report 66 (Ref 66) gives the test results of the AASHO Road Test materials as reported by various agencies. An average resilient modulus value of 15,000 psi for base and 8,000 psi for subbase are reported in this reference. Based on the AASHO Road Test results, a correlation between the AASHO Road Test strength coefficients and resilient modulus was developed (Ref 94). This criterion gives an expected variation in resilient modulus from 48,000 to 9,000 psi for base and 9,300 to 6,200 psi for subbase materials used in the AASHO Road Test. Heukelom and Klomp (Ref 60) observed that the modular ratio of the untreated material layers in the pavement in a stable condition are not expected to be more than 1.5 to 2.5. The expected values of resilient modulus for base and subbase based on this observation and assuming an average modular ratio of 2 are entered in Column 8. Special Report 66 (Ref 66) gives some CBR test values corresponding to the observed moisture contents. Corresponding values of resilient modulus estimated from NCHRP Report 1-11 (Ref 117) are entered in Column 11. Haynes and Yoder (Ref 57) have reported a range of modulus for base material as 33,500 psi to 39,500 psi. The plots of moisture content versus resilient modulus are shown in Figs A2.2 and A2.3.

From the above discussion, it is seen that it is difficult to pinpoint an absolute value of the resilient modulus for these materials. However, the modulus values for subbase were adopted on the basis of Heukelom and Klomp (Ref 60) criteria given in Column 8. These values are also about an average of the values given by other criteria. The modulus values of base were also based on Heukelom and Klomp criteria except that some adjustments, as shown in Column 12, were made because of very little change in the base CBR values during the months from September to March.

Fine Grained Subgrade Materials. No direct information regarding the monthly resilient modulus of the AASHO Road Test subgrade material is available from the test results. However, an indirect estimation of monthly resilient modulus is possible by use of available information (consolidated in Table A2.9) of the test results of this material.

TABLE A2.7. MODULUS OF RESILIENCE FOR AASHO ROAD TEST BASE MATERIAL

Month	AASHO Road Test Report 5 (Ref 70)		NCHRP 1-11 (Ref 117) E lbs/sq in.	Average Value of E reported in Ref 66	Jain Model (Ref 94)**		Heuklom & Klomp (Ref 60) $E_{base} = 2 \times E_{subbase}$ (1.5 to 2.5)	Page 89 of Ref 66** Test Results of AASHO Base Material by Various Agencies			Values Adopted E lbs/sq in.
	CBR	M.C. Corresponding to CBR Test			Strength Coefficients A <sub>2</sub>	E lbs/sq in.		Moisture Content	CBR	NCHRP 1-11	
1	2	3	4	5	6	7	8	9	10	11	12
Jan.	36*	4.2*	21,000	15,000**	0.25	48,000	26,400	1. 7.3	72	28,000	24,000
Feb.	36*	4.2*	21,000				26,400	2. 6.1	170	-	24,000
March	36*	4.2*	21,000				26,400				24,000
April	19	6.7	14,000		0.11	9,300	14,400	3. 6.8	120	-	15,600
May	22	6.0	16,000				17,200	4. 6.6	80	28,500	18,000
June	25	5.3	17,500				19,600	5. 6.8	87	29,000	19,600
July	28	4.8	18,500				21,600	6. 10.0	92	30,000	21,600
Aug.	32	4.6	19,000				23,200	7. 7.0	34	19,500	23,200
Sep.	34	4.5	19,500				24,400	8. 6.3	93	30,000	24,000
Oct.	35	4.4	20,000				24,800	9. 12.5	3	-	24,000
Nov.	36	4.3	20,500				25,600				24,000
Dec.	36	4.3	21,000				26,400				24,000

\* Assumed Values

\*\* Values do not correspond to any particular month

According to the material specifications (AASHO Road Test Report 2 pp 64) the CBR for the base material was specified as 75, which according to NCHRP 1-11 will correspond to an E value of 28,000 psi.

Haynes and Yoder (Ref 57) reported values ranging from 33,500 to 39,500 psi.

TABLE A2.8. MODULUS OF RESILIENCE FOR AASHO ROAD TEST SUBBASE MATERIAL

Month	AASHO Road Test Report 5 (Ref 70)		NCHRP 1-11 (Ref 117) E lbs/sq in.	Average Value of E reported in Ref 66	Jain Model (Ref 94)**		Heuklom & Klomp (Ref 60) $2 \times E_{\text{subgrade}}$	Page 79, Table 17 of Ref 66 Test Results of AASHO Subbase Materials by Various Agencies**			Values Adopted E lbs/sq in.		
	M.C.	CBR			Strength Coefficients A <sub>3</sub>	E lbs/sq in.		M.C.	CBR	E From NCHRP 1-11			
	1	2										3	4
Jan.	5.0*	26	14,000	8,000**	0.11	9,300	13,200	1. 11.7	45	17,000	13,200		
Feb.	5.0*	26	14,000				13,200	2. 7.3	27	13,500	13,200		
March	5.0*	26	14,000				13,200	3. 8.0	28	14,000	7,200		
April	7.6	12	10,800		0.09	6,200	7,200	8,600	4. 6.8	66	18,000	9,800	
May	7.0	15	11,200		8,600	5. 5.7	16	11,400	10,800	5. 5.7	16	11,400	10,800
June	6.2	17	11,500		9,800	6. 7.2	40	16,000	12,200	6. 7.2	40	16,000	12,200
July	5.7	19	12,300		10,800	7. 7.9	42	16,500	11,600	7. 7.9	42	16,500	12,400
Aug.	5.4	21	13,000		11,600	8. 7.4	47	17,500	12,400	8. 7.4	47	17,500	12,800
Sep.	5.3	22	13,200		12,200	9. 8.3	41	16,000	12,800	9. 8.3	41	16,000	13,200
Oct.	5.2	23	13,400		12,400								
Nov.	5.1	25	13,700		12,800								
Dec.	5.0*	26	14,000		13,200								

\* Assumed Values

\*\* Values do not correspond to any particular month

TABLE A2.9. MODULUS OF RESILIENCE FOR AASHO ROAD TEST SUBGRADE MATERIALS

Month	AASHO Road Test Report 5 (Ref 70)		NCHRP 1-11 (Ref 117) E lbs/sq in.	Average Values of E Reported in Ref 66**	Heuklom & Klomp (Ref 60) E=1420x CBR	Page 70, Ref 66. Test Results of Subgrade Material by Various Agencies**			Fig 16 Page 555 (HRB Proc Vol 34) & Fig 5 (C.K. Chan & S.B. Seed) AASHO Subgrade & Vicksburg Clay Test Results ***					NCHRP 35 Page 15 and Table 5			Values Adopted E lbs/sq in.
	M.C.	CBR				M.C.	CBR	E NCHRP 1-11	M.C.	Stress	Strain	E Vicksburg	E AASHO	M.C.	Coffman et al psi	Seed et al psi	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Jan.	15.0*	3.7*	4700	5500	5300	15.9	2.8	4000						13.5	6200	13000	6600
Feb.	15.0*	3.7*	4700		5300				14.0	80	.01	8000	6400				6600
March	15.0*	3.7*	4700		5300	16.2	2.6	3700						15	4900	8000	6600
April	16.9	2.0	3000	1300	2840	14.8	3.7	4700	15.1	80	.0125	6400	5000				3600
May	16.5	2.4	3500		3400				16.7	80	.042	1900	1500	16	4500	6600	4300
June	16.1	2.8	4000		4000	14.5	3.8	4800									4900
July	15.8	3.1	4100		4400	16.1	1.5	2500	17.2	80	.05	1200	1000				5400
Aug.	15.5	3.3	4300		4700												5800
Sep.	15.3	3.4	4400		4800	13.4	4.5	5200									6100
Oct.	15.2	3.5	4500		5000	12.7	7.2	7500									6200
Nov.	15.1	3.6	4600		5100												6400
Dec.	15.0*	3.7*	4700		5300												6600
															Nijboer & Metcraft give values:		
															After frost = 8500 psi		
															Before frost = 21000 psi		

\* Assumed values

\*\* Values do not correspond to any particular month

\*\*\* For the same strain AASHO soil required about 80 percent stress of Vicksburg clay

Reference AASHO Subgrade Test results page 616 of First Conference on Structural Design of Asphalt Pavement, at M.C. of 15.3 the value of E varied from 3000 to 12000 psi.

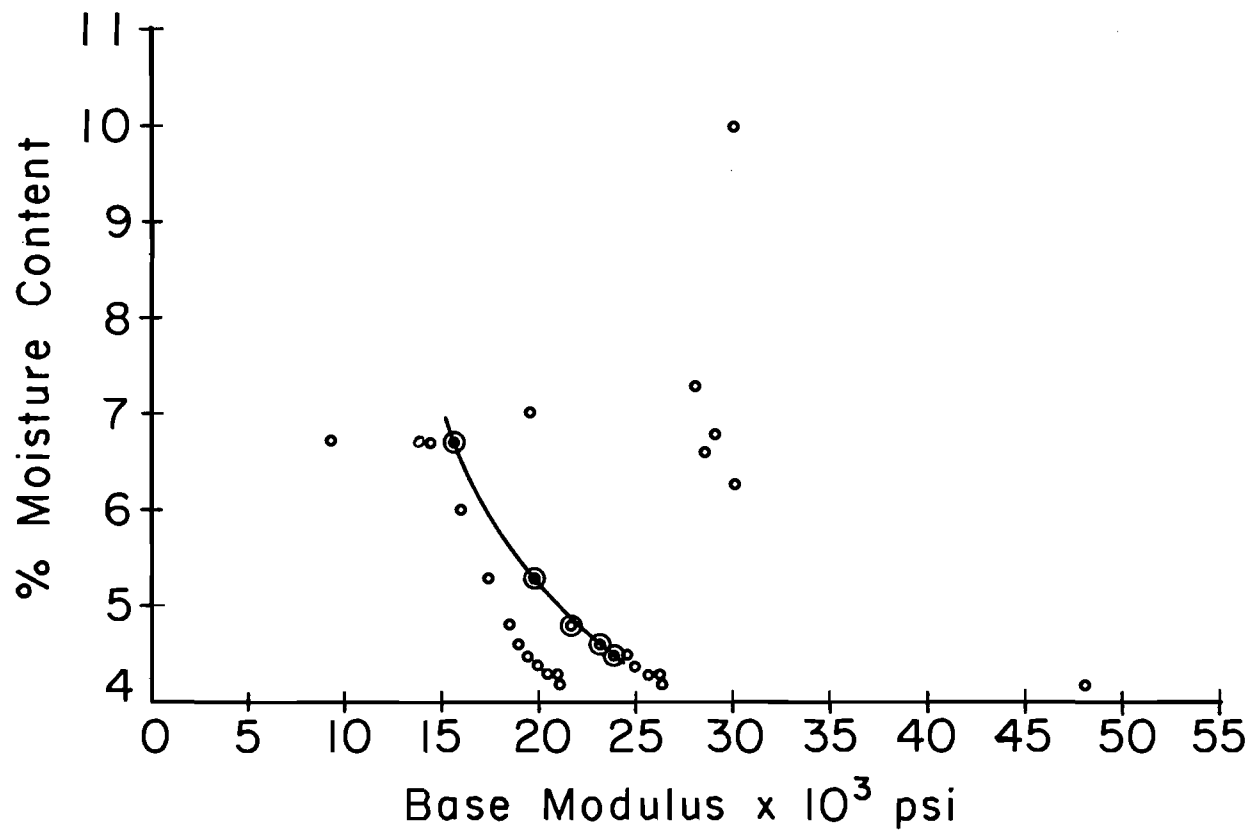


Fig A2.2. Base modulus versus moisture content.

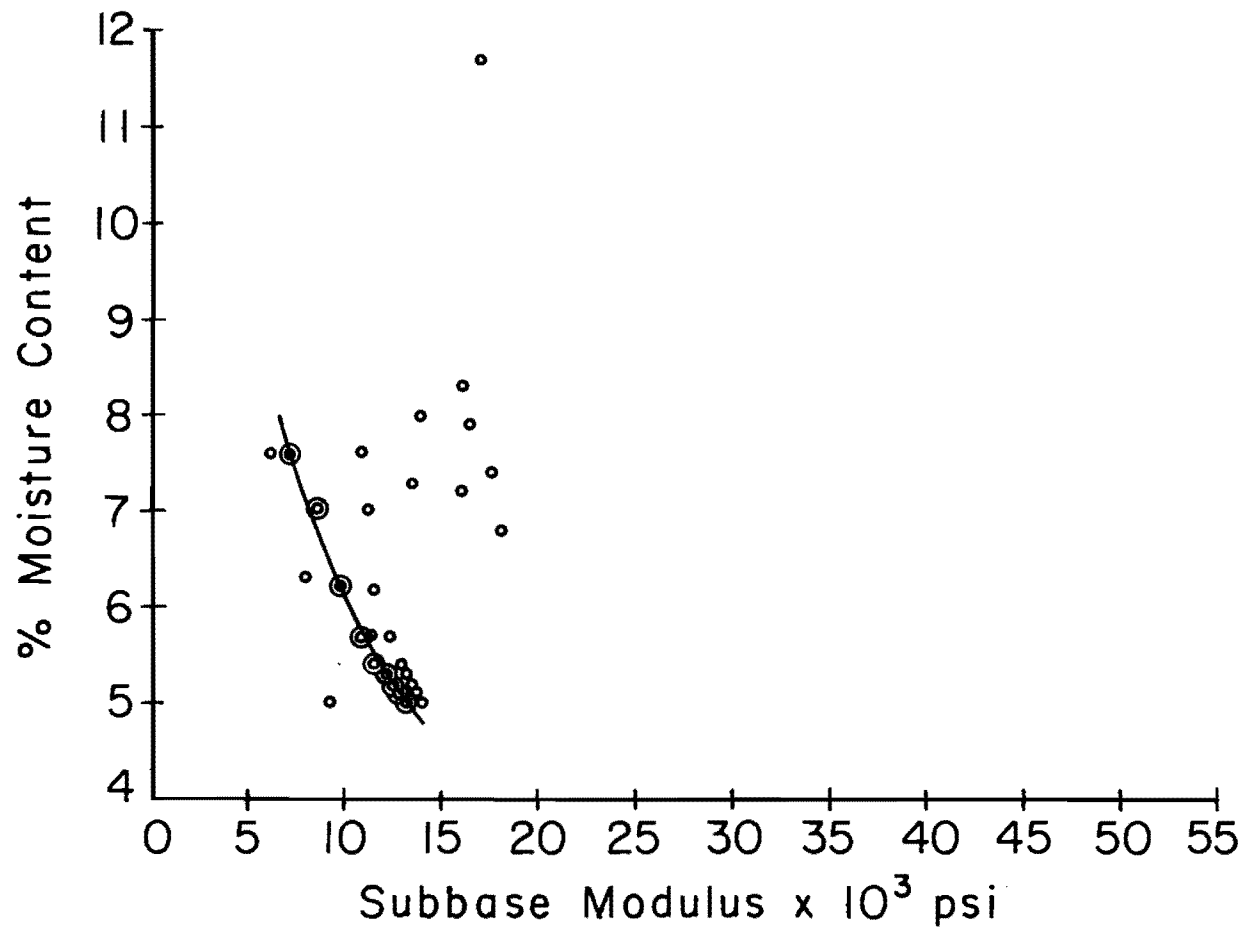


Fig A2.3. Subbase modulus versus moisture content.

TABLE A2.10. VARIATION IN THE STIFFNESS VALUES OF ASPHALT CONCRETE  
(Ref Table 2, NCHRP-39 pp 12)

Stiffness Modulus E lbs/sq in. $\times 10^5$	Std Deviation $E_{sd}$ lbs/sq in. $\times 10^5$	Coefficient of Variation % 1/2	Number of Samples	Product 3 $\times$ 4	Remarks
6.80	1.53	22.5	19	427.5	
7.03	1.91	27.2	20	544.0	
7.12	1.41	19.8	20	396.0	
5.90	1.11	18.8	19	357.2	
1.79	0.42	23.4	19	444.6	
1.65	0.39	23.6	20	472.0	
1.52	0.41	27.0	20	540.0	
1.34	0.37	27.6	19	524.4	
5.95	1.54	25.9	12	310.8	
4.40	0.90	20.4	8	163.2	
4.96	1.22	24.6	10	246.0	
1.57	0.42	26.8	12	321.6	
1.39	0.26	18.7	12	224.4	
1.47	0.41	27.8	8	222.4	
1.42	0.42	29.6	10	296.0	
Total			228	5490.1	Avg. Coeff. of variation $\frac{5490}{228}=24.0$



TABLE A2.11. VARIATION IN THE STIFFNESS MEASUREMENTS ON PAVEMENT SAMPLES  
 (Ref Page 136 2nd International Conference on Structural  
 Design of Asphalt Pavements)

Location	No. of Specimens	Measured Stiffness, psi $\times 10^5$						Mean Coefficient of Variation %
		68° F			40° F			
		Mean	Standard Deviation	Coefficient of Variation %	Mean	Standard Deviation	Coefficient of Variation %	
Surface Course	20	1.52	0.41	27.0	7.12	1.41	20.0	23.5
Base Course	8	1.47	0.41	28.0	4.40	0.90	20.0	24.0

AASHO Road Test Report 5 (Ref 70), in Figs 96 and 97, gives the moisture content and CBR value, of the AASHO Road Test pavement subgrade for various months. This information is entered in Columns 1 to 3 of Table A2.9. Based on NCHRP Report 1-11 (Ref 117) Column 4 contains the estimated value of resilient modulus. Special Report 66 (Ref 66) contains the test results of the AASHO Road Test material. The reported values of resilient modulus vary from 1,300 psi to 5,500 psi. The test results in this report also show the CBR values for various moisture contents. These values of CBR with corresponding estimated values (based on NCHRP Report 1-11) of resilient modulus are shown in Column 9. Heukelom and Klomp (Ref 60) have given an approximate relation between CBR and resilient modulus. An evaluation of modulus values based on this criterion is entered in Column 6. In Highway Research Board proceedings Vol 34, Chan and Seed have reported the stress-strain test results on AASHO subgrade and Vicksburg clay at various moisture contents. From these test results it was observed that for the same strain AASHO subgrade clay needed about 80 percent stress in comparison to the Vicksburg clay. Based on this information estimated values of resilient modulus of AASHO subgrade soil are given in Column 14. NCHRP Report 35 (Ref 165) also contains some test results on the AASHO Road Test subgrade soil. These are tabulated in Columns 15 to 17. Fluctuations in resilient modulus values based on different criteria are apparent from Table A2.9. However, based on such information a decision on values to be adopted for design purposes is not difficult, at least for an experienced designer. Based on averaging out the available information the values adopted for the present analysis are obtained from Fig A2.4 and tabulated in Column 18 of Table A2.9.

Stochastic Variations in Elastic Modulus. Direct observations are not available to estimate the expected variations in the elastic modulus values of the subgrade, subbase, base materials, and surface asphalt concrete used in the AASHO Road Test. However, some indirect information was utilized to determine the expected statistical variations in the elastic modulus of the AASHO Road Test materials.

Asphalt Concrete. In NCHRP Report 39 (Ref 42), some test data are available for various field specimens. These are shown in Table A2.10. A weighted mean calculation of the test results of the specimens shows a coefficient of variation of about 24 percent. Table A2.11 is an extract of

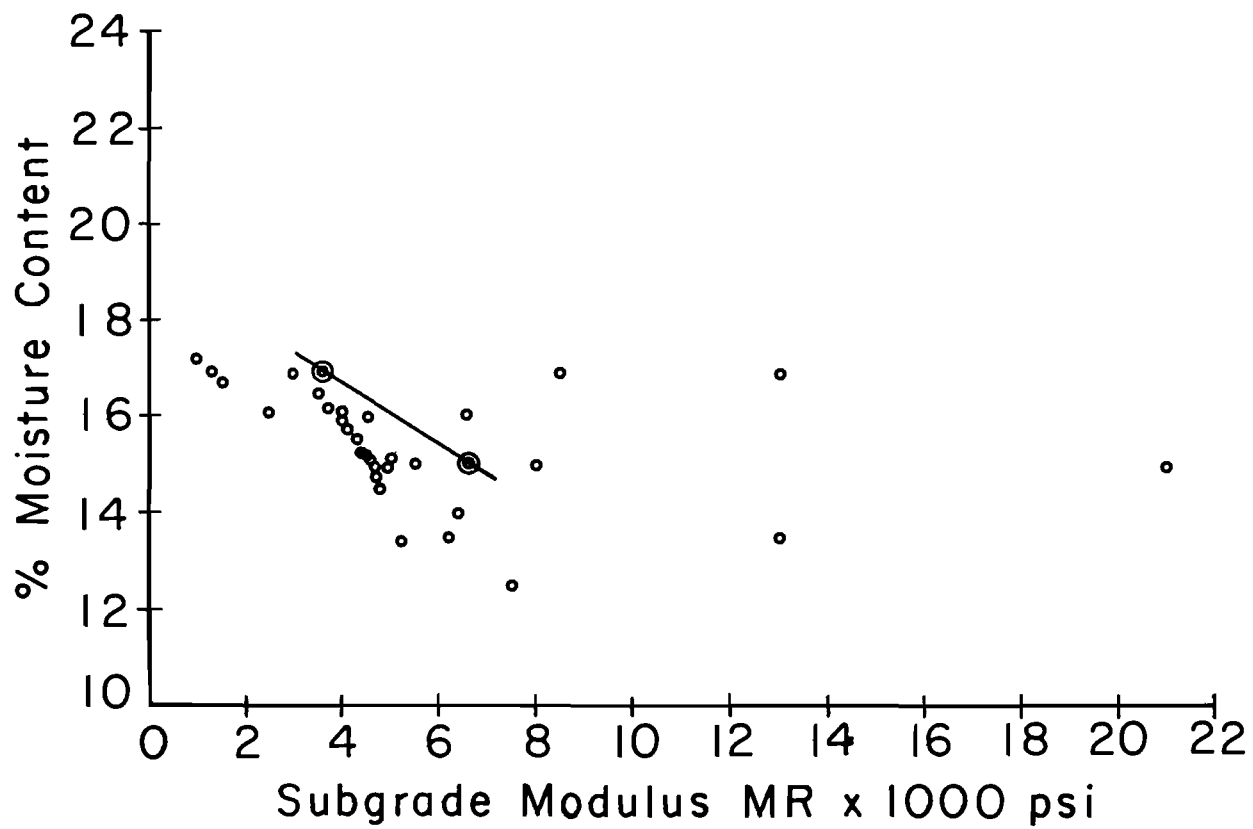


Fig A2.4. Subgrade modulus versus moisture content.

stiffness measurements of some pavement samples taken from page 136 of the Proceedings of the Second International Conference on Structural Design of Asphalt Pavements. This table gives an approximate value of coefficient of variation as 24 percent of the mean value. Table A2.12 shows the observed variations in the test results of the "stability test" performed on some asphalt concrete sample and reported in Public Roads, August 1969. The coefficient of variation in these test results varied from 12.2 to 23 percent. Based on this information, a value of coefficient of variation of 25 percent is assumed for analysis in this report.

Untreated Granular Base Material. Test results on AASHO Road Test materials for CBR values as reported in Special Report 66 (page 96 of Ref 66) are reproduced below with the statistical calculations made for the coefficient variation (page 40, Statistical Methods, by Snedecor).

Item	Number of Samples	Mean Value	Range	Expected Standard Deviation	Coefficient of Variation, %
CBR	5	100	83 - 140	$57 \times 0.53 = 30$	30

The coefficient of variation is about 30 percent of mean value. The other test results (Ref 66, page 90) show that the coefficient of variation was observed as 3 percent for maximum density and 16 percent for optimum moisture content. The results of 24 tests on the gravel base mixture (Ref 66, page 73) gave an average liquid limit of 18.9 and an average plasticity index of 3.1 with corresponding standard deviations of 2.1 and 1.9 which will give coefficients of variation of 11 percent and 60 percent, respectively. While other test results may not give a direct value of statistical variations expected in resilient modulus, the CBR has a direct correlation with this parameter. A 30 percent coefficient of variation calculated above for CBR may also be expected for the resilient modulus. However, this information is based on five samples and can only be treated as approximate. For the analysis in this report, a value of 25 percent for the coefficient of variation was adopted.

TABLE A2.12. HOT MIX MARSHALL TEST DATA "VARIATION IN STABILITY TEST VALUES"  
(Ref "Public Roads" August 1969)

Test	Project Nos.	States Nos.	Average Standard Deviation	Average Mean	Coefficient of Variation %
Marshall Stability lbs	18	4	283.00	2305.00	12.2
Marshall Flow 100/in	15	2	1.29	8.62	15.0
Marshall Air Voids Pct	18	4	1.00	4.33	23.0

Untreated Granular Subbase. The results of the CBR test in Special Report 66, Ref 66) are given below.

Item	Number of Samples	Mean Value	Range	Expected Standard Deviation	Coefficient of Variation, %
CBR	5	58	32 - 86	$54 \times 0.53 = 29$	50

The Computed Coefficient of Variation for the CBR value from the above test results is 50 percent.

Also, Fig 60 of AASHO Road Test Report 2 (Ref 67) gives the results of CBR determinations on 80 test samples. This shows a mean value as 34.7 and standard deviation 9.3. The coefficient of variation works out as 26.8 percent. Page 90 of Special Report 66 (Ref 66) gives the coefficient of variation in the measurement of maximum density as 2.5 percent, while in optimum moisture content it was 13 percent. Thus, based on the above results, the subbase coefficient of variation is expected to be of the same order as for the base material. Therefore, for analysis in this report, a 25 percent value for the coefficient of variation for the AASHO Road Test Subbase is adopted.

Fine Grained Subgrade. The data for CBR are given below, according to page 96 of Special Report 66.

Item	Number of Samples	Mean Value	Range	Expected Standard Deviation	Coefficient of Variation, %
CBR	8	5.0	4 - 6.7	$.351 \times 2.7 = 1.0$	20

Figure 49 of AASHO Road Test Report 2 (Ref 67) gives the CBR determination of 80 test samples. The test shows a mean value of CBR as 2.9 and its standard deviation as 1.28. The coefficient of variation on this basis works out as 44 percent of the mean value. Page 90 of Ref 66 gives the coefficient of variation for optimum moisture content as 7.4 percent and for maximum density as 1.8 percent. Page 32 of AASHO Road Test Report 2 (Ref 67) shows statistical variations found for the Atterbergs limit test results, as follows:

Test	Coefficient of Variation, %
Liquid limit	19.5
Plastic limit	11.5
Plasticity index	31.0
Optimum moisture content	14.2
Maximum dry density	2.5

Based on the above test results it is seen that the average statistical variations in the test results of the subgrade are expected to be about the same order as for the base and subbase. Therefore, for the analysis in this report, a value of 25 percent for coefficient of variation is adopted.

TABLE A2.13. VARIABLES AFFECTING MATERIAL RESPONSE

- I. Loading variables
  - A. Stress history (nature of prior loading)
    - 1. Non-repetitive loading (such as preconsolidation)
    - 2. Repetitive loading
      - a. Nature
        - (1) Simple
        - (2) Compound
      - b. Number of repetitive applications
  - B. Initial stress state (magnitude and direction of normal and shear stresses)
  - C. Incremental loading
    - 1. Mode of loading
      - a. Controlled stress (or load)
      - b. Controlled strain (or deformation)
      - c. Intermediate modes
    - 2. Intensity (magnitude and direction of incremental normal and shear stresses)
    - 3. Stress path (relation among stresses - both normal and shear - as test progresses)
    - 4. Time path
      - a. Static
        - (1) Constant rate of stress (or load)
        - (2) Constant rate of strain (or deformation)
        - (3) Creep
        - (4) Relaxation
      - b. Dynamic
        - (1) Impact
        - (2) Resonance
        - (3) Other
          - (a) Sinusoidal (rate of loading is variable)
          - (b) Pulsating (duration, frequency, and shape of load curve are variables)
    - 5. Type of behavior observed
      - a. Strength (limiting stresses and strains)
      - b. Deformability
    - 6. Homogeneity of stresses
    - 7. Drainage (drained or undrained)
- II. Mixture variables
  - A. Mineral particles
    - 1. Maximum and minimum size
    - 2. Gradation
    - 3. Shape
    - 4. Surface texture
    - 5. Angularity
    - 6. Mineralogy
    - 7. Adsorbed ions
    - 8. Quantity

(Continued)



TABLE A2.13. (Continued)

- B. Binder
    - 1. Type
    - 2. Hardness
    - 3. Quantity
  - C. Water
    - 1. Quantity
  - D. Voids
    - 1. Quantity
    - 2. Size
    - 3. Shape
  - E. Construction Process
    - 1. Density
    - 2. Structure
    - 3. Degree of anisotrophy
    - 4. Temperature
  - F. Homogeneity
- III. Environmental variables
- A. Temperature
  - B. Moisture
  - C. Alteration of material properties
    - 1. Thixotropy
    - 2. Aging
    - 3. Curing
    - 4. Densification

TABLE A2.14. TEST CONFIGURATIONS

- I. Tension
  - A. Uniaxial tension
  - B. Indirect tension
    - 1. Splitting tension
    - 2. Cohesimeter
  
- II. Compression
  - A. Unconfined, uniaxial compression
  - B. Triaxial compression
    - 1. Open system
      - a. Conventional triaxial compression
        - (1) Normal
        - (2) Vacuum
        - (3) High pressure
      - b. Box with cubical specimen
    - 2. Closed system
      - a. Oedometer
      - b. Cell
      - c. Hveem stabilometer
  
- III. Flexure
  - A. System
    - 1. Revolving bar
    - 2. Simple flexural
  - B. Loading
    - 1. Cantilever
    - 2. Simple beam
      - a. Point supports
      - b. Uniform supports
  
- IV. Direct shear
  - A. Direct shear (rigid split box)
  - B. Double direct shear
  - C. Uniform direct shear (rigid caps with confined rubber membrane and split rings for lateral restraint)
  - D. Uniform strain direct-shear (hinged box)
  - E. Punching shear
  
- V. Torsion
  - A. Pure torsion
  - B. Triaxial torsion
  - C. Specimen shape
    - 1. Solid cylinder
    - 2. Thick-walled, hollow cylinder

TABLE A2.15. SPECIMEN SHAPE

- I. Rectangular parallelepiped
  - A. Short
  - B. Long
  - C. Cubic
  
- II. Cylinder
  - A. Solid
    - 1. Short
    - 2. Long
  - B. Thick-walled, hollow
    - 1. Short
    - 2. Long
  
- III. Plate
  
- IV. Other

TABLE A2.16. SUMMARY OF CHARACTERISTICS OF RESEARCH MATERIALS

Designation	Description	Gradation	Compaction Characteristics*		Actual Unit Weight, D <sub>A</sub> 100% Compaction Ratio pcf	Texas Triaxial Test			
			Optimum Moisture %	Maximum Unit Weight pcf		Average Moisture After Capillarity %	Failure Stress At Indicated Lateral Pressure, psi		Triaxial Class
							0 psi	15 psi	
HP-27-8	Rounded	Fine	7.3	133.9	-----	7.4	18.1	147.7	3.0
		Medium	6.8	135.4	133.8	7.0	23.8	158.7	2.8
		Coarse	6.7	135.2	-----	7.3	23.2	161.9	2.7
HP-27-9	Angular	Fine	7.3	133.9	-----	7.0	42.1	223.2	1**
		Medium	7.0	136.0	136.0	6.8	62.1	246.9	1***
		Coarse	6.8	137.7	-----	5.9	57.7	270.7	1****
HP-27-10	Soft	Fine	11.9	124.2	-----	11.5	28.8	169.5	2.5
		Medium	11.9	124.2	124.3	11.6	52.0	167.8	2.1
		Coarse	11.9	124.2	-----	11.3	48.2	175.4	2.1

\* Compactive effort = 13.26 ft lbs per cu in.  
 \*\* Lowest classification of HP-27-9  
 \*\*\* Medium classification of HP-27-9  
 \*\*\*\* Highest classification of HP-27-9

(Continued)

TABLE A2.16. (Continued)

Designation	Description	Gradation	Plasticity			Los Angeles Abrasion ("A" Grading)		Texas Wet Ball Mill	Classification		Specific Gravity	Permeability ft/day
			Liquid Limit Liquid Class	Plasti-city Index	Linear Shrink-age	100 rev.	500 rev.		Texas	Unified		
HP-27-8	Rounded	Fine						37.2	Type B, Grade 3	GMd	2.64	-----
		Medium	21.3	7.4	5.6	7.2	27.3	36.2	Type B, Grade 3	GMd	2.63	0.006
		Coarse						32.0	Type B, Grade 3	GMd	2.65	-----
HP-27-9	Angular	Fine						39.0	Type A, Grade 2	GMu	2.64	-----
		Medium	17.8	2.3	2.4	6.8	25.3	36.1	Type A, Grade 1	GMu	2.63	0.003
		Coarse						33.5	Type A, Grade 1	GMu	2.64	-----
HP-27-10	Soft	Fine						50.3	Type A, Grade 2	GMu	2.67	-----
		Medium	20.2	4.8	2.7	19.0	57.9	47.8	Type A, Grade 2	GMu	2.67	0.002
		Coarse						48.1	Type A, Grade 2	GMu	2.67	-----

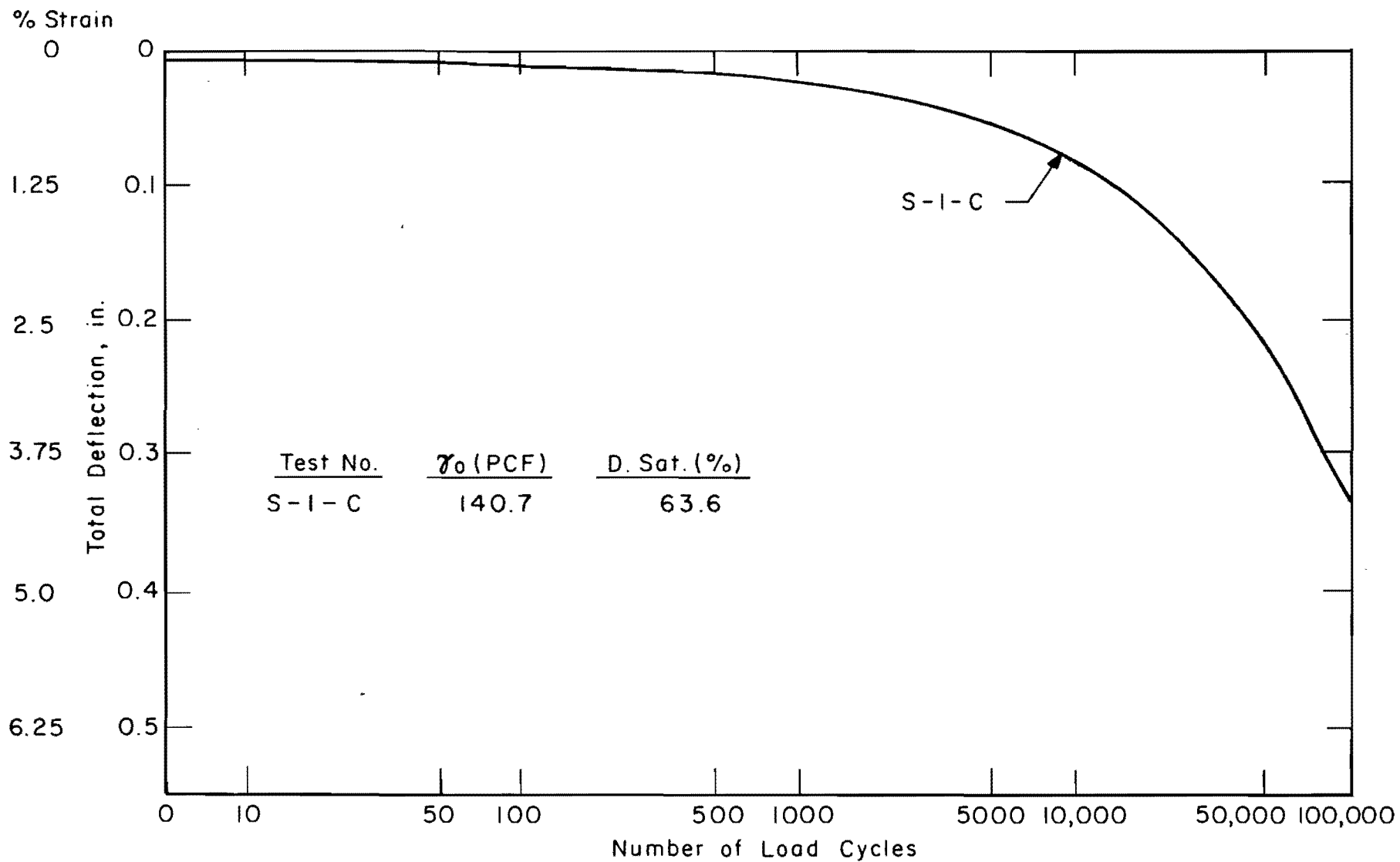


Fig A2.5. Deflection history of crushed stone specimens 11.5 percent passing No. 200 sieve (after Ref 57).

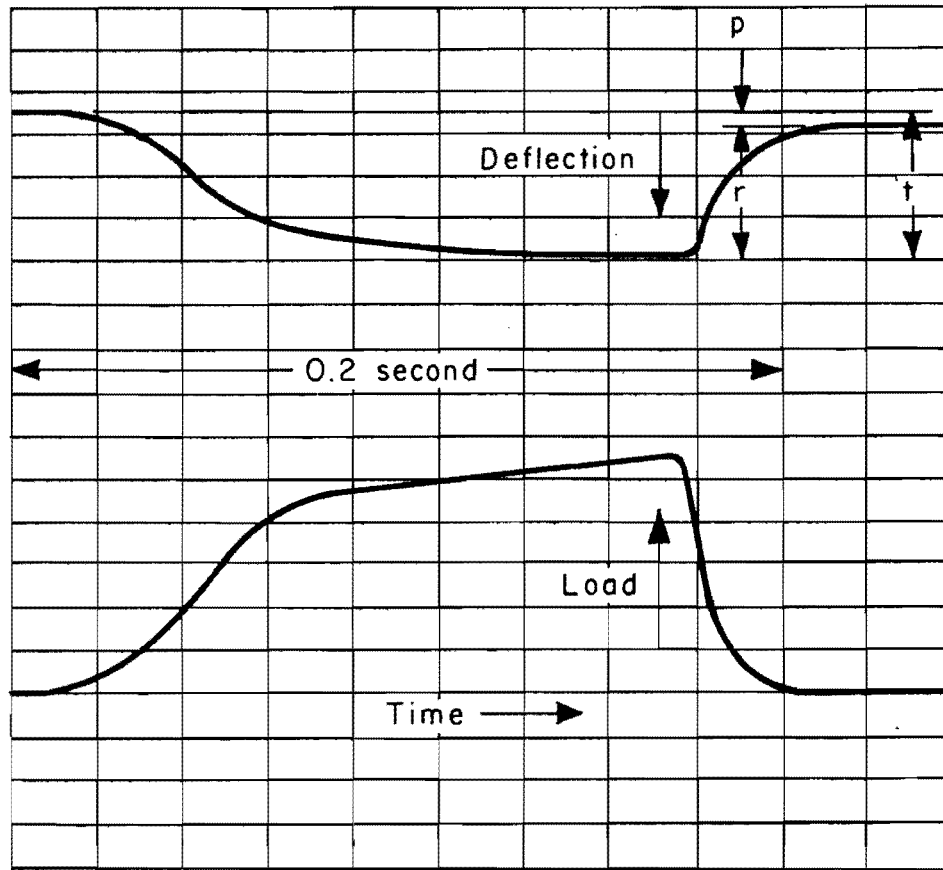


Fig A2.6. A typical load-deformation oscillograph (A&M material) (after Ref 35).

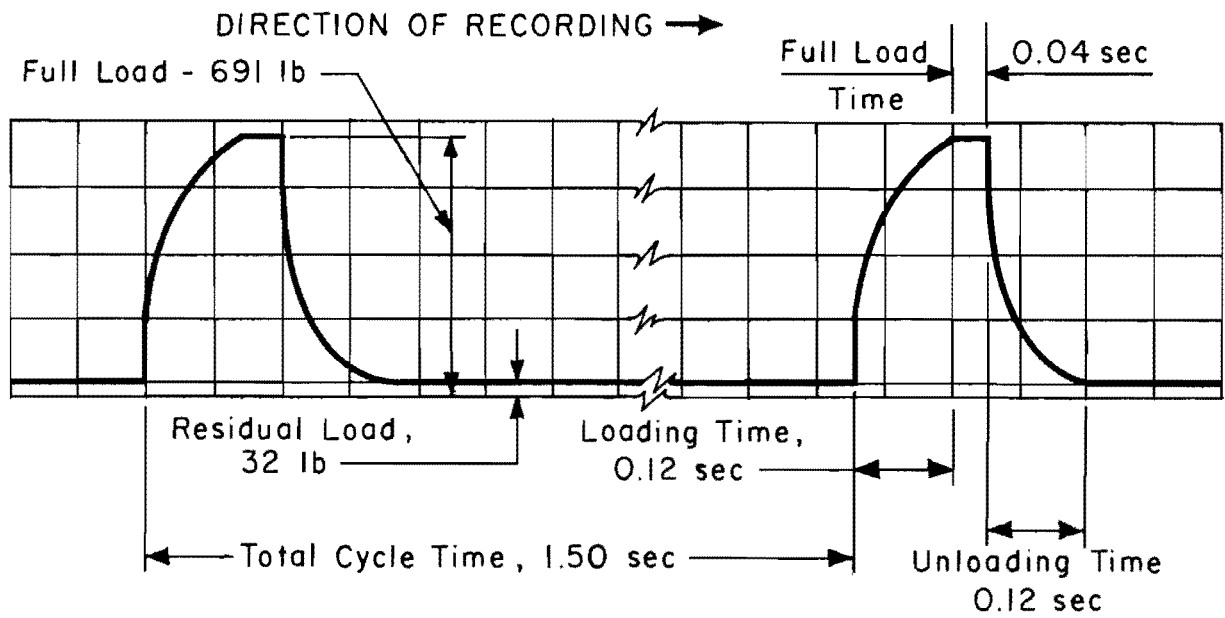


Fig A2.7. Typical load-time trace for two load cycles (AASHO material) (after Ref 57).

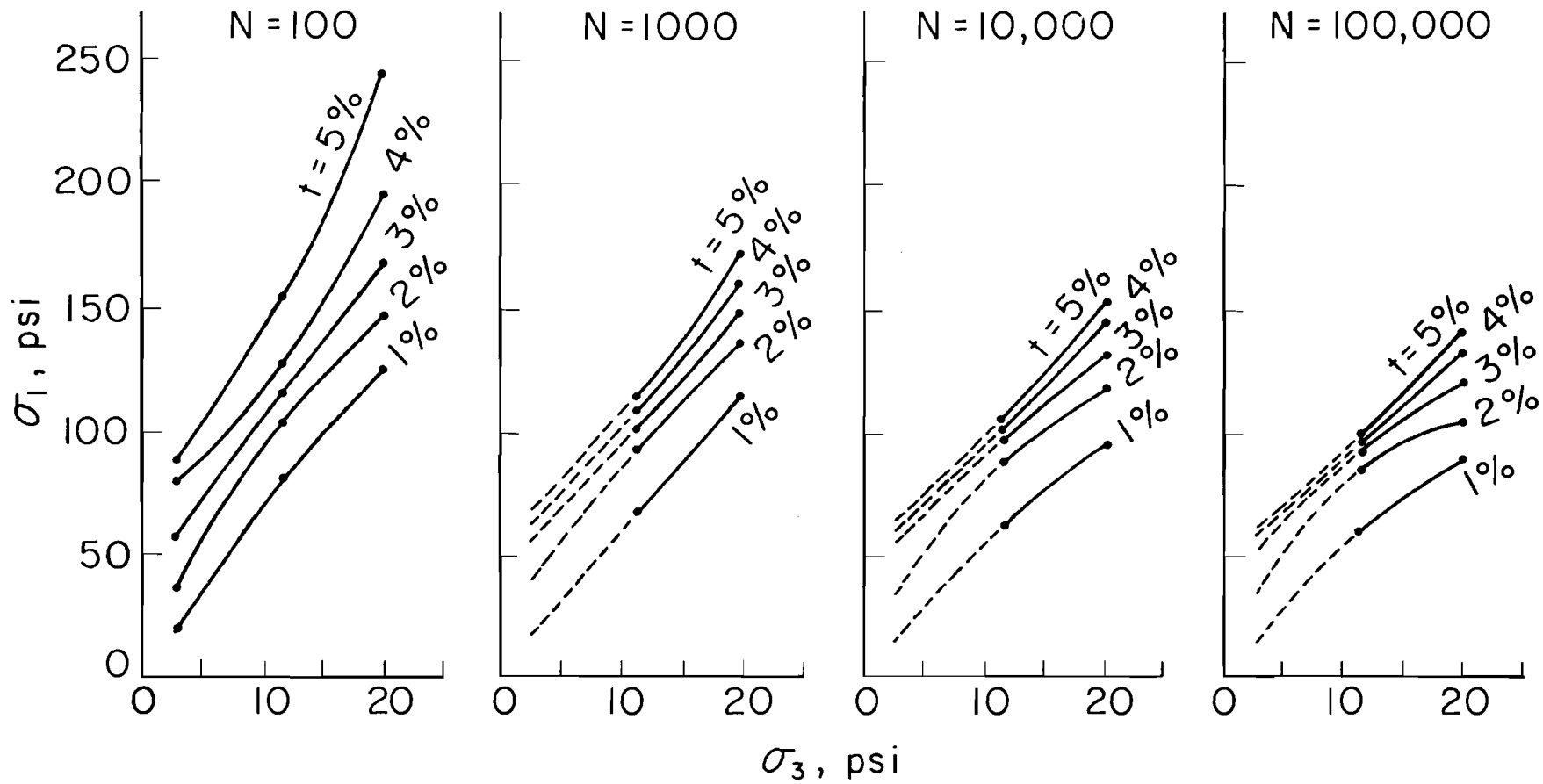


Fig A2.8. Angular medium aggregate similar to AASHO base material (after Ref 35).



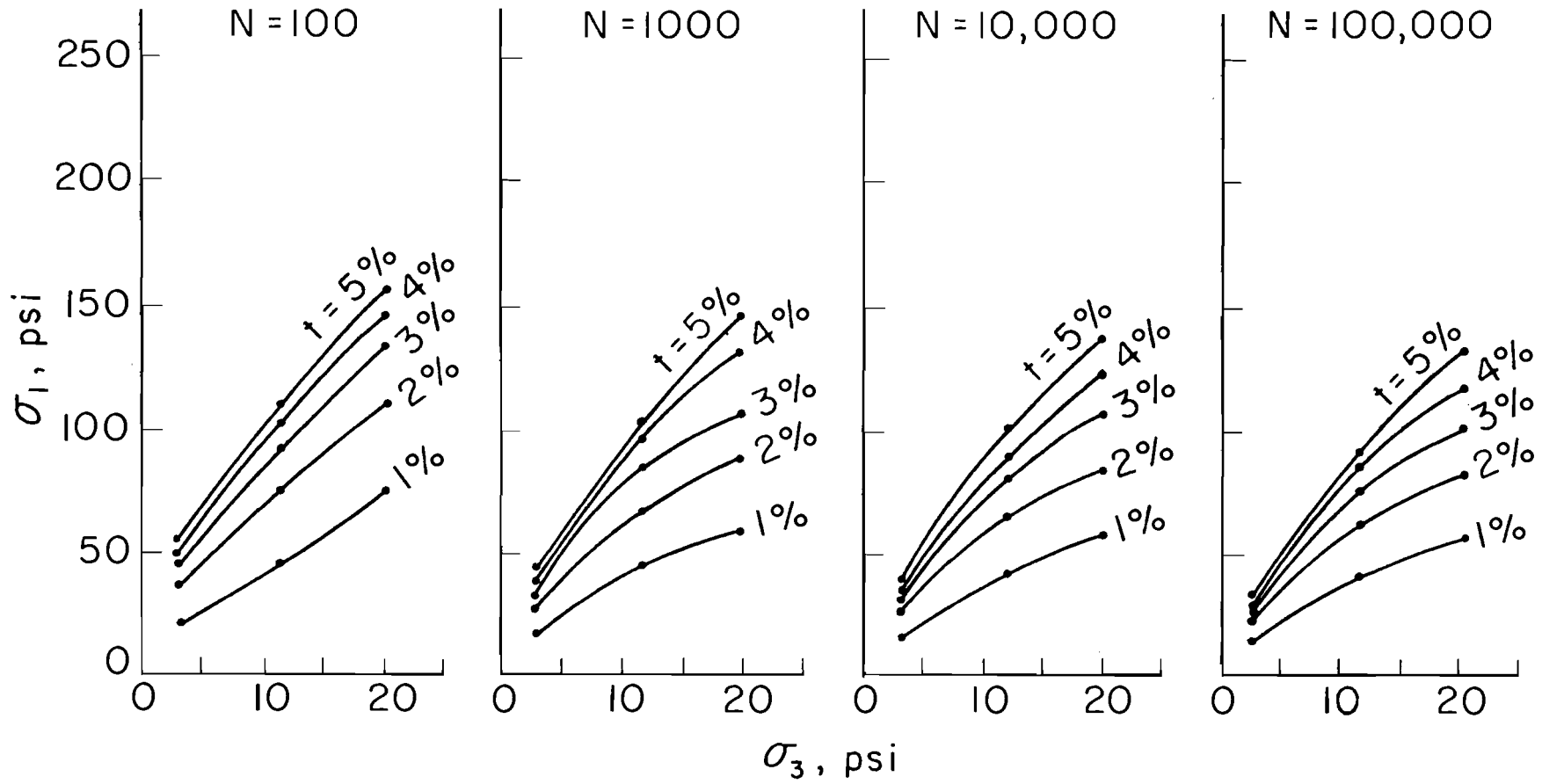


Fig A2.9. Rounded fine aggregate similar to AASHO subbase material (after Ref 35).

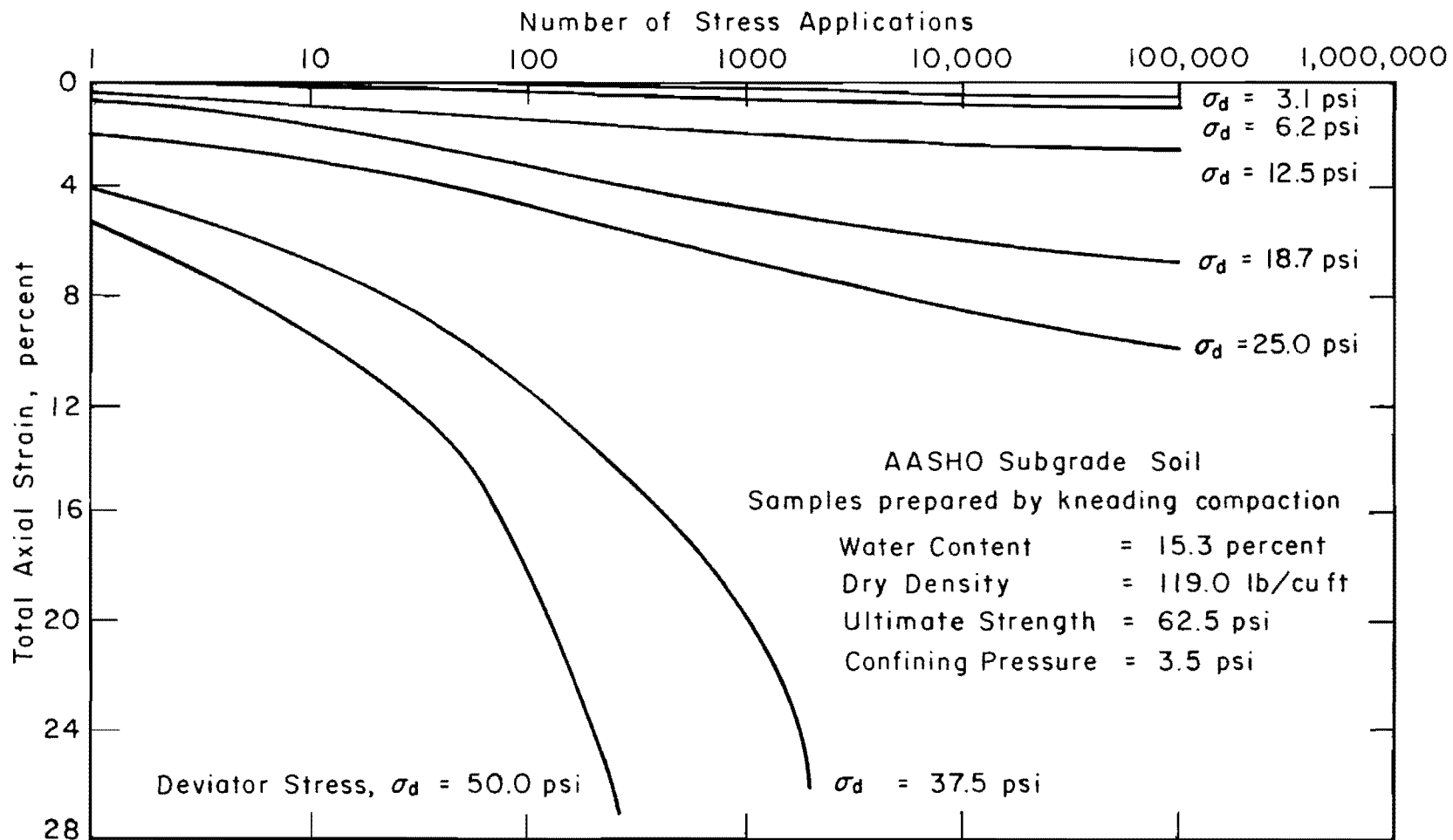


Fig A2.10. Deformation characteristics - AASHO Road Test subgrade soil (after Ref 165).

APPENDIX 3

OUTLINE OF COMPUTER PROGRAM AVAILABLE FOR ANALYSIS OF STRESSES,  
STRAINS, AND DISPLACEMENTS IN A FIVE-LAYERED ELASTIC SYSTEM  
UNDER A LOAD UNIFORMLY DISTRIBUTED ON A CIRCULAR AREA

APPENDIX 3. OUTLINE OF COMPUTER PROGRAM AVAILABLE FOR ANALYSIS OF STRESSES, STRAINS, AND DISPLACEMENTS IN A FIVE-LAYERED ELASTIC SYSTEM UNDER A LOAD UNIFORMLY DISTRIBUTED ON A CIRCULAR AREA

This program was developed by H. Warren and W. L. Dieckmann of California Research Corporation, Richmond, California, in 1963 and is based on the solution which was used by Mr. J. Michelow (California Research Corporation) in his analysis of multi-layered asphalt pavement system. The following is the outline of the computer program (after Ref 167).

Introduction

A. The program computes the following items numerically at any point in the layered system for a given load on a circular area of the free surface:

1. vertical, tangential, radial, shear, and bulk stress;
2. vertical displacement;
3. radial, tangential, and shear strain.

Note that a cylindrical system of coordinates is used.

B. The following input data should be provided:

1. the wheel load (total load);
2. tire pressure;
3. load radius (= total load/ $\pi$   $\times$  tire pressure);
4. for each layer, layer number, modulus of elasticity, Poisson's ratio, and thickness.

Description of the Layered System

The system consists of (5) layers of different homogeneous, ideally elastic materials. Each layer is of uniform thickness and infinite dimensions in all horizontal directions, stratified vertically over the semi-infinite bottom layer. Figure A3.1 shows complete details of the system.

Limitations

A. There are no body forces or couples present and inertia forces are neglected.

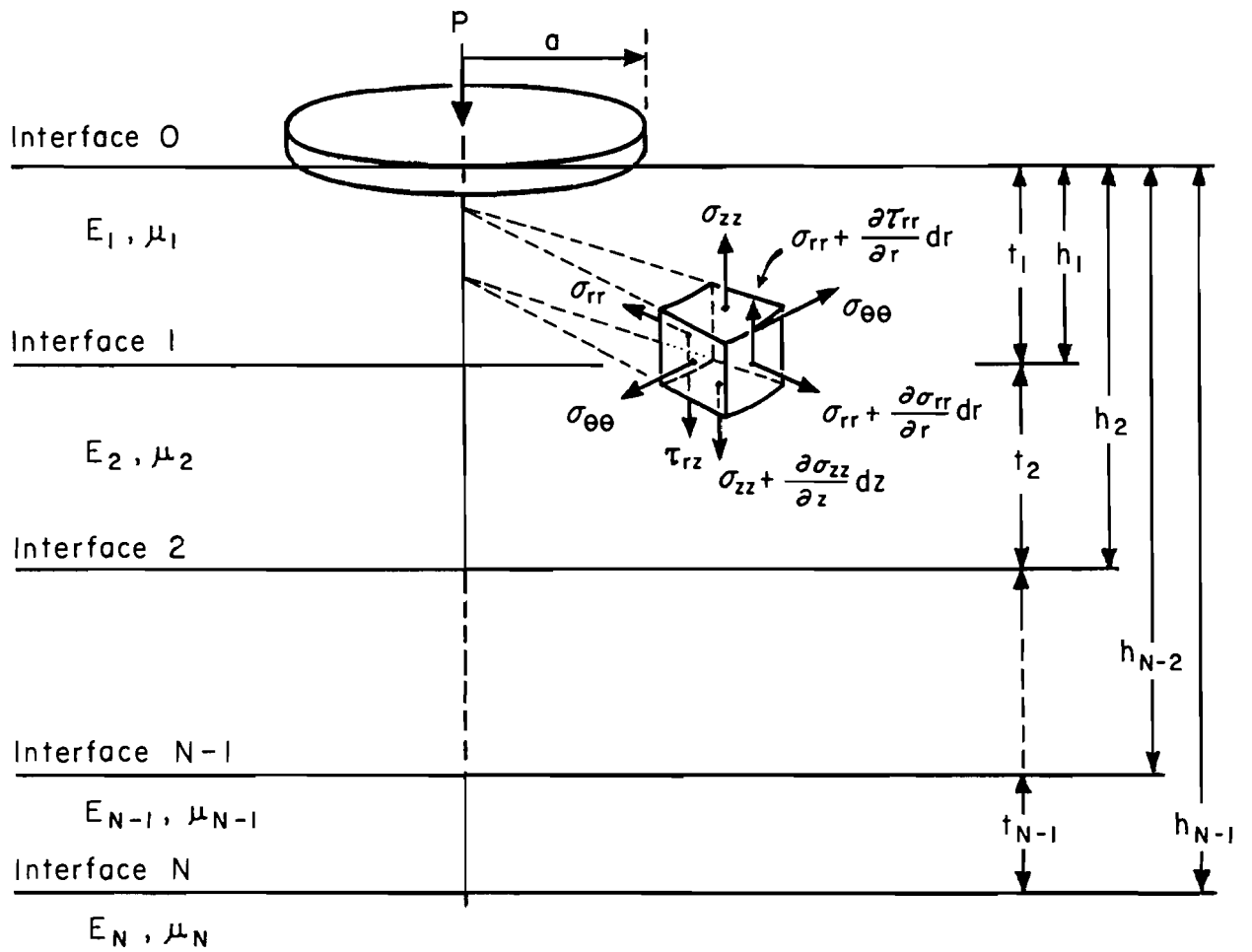


Fig A3.1. Stresses in a multi-layer system.

- B. Stresses and strains small enough to be described in an infinitesimal elastic theory.
- C. The load is uniformly distributed over a circular area.
- D. The system is axisymmetric (z is axis of symmetry), which requires that each layer is uniform, homogeneous, and isotropic.

#### Summary of the Mathematical Model

For a cylindrical system of coordinates, the components of stress are:

$$\sigma_z = \text{vertical stress}$$

$$\sigma_r = \text{radial stress}$$

$$\sigma_\theta = \text{tangential stress}$$

$$\tau_{zr}, \tau_{r\theta}, \tau_{z\theta} = \text{shear stresses.}$$

and the components of displacement are:

$$u = \text{the radial displacement}$$

$$v = \text{the tangential displacement, and}$$

$$w = \text{the vertical displacement.}$$

Because of the symmetry of the system under consideration (z axis is an axis of symmetry):

$$\tau_{r\theta} = \tau_{z\theta} = v = 0$$

Satisfying equilibrium and compatibility conditions and solving for the stresses, strains, and displacements, one finds that with each layer there are four unknowns (A, B, C, and D).

In total there are 4N unknowns to be solved from 4N boundary conditions.

#### Boundary Conditions

- A. The assumption that adjacent layers are bonded and no slip occurs at the interfaces (rough interface) gives (4N-4) boundary conditions, i.e.;

$$1. \quad \begin{matrix} i \\ i \end{matrix} \sigma_z = \begin{matrix} i+1 \\ i \end{matrix} \sigma_z$$

$$2. \quad \begin{matrix} i \\ i \end{matrix} w = \begin{matrix} i+1 \\ i \end{matrix} w$$

$$3. \quad \begin{matrix} i \\ i \end{matrix} \tau_{rz} = \begin{matrix} i+1 \\ i \end{matrix} \tau_{rz}$$

$$4. \quad \begin{matrix} i \\ i \end{matrix} u = \begin{matrix} i+1 \\ i \end{matrix} u$$

Super- and sub-prefixes refer to the layer number and the interface number in the system, respectively.

B. The load situation at the surface gives two boundary conditions:

$$1. \sigma_z = -p \quad 0 \leq r < a, \quad z = 0$$

$$2. \tau_{rz} = 0 \quad 0 \leq r < \infty, \quad z = 0$$

C. The two last boundary conditions result from the requirement that the stresses, strains, and displacements are finite at infinite depth. This will lead to the fact that two of the unknowns of the bottom layers are zero.

$$1. A_N = 0$$

$$2. B_N = 0$$

At this point one has  $4N$  unknowns and  $4N$  boundary conditions.

APPENDIX 4

COMPUTER PROGRAM AND ANALYSIS



APPENDIX 4.1

FLOW CHART

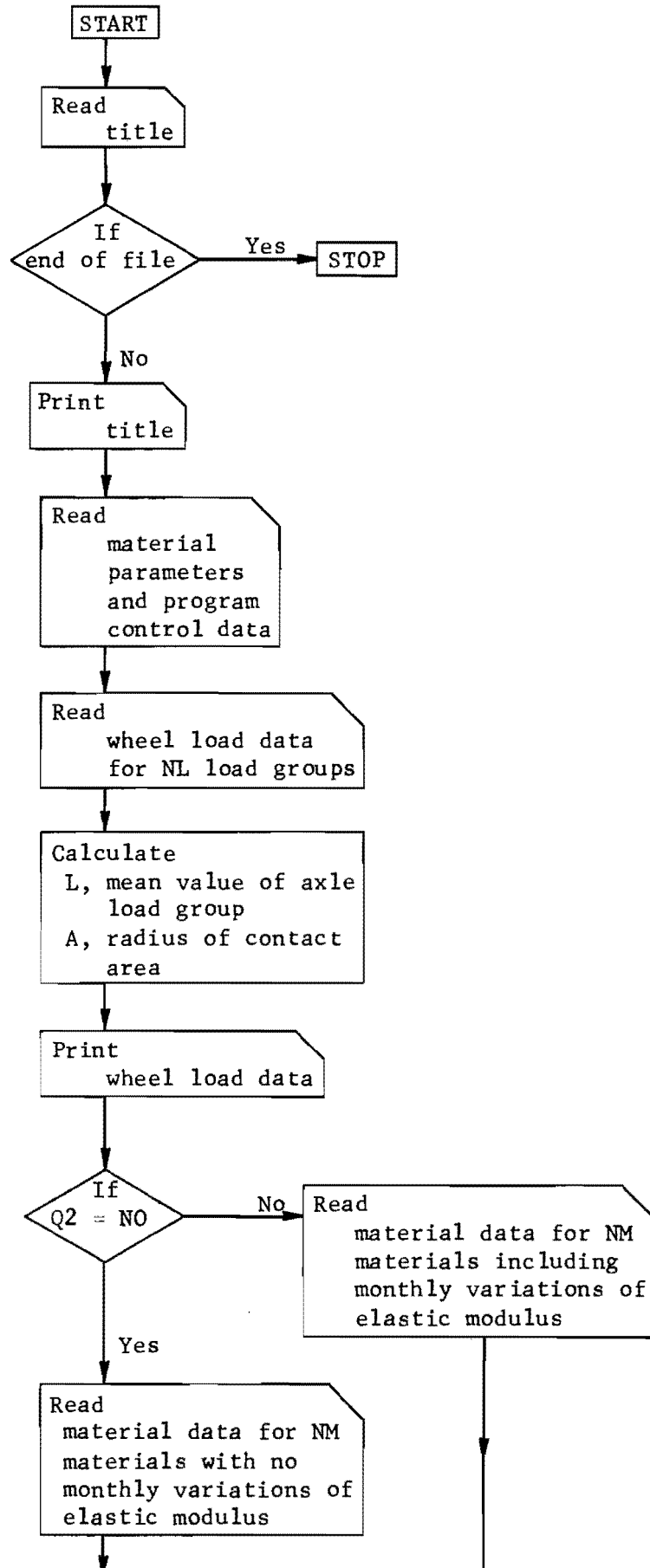


Fig A4.1. Flow chart.

(Continued)

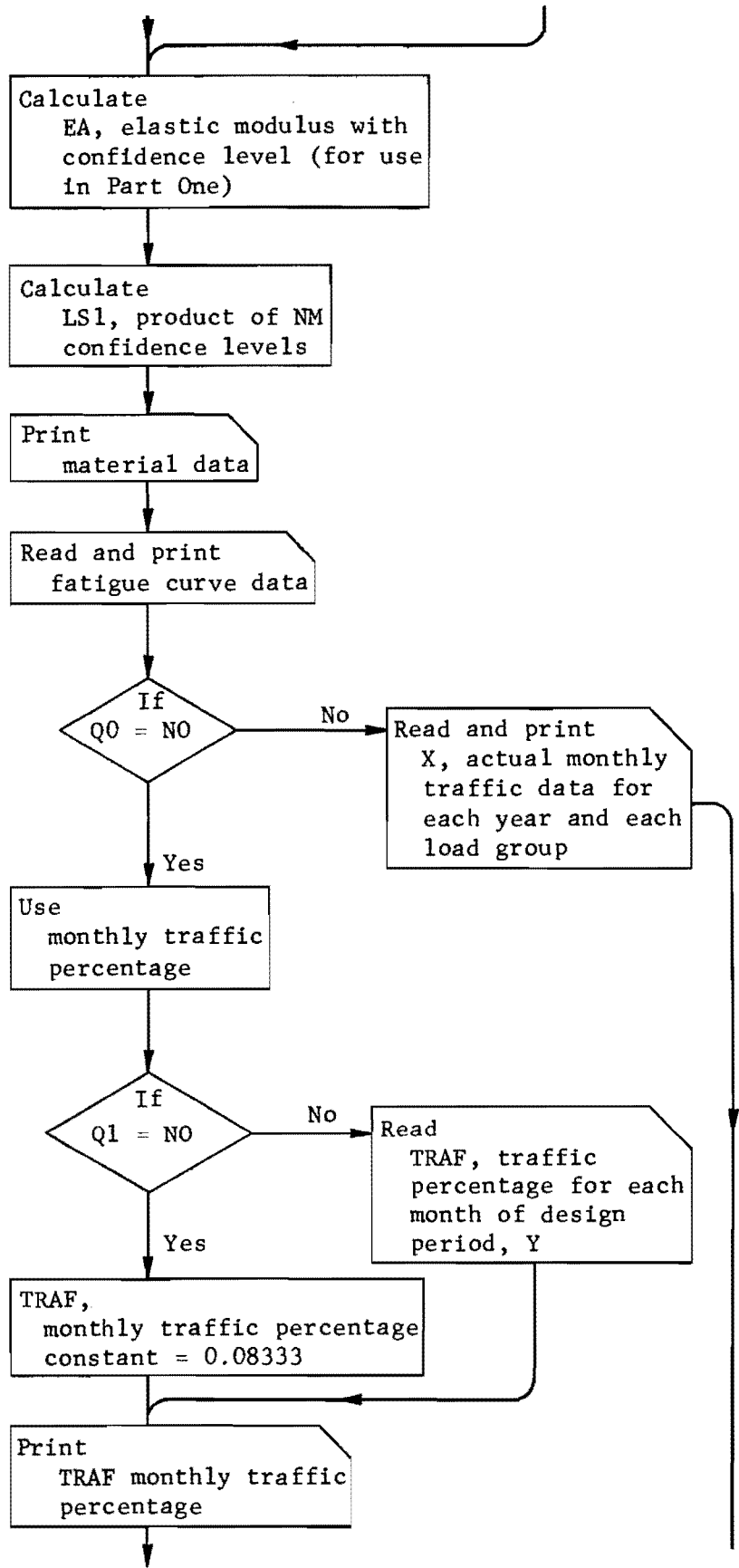


Fig A4.1. (Continued)

(Continued)

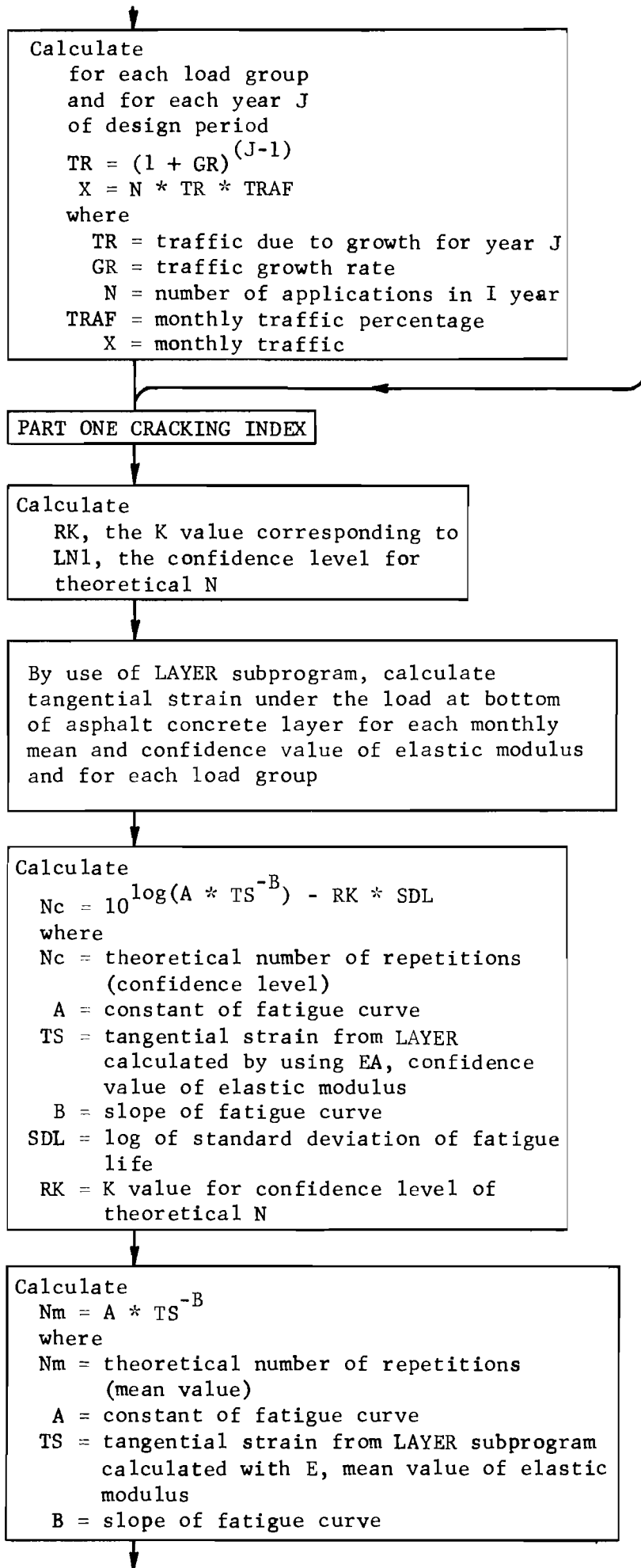


Fig A4.1. (Continued)

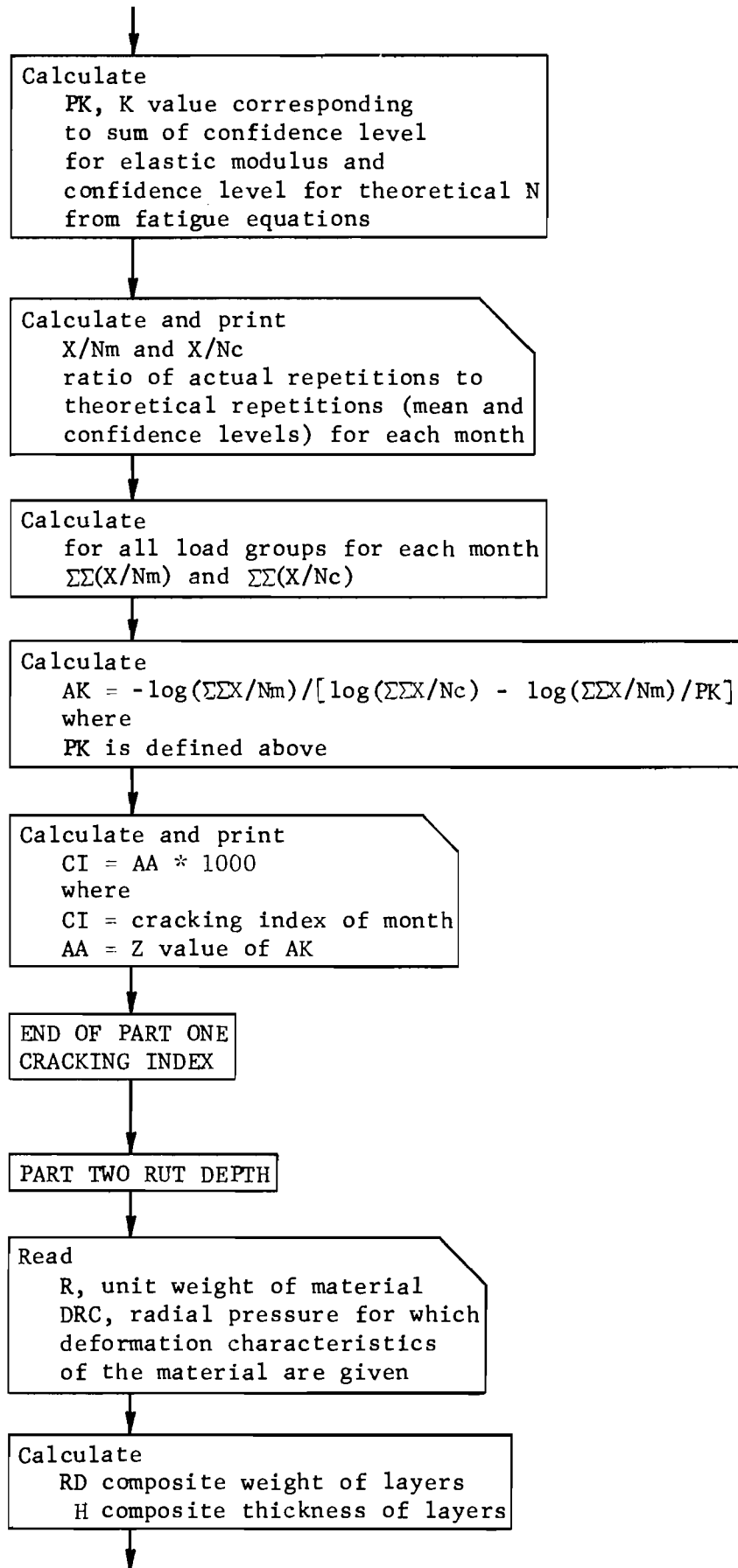


Fig A4.1. (Continued)

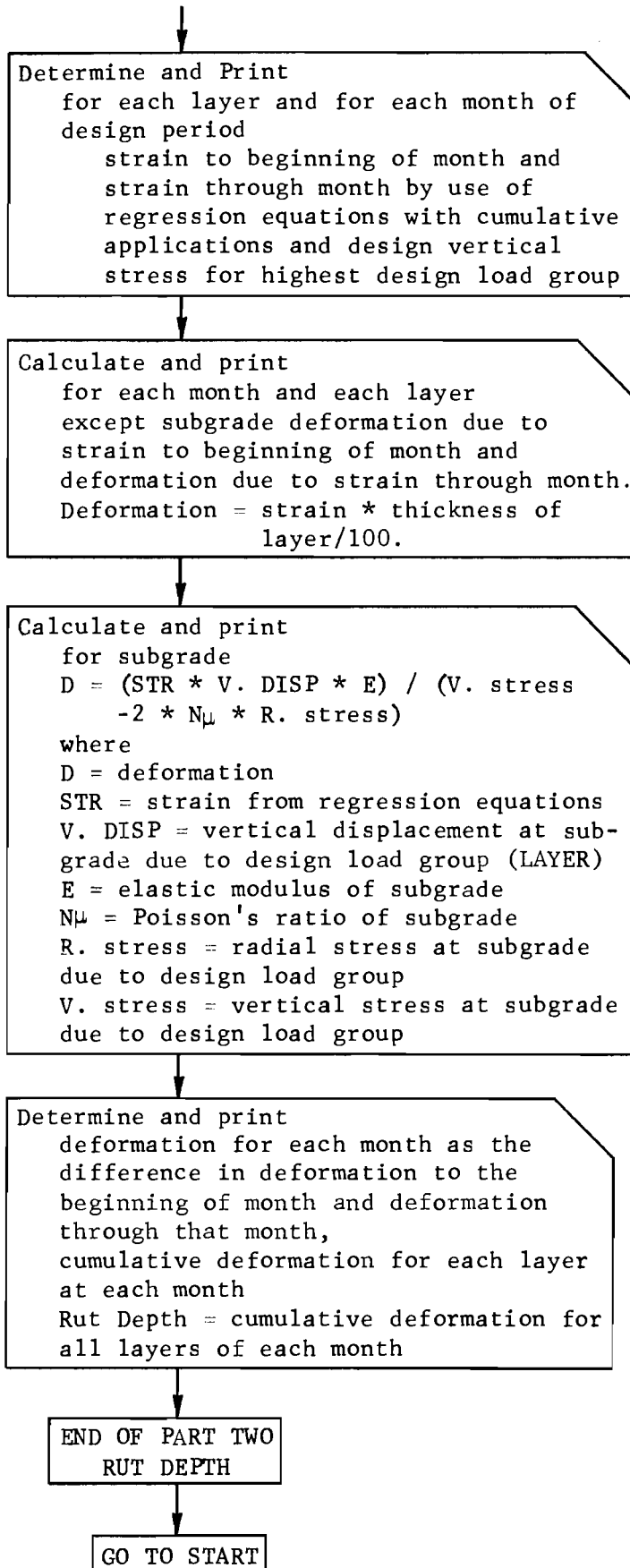


Fig A4.1. (Continued)

(Continued)

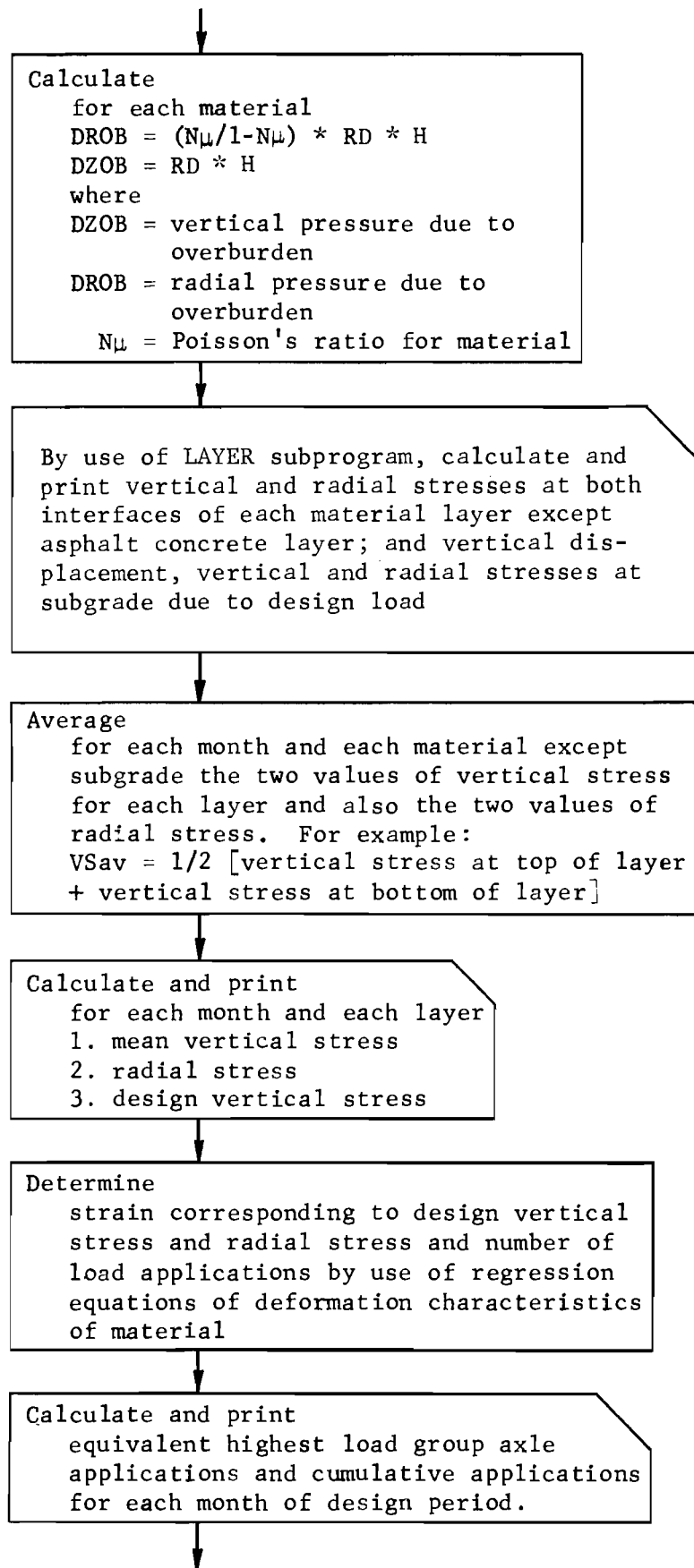


Fig A4.1. (Continued)

APPENDIX 4.2

LISTING OF COMPUTER PROGRAM FOR CRACKING INDEX AND RUT DEPTH INDEX





```

IF (TEST .EQ. 2.0) GO TO 70
DO 80 I = 1, 1M
  EA(M, I) = E(M, I)-ALPHA(CONF(M))*SIG(M)/100.*E(M, I)
EA ELASTIC MODULUS AT CONFIDENCE LEVEL
CONTINUE

PRINT MATERIAL PARAMETERS
PRINT 1060, (MAT(I), I = 1, NM)
  NS = NM-1
PRINT 1070, (TMIN(I), I = 1, NS)
PRINT 1080, (NU(I), I = 1, NM)
IF (TEST .NE. 2.0) PRINT 1160, (SIG(J), J = 1, NM)

IF (Q2 .EQ. NO) GO TO 100
PRINT 1090, MONTH
IF (TEST .EQ. 2.0) GO TO 90
DO 80 I = 1, NM
  PRINT 1100, MAT(I), CONF(I), (EA(I, J), J = 1, 12)
  PRINT 1110, (E(I, J), J = 1, 12)
  GO TO 110
90 PRINT 1120, (MAT(I), (E(I, J), J = 1, 12), I = 1, NM)
  GO TO 110
100 PRINT 1130, (E(J, I), J = 1, NM)
  IF (TEST .EQ. 2.0) GO TO 120
  PRINT 1140, (EA(J, I), J = 1, NM)
  PRINT 1150, (CONF(J), J = 1, NM)
110 CONTINUE
120 CONTINUE
PRINT 990, TITLE

READ AND PRINT FATIGUE CURVE DATA
READ (5,1010) A1, B, SDLOG, LN1
A1 CONSTANT OF FATIGUE CURVE
B SLOPE OF FATIGUE CURVE
SDLOG STANDARD DEVIATION OF FATIGUE LIFE (LOG)
LN1 CONFIDENCE LEVEL FOR THEORETICAL N

TEST = 2 SKIP CALCULATIONS IN PART ONE
IF (TEST .EQ. 2.) GO TO 150
PRINT 1170, A1, B, SDLOG, LN1

CALCULATE TANGENTIAL STRAIN FOR EACH LOAD GROUP AND
LAYER THICKNESS(CONF. AND MEAN VALUE)

IF (Q2 .NE. NO) GO TO 130
PRINT 1190
CALL LAYER (IM, I)
CONTINUE

IY = Y
IR = 0
IF (Q0-NO) 260,160,260
READ TRAFFIC PERCENTAGES FOR EACH MONTH
*** ONLY *** IF ACTUAL TRAFFIC DATA IS NOT GIVEN

```

```

CRK 116
CRK 117
CRK 118
CRK 119
CRK 120
CRK 121
CRK 122
CRK 123
CRK 124
CRK 125
CRK 126
CRK 127
CRK 128
CRK 129
CRK 130
CRK 131
CRK 132
CRK 133
CRK 134
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CRK 161
CRK 162
CRK 163
CRK 164
CRK 165
CRK 166
CRK 167
CRK 168
CRK 169
CRK 170
CRK 171
CRK 172
CRK 173

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IF TRAFFIC PERCENTAGES ARE CONSTANT FOR EACH DESIGN YEAR.
THESE VALUES MAY BE OMITTED IF Q1 = NO
CONTINUE
IF (Q1-NO) READ (5,1200) TRAF
TRAF TRAFFIC PERCENTAGE OF MONTH
SUM = 0.0
DO 170 I = 1, 12
  SUM = SUM+TRAF(I)
PRINT 1210, (MONTH(I), TRAF(I), I = 1, 12), SUM

CALCULATE YEARLY TRAFFIC GROWTH
DO 180 J = 1, IY
  TR(J) = (1.+GR)**(J-1)
TR TRAFFIC DUE TO GROWTH FOR YEAR

PRINT TRAFFIC DATA CALCULATED FROM TRAFFIC PERCENTAGES
AND TRAFFIC GROWTH RATE AND AXLE APPLICATIONS
IF (Q1-NO) 200,190,200
190 PRINT 1220
  GO TO 210
200 PRINT 1230, MONTH
210 DO 250 J = 1, NL
  IF (Q1-NO) PRINT 1240, L(J)
  DO 250 K = 1, IY
  GO 220 K = 1, 12
  X(K) = A(J)*TR(M)*TRAF(K)
X MONTHLY TRAFFIC CALCULATED FROM PERCENTAGES,
GROWTH RATE, AND LOAD
IR = IR+1
IR INDEX FOR IOBIN WRITE OF TRAFFIC FOR YEAR
CALL IOBIN (6MWRITER, 3, X(1), 12, IAC(IR))
IF (Q1-NO) 230,230,240
230 PRINT 1250, L(J), M, X(1)
  GO TO 250
240 PRINT 1260, M, (X(K), K = 1, 12)
  CONTINUE
250 GO TO 271
260 Q1 = 2M
PRINT 1230, MONTH
DO 270 J = 1, NL
  PRINT 1235, L(J)
  DO 270 K = 1, IY

IF Q0 IS NOT NO, READ ACTUAL TRAFFIC DATA
GIVE MONTHLY VALUES FOR EACH YEAR FOR FIRST LOAD
GROUP, THEN REPEAT SET FOR ADDITIONAL LOAD GROUPS

READ (5,1270) YR, (X(I), I = 1, 12)
X MONTHLY TRAFFIC VALUE
YR IDENTIFICATION VARIABLE
PRINT 1280, YR, (X(I), I = 1, 12)
IR = IR+1
CALL IOBIN (6MWRITER, 3, X(1), 12, IAC(IR))
CONTINUE
270 CONTINUE
271 CONTINUE

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CRK 230
CRK 231

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C      IF (TEST .EC. 2.0) GO TO 500
C      CALCULATE THEORETICAL REPETITIONS BY STRAINS FROM
C      LAYER SUBROUTINE
C      RK = ALPHA(LN1)
C      IT = 0
C      RK      K VALUE CORRESPONDING TO LN1
C      IT      INDEX FOR IOBIN WRITE OF THEORETICAL TRAFFIC
C
C      IW1 = IW*1
C      DO 400 I = 2, IW1/2
C      I2 = (I+1)/2
C      IF (IM-1) GO TO 280
C      PRINT 1290, LN1
C      GO TO 290
C      CONTINUE
C      PRINT 1300, L(I2), MONTH
C      CALL IOBIN (7*HEADSKP, 2, ANS(I), IM, IDX(I))
C      IW      INDEX FOR IOBIN READ OF STRAINS ANS
C      ANS      TANGENTIAL STRAIN AT BOTTOM OF ASPHALT
C      CONCRETE CALCULATED IN SUBROUTINE LAYER
C      USING CONFIDENCE LEVEL OF ELASTIC MODULUS
C
C      DO 300 J = 1, IM
C      THN(J) = 10.**((ALOG10(A1*ANS(J)**(-B))-RK)*SDLDG)
C      THN      THEORETICAL NUMBER OF REPETITIONS FROM
C      INPUT FATIGUE EQUATION AT SOME SPECIFIED
C      CONFIDENCE LEVEL
C      IF (IM-1) 320+320+310
C      PRINT 1310, LN1, (THN(J), J = 1, 12)
C      GO TO 340
C
C      IF MODULUS IS CONSTANT, THN IS CONSTANT
C      SAV = THN(1)
C      DO 330 J = 2, 12
C      THN(J) = SAV
C
C      IT      INDEX FOR IOBIN WRITE OF THN
C      IT = IT+1
C      CALL IOBIN (6*HWRITER, 4, THN(I), 12, ITH(IT))
C      I1 = I-1
C      I = 1 IS USED BECAUSE IN THE LAYER SUBPROGRAM, THE
C      STRAINS ARE WRITTEN ON TAPE IN ALTERNATING ORDER --
C      MEAN LEVEL AND CONFIDENCE LEVEL
C      CALL IOBIN (7*HEADSKP, 2, ANS(I), IM, IDX(I1))
C      ANS      TANGENTIAL STRAIN AT BOTTOM OF ASPHALT
C      CONCRETE CALCULATED IN SUBROUTINE LAYER
C      USING MEAN VALUE OF ELASTIC MODULUS
C
C      DO 350 J = 1, IM
C      THN(J) = A1*ANS(J)**(-B)
C      THN      THEORETICAL NUMBER OF REPETITIONS FROM
C      INPUT FATIGUE EQUATION AT SOME SPECIFIED
C      MEAN LEVEL
C      IF (IM-1) 370+370+360
C      PRINT 1220, (THN(J), J = 1, 12)
C      GO TO 390
C      DO 380 J = 2, 12

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360      THN(J) = THN(I)
390      PRINT 1330, L(I2), SAV, THN(I)
      IT = IT+1
400      CALL IOBIN (6*HWRITER, 4, THN(I), 12, ITH(IT))
      CONTINUE
C
C      LSN = LSN+LN1
C      PK = ALPHA(LSN)
C      LSN      SUM OF CONFIDENCE LEVELS OF ELASTIC MODULUS
C      AND FATIGUE EQUATION
C      PK      K VALUE CORRESPONDING TO LSN
C      PRINT 990, TITLE
C      NLN = 0
C      NN = 0
C      MM = 0
C
C      CALCULATE RATIO OF N ACTUAL TO N THEORETICAL
C      DO 430 I = 1, NL
C      PRINT 1340, L(I), MONTH
C      IR = 0
C      NA = NA+1
C      CALL IOBIN (7*HEADSKP, 4, L(I), 12, ITH(NN))
C      NN = NN+1
C      CALL IOBIN (7*HEADSKP, 4, THN(I), 12, ITH(NN))
C      DO 430 IO = 1, IV
C      MM = MM+1
C      CALL IOBIN (7*HEADSKP, 3, X(I), 12, IAC(MM))
C      DO 410 II = 1, 12
C      EN(1, II) = X(II)/L(II)
C      EN(2, II) = X(II)/THN(II)
C      EN(1)      RATIO OF MONTHLY TRAFFIC TO MEAN REPETITIONS
C      EN(2)      TO CONF. REPETITIONS
C
C      IR = IR+1
C      ENAT(IR) = ENNT(IR)*EN(1, I)
C      IR = IR+1
C      ENAT(IR) = ENNT(IR)*EN(2, I)
C      ENNT      CUMULATIVE SUM OF EN
C      PRINT 1360, MO, LSN, ((EN(J), I) * I = 1, 12), J1 = 1, 2)
C      NLN      PAGE SKIP CONTROL
C      NLN = NLN+4
C      IF (NLN-44) 430,420,420
C      NLN = 0
C      PRINT 990, TITLE
C      PRINT 1340, L(I), MONTH
C      CONTINUE
C      DO 440 I = 3, IR, 2
C      ENNT(I) = ENAT(I)+ENNT(I-2)
C      ENNT(I+1) = ENNT(I+1)+ENNT(I-1)
C      PRINT 1370
C      NLN = 1
C      DO 490 J = 1, IR, 2
C      IF (NLN-44) 460,450,450
C      NLN = NLN-44
C      PRINT 990, TITLE
C      PRINT 1370
C      NLN = NLN+1
C      J2 = (J+1)/2

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DO 470 JJ = 1, 2
  L1(JJ) = ENNT(J,JJ-1)
  IF (L1(JJ) = 470, 480, 470)
    L2(JJ) = ALOG10(L1(JJ))
    SDL = (L2(1) - L2(2)) / PK
    AK = -L2(2) / SDL
    AA = XNORM(AK, 0, 1)
    CI = AA * 10 ** 3
  PRINT 1380, J2, LSM, L1(1), L2(1), SDL, AK, AA, CI, L1(2),
  1 L2(2)
  GO TO 490
480 ZERO = 0.0
  PRINT 1390, J2, LSM, (ZERO, III = 1, 4)
490 CONTINUE
500 CONTINUE
  IF (TEST .EQ. 1.0) GO TO 970

PART TWO

  IZ NUMBER OF DEPTHS FOR CALCULATIONS
  IZ = NM - 1
  IF (IM .EQ. 1) PRINT 1400
  CALL LAYER (IM, IZ)

CALCULATIONS OF CONFINING AND VERTICAL PRESSURES
DUE TO OVERBURDEN

R UNIT WEIGHT OF MATERIAL (LB/ CU IN)
DZOB VERTICAL PRESSURE DUE TO OVERBURDEN
DROB RADIAL PRESSURE DUE TO OVERBURDEN
DMC INPUT RADIAL PRESSURE FOR WHICH CURVES ARE
GIVEN FOR MATERIALS
EM INPUT PARAMETER TO CALCULATE CORRECT RADIAL
PRESSURE FOR GIVEN CURVE. VALUE IS 0 WHEN
REGRESSION EQN IN N,Z1,Z3 AND E IS GIVEN

READ (5,1410) (R(1), EM(I), DRC(I), I = 1, NM)
  RD(1) = (R(1) + R(2) * 0.5) / 1.5
  RD(2) = (R(1) + R(2) + R(3) * 0.5) / 2.5
  RD(3) = (R(1) + R(2) + R(3)) / 3.0
  RD COMPOSITE WEIGHT
  H COMPOSITE THICKNESS
DO 510 I = 1, NM
  EM(I) = ABS(EM(I))
510 IF (DRC(I) .EQ. 0.0) DRC(I) = 1.0
  H(1) = TMIN(1) * TMIN(2) * 0.5
  H(2) = TMIN(1) * TMIN(2) + TMIN(3) * 0.5
  H(3) = TMIN(1) + TMIN(2) + TMIN(3)
DO 520 I = 1, NS

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520 K = 1.0
  DRDB(I) = NU(K) / (1.0 - NU(K)) * RD(I) * H(I)
  DRDB(I) = RD(I) * H(I)
DO 540 I = 1, NL
DO 540 K = 1, IM
DO 530 J = 1, NS
  J = J1 - 1
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BASE AND SUBBASE CALCULATIONS

  AN1(K, J, I) = AN1(K, J, I) - NZOB(J1)
  MEAN VERTICAL STRESS
  AN3(K, J, I) = AMIN1(-1.0, (AN3(K, J, I) - DROB(J1)))
  RADIAL STRESS
  SI = (ABS(AN3(K, J, I)) ** EM(J1))
  AN2(K, J, I) = AN1(K, J, I) * (DRC(J1)) / SI
  AN2(K, J, I) = SIGN(AN2(K, J, I), AN1(K, J, I))
  DESIGN VERTICAL STRESS

SUBGRADE CALCULATIONS
  AN1(K, NS, I) = AN1(K, NS, I) * 2.0 - DZOB(NS)
  MEAN VERTICAL STRESS
  AN3(K, NS, I) = AMIN1(-1.0, (AN3(K, NS, I) * 2.0 - DROB(NS)))
  RADIAL STRESS
  AN2(K, NS, I) = AN1(K, NS, I) * DRC(NM) - AN3(K, NS, I)
  DESIGN VERTICAL STRESS

IM = 1 ELASTIC MODULUS VALUE CONSTANT MONTHLY
IF (IM = 1) 550, 550, 580
DO 570 J = 1, 12
DO 560 I = 1, NS
  E(I, M) = E(I, 1)
DO 560 I = 1, NL
  AN1(M, J, I) = AN1(I, J, I)
  AN2(M, J, I) = AN2(I, J, I)
  AN3(M, J, I) = AN3(I, J, I)
  E(NM, M) = E(NM, 1)
DO 570 IJ = 1, 3
  DSP(M, IJ) = DSP(1, IJ)
570 CONTINUE
580 CONTINUE

PRINT DESIGN CONFINING PRESSURE AND VERTICAL PRESSURE

  NLN = 0
  NLN NUMBER OF LINES PRINTED -- PAGE SKIP CONTROL
DO 660 II = 1, NL
  CRK 456
  CRK 457
  CRK 458
  CRK 459
  CRK 460
  CRK 461
  CRK 462
  CRK 463

  ANI (MONTH, MATERIAL, LOAD GROUP)
  I = 1 VERTICAL STRESS MEAN
  I = 2 VERTICAL STRESS DESIGN
  I = 3 RADIAL STRESS
  IF (NLN = 45) 600, 590, 590
590 PRINT 990, TITLE
  NLN = 0

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600      NLN = NLN*NS*3
        NS1 = NS - 1
        PRINT 1420, MONTH(1), MONTH(2), L(1), (MAT(K*1),
1      (AN1(I, K, II), AN2(I, K, II), AN3(I, K, II)), I = 1,
2      2), K = 1, NS1)
        PRINT 1421, MAT(NM), DRC(NM), (AN1(I, NS, II), AN2(I, NS, II),
1      AN3(I, NS, II)), I = 1, 2)

        MONTHS 3 THROUGH 11 IN GROUPS OF THREE
        DO 630 JJ = 1, 3
          J3 = JJ*3
          J5 = J3+2
610      IF (NLN=45) 620,610,610
          PRINT 990, TITLE
620      NLN = 0
630      NLN = NLN*NS*3
          PRINT 1430, (MONTH(J), J = J3, J5), ((AN1(J, K, II), AN2(J,
1      K, II), AN3(J, K, II)), J = J3, J5), K = 1, NS)

        MONTH 12
        IF (NLN=45) 650,640,640
640      PRINT 990, TITLE
650      NLN = 0
650      NLN = NLN*NS*3
          PRINT 1440, MONTH(12), ((AN1(12, K, II), AN2(12, K, II), AN3(12,
1      K, II)), K = 1, NS)
660      CONTINUE

        DETERMINE STRAIN CORRESPONDING TO DESIGN VERTICAL
        STRESS ACCORDING TO INPUT FATIGUE CURVES

        DO 960 II = 1, IY
        DO 670 JJ = 1, NL
          J = (JJ-1)*IY+II
        CALL IOBIN (7*HEADSKP, 3, X(1), 12, IAC(J))
        DO 670 K = 1, 12
          XX(JJ, K) = X(K)
          XX(LOAD GROUP, MONTH), TRAFFIC

        LOOP WHICH INCLUDES SIX MONTHS FOR PRINTOUT PURPOSES ONLY
        DO 960 JJ = 1, 12, 6
        DO 680 I = 1, 12
        DO 680 J = 1, 5
680      AN1(13, J, I) = 0.0
          J = JJ*5

        AN1 (MONTH, MATERIAL, LOAD GROUP) IS NOW THE STRAIN
        APPROPRIATE TO THE CORRESPONDING DESIGN VERTICAL STRESS
        AN1(13, MATERIAL, MONTH) IS THE CUMULATIVE STRAIN
        PRINT 1450, II, (MONTH(1), I = JJ, J)
        DO 790 I = JJ, J
        IF (XX(1, I)) 690,690,720

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690      IF (I-1) 790,790,700
700      DO 710 K = 1, NS
          EQ(K, I) = 0.0
          IF (I, EQ, 1) CM(K, I) = CM(K, I-1)
          IF (I, EQ, 1) CM(K, I) = CM(K, I-1)
          CONTINUE
710      GO TO 790
720      CONTINUE
          DO 780 K = 1, NS
          DO 750 J = 1, NL
            AN1(I, K, M) = POLY(XX(M, I), AN2(I, K, M), K, AN3(I,
1      K, M))
          IF (AN1(I, K, N18)) 730,750,730
730      IF (XX(N18, I)) 740,750,740
740      AN1(13, K, I) = AN1(13, K, I)+10.0*(AN1(I, K, M)
1      *ALCG10(XX(N18, I))/AN1(I, K, N18))
750      CONTINUE
          IF (AN1(13, K, I)) EQ(K, I) = AN1(13, K, I)
          EQ EQUIVALENT 18-KIP APPLICATIONS
          IF (I-1) 760,760,770
          CM CUMULATIVE EQUIVALENT 18-KIP APPLICATIONS
760      CM(K, I) = EQ(K, I)+CM(K, I-1)
          GO TO 780
770      CM(K, I) = EQ(K, I)+CM(K, I-1)
780      CONTINUE
790      CONTINUE

          PRINT EQUIV 18-KIP APPLICATIONS AND CUMULATIVE APPLICATIONS
          PRINT 1460, (MAT(K*1), (EQ(K, I), I = JJ, J), K = 1, NS)
          PRINT 1470
          PRINT 1460, (MAT(K*1), (CM(K, I), I = JJ, J), K = 1, NS)

          DETERMINE STRAIN CORRESPONDING TO CUMULATIVE 18-KIP
          AXLE APPLICATIONS AND DESIGN VERTICAL STRESS

          PRINT 1480
          DO 920 K = 1, NS
          DO 910 I = JJ, J
          IF (I-1) 800,800,850
          CONTINUE
          IF (I-1) 810,810,820
          STRAIN(I, K, I) = 0.0
          GO TO 880
          IF (CM(K, I-1)) 830,830,840
          STRAIN(I, K, I) = 0.0
          GO TO 880
          STRAIN(I, K, I) = POLY(CM(K, I-1), AN2(I, K, N18),
1      K, AN3(I, K, N18))-(ABS(AN2(I, K, N18))-2.0
2      *NU(K*1)*AN3(I, K, N18))/E(K*1, I)*100.
          GO TO 880
          IF (CM(K, I-1)) 860,860,870
          STRAIN(I, K, I) = 0.0
          GO TO 880
          STRAIN(I, K, I) = POLY(CM(K, I-1), AN2(I, K, N18),
1      K, AN3(I, K, N18))-(ABS(AN2(I, K, N18))-2.0
2      *NU(K*1)*AN3(I, K, N18))/E(K*1, I)*100.
          CONTINUE

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890 IF (CM(K, I)) 890+890+900
      STRAIN(I, K, 2) = 0.0
900 GO TO 910
      CONTINUE
      STRAIN(I, K, 2) = POLY(CM(K, I), AN2(I, K, N18),
1      K, AN3(I, K, N18))-(ABS(AN2(I, K, N18))^2.0
2      *NU(K, I)*AN3(I, K, N18))/E(K, I, I)*100.
910 CONTINUE
      PRINT 1490, MAT(K+1), (STRAIN(M, K, I), M = JJ, J)
920 CONTINUE
      PRINT 1500
      PRINT 1490, (MAT(K+1), (STRAIN(M, K, 2), M = JJ, J), K = 1, NS)

      PRINT DEPTH OF LAYERS
      PRINT 1510, ((MAT(I), TMIN(I)), I = 2, NS), MAT(NM)

      PRINT 1520, ((DSP(I, K), I = JJ, J), K = 1, 3)
      CALCULATE DEFORMATION
      DO 925 I = 1, 12
925 DEFTOT(I) = 0.0
      DO 950 NN = 2, NM
        I = NN-1

      DEF DEFORMATION AT MONTH N DUE TO REPETITIONS THROUGH
          MONTH N
      DEFN DEFORMATIONS AT MONTH N DUE TO REPITIONS THROUGH
          MONTH (N-1)
      DEFD DIFFERENCE BETWEEN DEFORMATION DEF-DEFN
      DEFC CUMMULATIVE DEFORMATION THROUGH MONTH N
      DEFTOT TOTAL CUMULATIVE DEFORMATION

      DO 950 K = JJ, J
      IF (NM .EQ. 2) DEFTOT(K) = 0.0

      TMIN(NM) THEORETICAL THICKNESS OF SUBGRADE BASED ON
          VERTICAL DISPLACEMENT, POISSONS RATIO,
          ELASTIC MODULUS, VERTICAL AND RADIAL STRESS ...
      IF (I .EQ. NS) TMIN(NM) = ABS(DSP(K, I)*E(NM, K)/(DSP(K,
          2)-2.0*NU(NM)*DSP(K, 3)))
      DSP(MONTH, I) VALUE AT SUBGRADE DUE TO 18-KIP LOAD
          I = 1 VERTICAL DISPLACEMENT
          I = 2 VERTICAL STRESS
          I = 3 RADIAL STRESS

      DEF(K, I) = STRAIN(K, I, 2)*TMIN(NM)/100.
      DEFN(K, I) = STRAIN(K, I, 1)*TMIN(NM)/100.
930 IF (K-1) 940,940,930
      CONTINUE
      DEFD(K, I) = DEF(K, I)-DEFN(K, I)
      DEFC(K, I) = DEFC(K-1, I)+DEFD(K, I)
      DEFTOT(K) = DEFTOT(K)+DEFC(K, I)

      GO TO 950
940 CONTINUE
      DEFD(I, I) = DEF(I, I)-DEFN(I, I)
      DEFC(K, I) = DEFC(I, I)+DEFD(K, I)
      DEFTOT(I) = DEFTOT(I)+DEFC(K, I)

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CRK 580
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CRK 636
CRK 637

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950 CONTINUE
      PRINT DEFORMATION AT MONTH N
      PRINT 1530
      PRINT 1490, (MAT(I+1), (DEF(K, I), K = JJ, J), I = I, NS)
      PRINT 1540
      PRINT 1490, (MAT(I+1), (DEFN(K, I), K = JJ, J), I = I, NS)
      PRINT 1550
      PRINT 1490, (MAT(I+1), (DEFD(K, I), K = JJ, J), I = I, NS)
      PRINT 1560
      PRINT 1490, (MAT(I+1), (DEFC(K, I), K = JJ, J), I = I, NS)
      PRINT 1570, (DEFTOT(K), K = JJ, J)
960 CONTINUE
      GO TO 10
970 CONTINUE

98) FCRMAT (BA10)
99) FCRMAT (*1* 15X, BA10)
100) FCRMAT (2I10, 4F10.0, 3(A2, 3X) ,F5.0)
101) FCRMAT (4F10.0)
102) FCRMAT (3(/), 10X *AXLE LOAD AXLE LOAD TIRE N *
      *INITIAL *
      / 10X *RANGE+KIPS MEAN+KIPS PRESSURE AXL*
      *E APPL * /
      ( 9X, F4.0, * -, F4.0, 5X, F7.2, 6X, F6.2, 6X, F7.0))
103) FCRMAT (3(/), 15X*LOAD DISTRIBUTION FACTOR, RATIO* 10X, F8.2
      / 15X*LANE DISTRIBUTION FACTOR, RATIO* 10X, F8.2
      / 15X*DESIGN PERIOD, YEARS * 10X, F8.0)
1035 FCRMAT (15X*TRAFFIC GROWTH RATE, RATIO * 10X, F8.2)
104) FCRMAT (A10, F10.0, 2F5.0, 2F10.0)
105) FCRMAT (A10, F10.0, 2F5.0, 5F10.0 / (8F10.0))
106) FCRMAT (3(/), 30X, *MATERIAL PARAMETERS, // 10X*MATERIAL,
      11X, 5( A10, 1X))
107) FCRMAT (10X*THICKNESS* 5X, 5F11.0)
108) FCRMAT (10X*POISSONS RATIO* 5F11.0)
109) FCRMAT (6(/), 10X *VARIATIONS OF E VALUES IN SPACE AND TIME*
      / 10X *MATERIAL CONF * 6A10 / 22X, 6A10/)
      ( 9X, A10, F5.2, F8.0, 5F10.0 / 19X, 6F10.0)
      ( 20X, *MEAN* F8.0, 5F10.0 / 19X, 6F10.0)
      ( 5X, A10, *MEAN * F8.0, 5F10.0 / 22X, 6F10.0)
      ( / , 10X*E - MEAN VALUE* 5F11.0)
      ( 10X *E - CONF VALUE* 5F11.0)
      ( 10X *CONFIDENCE LEVEL* 5(F9.2+2X))
      ( 10X *CCF VAR, PERCENT * 5(F9.2+2X))
      (3(/), 20X, 3M, = Ee.1, 11M * (17E) ** F5.2
      / 30X *LOG STANDARD DEVIATION OF* F5.2
      / 30X *ANO CONFIDENCE LEVEL * F5.2, 4(/))
119) FCRMAT (10X* LOAD DEPTH - INCHES TANGENTIAL STRAIN*/)
120) FCRMAT (12F5.0)
121) FCRMAT (3(/), 23X *MONTH* 12X *TRAFFIC PERCENTAGE* / 12
      ( / 20X, A10, 15X, F10.2), // 25X *SUM* 17X, F10.2)
      (4(/), 50X *N TABLE = ACTUAL* // 10X *LOAD YEAR *
      * N = MONTHLY * /)
      (4(/), 50X *N TABLE = ACTUAL* / 46X *FROM TRAFFIC *
      *CATA INPUT* // 15X, 6A10/20X, 6A10)
      ( 20X *LOAD* F5.0)
      ( / , 10X, F3.0)
125) FCRMAT (10X* F3.0* 5X, I3* 6X, F10.2)

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CRK 694
CRK 695

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1260 FORMAT ( 18X, I2, 4X, 6F10.2 / 28X, 6F10.2)
1270 FORMAT ( A8, 12F6.0 )
1280 FORMAT ( 10X, A8, F7.0, 5F10.0 / 28X, 6F10.0 )
1290 FORMAT ( 4(/), 50X *N TABLE - THEORETICAL* // 10X *LOAD* 5X
1 *CONFIDENCE* 5X *MEAN VALUE* / 19X *LEVEL* F6.3/)
1300 FORMAT (4(/), 50X *N TABLE - THEORETICAL* //15X *LOAD*F5.0/10X
1 *CONF * 4X, 6A10 / 10X *LEVEL* 7X, 6A10)
1310 FORMAT ( 10X, F5.3, 2X, 6F10.0 / 28X, 6F10.0 )
1320 FORMAT ( 10X*MEAN* 3X, 6F10.0, 20X, 6F10.0)
1330 FORMAT ( 10X, F3.0, 3X, F10.2, 5X, F10.2)
1340 FORMAT ( 4(/), 50X *N ACTUAL / N THEORETICAL* //10X*LOAD*F5.0/
1 10X *YEAR CONF* 6A10 / 22X, 6A10 )
1360 FORMAT (10X, I2, F6.3, 2X, 6E10.3, 24X, 6E10.3/14X*MEAN*3X
1 6E10.3 / 24X, 6E10.3 )
1370 FORMAT ( 4(/), 10X *MONTH CONF (N/NIT LOG(N/NIT LOG SD*
1 6X *K* 7X *A* 7X *C)* / )
1380 FORMAT ( 10X, I3, F8.3, 3E11.3, 3E9.2 / 17X *MEAN*2E11.3)
1390 FORMAT (10X, I3, F8.4, F6.0, 2(10X*--*) , AX, *--*2F9.0/17X*MEAN*F6.0,
1 10X*--*)
1400 FORMAT ( 4(/), 15X *LOAD* 5X *MATERIAL* 8X *VERTICAL STRESS*
1 7X *TANGENTIAL STRESS* / 40X, 2(*TOP* 7X *BOTTOM*7X))
1410 FORMAT ( 2 (3F10.0) )
1420 FORMAT (1F1.3(/), 23X, *RADIAL* 4X, 2(*.....*A10*.....*2X)/
1 7X, *LOAD MATERIAL PRESSURE*2X, 2(*VERTICAL *
2 *STRESS RADIAL*2X) / 23X *(INPUT)* 3X, 2(*MEAN* 5X
3 *DESIGN STRESS*2X) // F10.0, 4(/, 11X, A10,
4 11X, 3F8.3, 1X, 3F8.3 ) )
1421 FORMAT (11X, A10, F8.3, 3X, 3F8.3, 1X, 3F8.3 / )
1430 FORMAT ( 4(/), 9X, 3(*.....*A10*.....*2X) // 9X, 3(*VER*
1 *TICAL STRESS RADIAL*2X) // 9X, 3(*MEAN* 5X *DESIGN*
2 * STRESS* 2X) // 4( 8X, 3F8.3, 1X, 3F8.3, 1X, 3F8.3 / ) )
1440 FORMAT (4(/), 9X, *.....*A10*.....* / 9X *VERTICAL STRESS*
1 * RADIAL* / 9X *MEAN* 5X *DESIGN STRESS* //
2 4(8X, 3F8.3 / ) )
1450 FORMAT (1F1.3(/), 5X, *YEAR*I3, /15X, 6(A10, 1X) /, 5X *MI-LD-REPT*)
1460 FORMAT ( 5X, A10, 6E11.3)
1470 FORMAT ( 5X *CUMULATIVE* )
1480 FORMAT ( 5X, *STRAIN TO BEGINNING OF MONTH * )
1490 FORMAT ( 5X, A10, 6E11.3 )
1500 FORMAT ( 5X, *STRAIN THROUGH MONTH* )
1510 FORMAT (5X, *DEPTH* 5 ( / 5X, A10, F11.1 ) )
1520 FORMAT (5X *VERT. DSP.* 6E11.3 / 5X *V STRESS * 6E11.3 /
1 5X *R STRESS * 6E11.3)
1530 FORMAT ( 5X, *DEFORMATION AT MONTH * )
1540 FORMAT (5X, *DEFORMATION DUE TO REPTITIONS THROUGH MONTH (N-1)* )
1550 FORMAT ( 5X, *DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1* )
1 1)
1560 FORMAT ( 5X, *CUMULATIVE DEFORMATION THROUGH MONTH * )
1570 FORMAT ( 5X, *TOTAL CUMULATIVE DEFORMATION RUT DEPTH* /
1 15X, 6E11.3)
END

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CRK 696
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CRK 746

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SUBROUTINE LAYER (IK, IZ)
COMMON /LAYER/ EE(5, 12), V(5), NS, TMIN(5), IM, NL, A(20),
1 TIRE(20), EA(5, 12), DRCS(5), DZOR(5), RD(5), EM(5),
2 DR(5)
COMMON /PRIN/ AN1(13, 5, 20), AN2(13, 5, 20), AN3(13, 5, 20),
1 DSP(12, 3), N18, LOAD(20), MAT(5), MONTH(12)
DIMENSION Z2(11), E(5), M(4), HM(4), AZ(400), AJ(400)
REAL LOAD
C LAYER DOES CALCULATIONS FOR EACH DEPTH, MEAN AND
C CONFIDENCE VALUES OF ELASTIC MODULUS, EITHER ONE
C OR TWELVE MONTHS, AND FINALLY FOR EACH LOAD GROUP
N = NS-1
C ** START ON A NEW R **
R = 0.0
ITN = 46
ITN4 = ITN*4
DO 220 M = 1, NL
IF (IZ-1) 5, 1, 5
1 IF (IK-1) 2, 2, 3
2 PRINT 240, LOAD(M)
GO TO 5
3 PRINT 230, LOAD(M), TMIN(I), MONTH
5 MT = M
AR = A(M)
ARF = AR*TIRE(M)
ZZ(I) = TMIN(I)
C ** ADJUST LAYER DEPTHS **
HM(I) = TMIN(I)
H(I) = HM(I)
CO 10 I = 2, N
HM(I) = TMIN(I)
H(I) = H(I-1)+HM(I)
DO 20 I = 1, IZ
20 ZZ(I) = H(I)
DO 220 IE = 1, 2
DO 210 IM = 1, IK
MM = 0
MMT = MH
IF (IE-1) GO TO 40
DO 30 I = 1, NS
30 E(I) = EE(I, MM)
GO TO 70
IF (IZ-1) 50, 50, 220
DO 90 I = 1, NS
60 E(I) = EA(I, MM)
CONTINUE
CO 90 I = 1, IZ
DO 90 J = 1, N
YZ = ABS(H(I)-ZZ(I))
IF (IZ-.0001) 80, 80, 90
ZZ(I) = -H(I)
80 CONTINUE
90 IF ((MM.EQ. 0) .AND. (IZ.GT. 1)) ZZ(I) = -ZZ(I)
C ** CALCULATE THE PARTITION **
CALL PART INTERST, AZ, ITN, AR)
C ** CALCULATE THE COEFFICIENTS **
CO 100 I = 1, ITN4
IT = I

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CRK 747
CRK 748
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CRK 799
CRK 800
CRK 801
CRK 802
CRK 803
CRK 804

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      P = AZ(I)
      CALL COE (IT, N, E, V, H, NS, P)
      PA = P*AR
      CALL BESSEL (I, PA, Y)
      AJ(I) = Y
100   CONTINUE
      NX = 0
      IZT = 0
C ** START ON A NEW Z **
110   IZT = IZT+1
      IF (IZT-IZ) 120,120,200
120   Z = ABS(ZZ(IZT))
C ** FIND THE LAYER CONTAINING Z **
      TZZ = 0.0
      DO 130 J = 1, N
      J = NS-J
      IF (Z-H(J)) 130,140,140
130   CONTINUE
      L = 1
      GO TO 160
140   L = J+1
      IF (ZZ(IZT)) 150,160,160
150   L = J
      TZZ = 1.0
160   CONTINUE
      CALL CALCIN (V, E, NTEST, ARP, ITN, AZ, Z, R, AJ, IK, IZ,
1      IM, MM, L, AR, TZZ, NX, M, ITZ, MN, IE)
      IF (IZ-1) 190,190,170
170   CONTINUE
      IF (TZZ) 190,190,180
180   ZZ(IZT) = -ZZ(IZT)
      IF (IZ, EQ, IZT) NX = 1
C      NA = 1 SUBGRADE CALCULATIONS
      IZT = IZT-1
190   CONTINUE
      GO TO 110
200   CONTINUE
210   CONTINUE
220   CONTINUE
      RETURN
230   FORMAT (4//, 50X *TANGENTIAL STRAIN*/ 15X *LOAD*
1      F5.0, 3X, *DEPTH* F5.0/ 10X, 6A10/ 15X, 6A10)
240   FORMAT (10X, F5.0 )
      END

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CRK 805
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CRK 843
CRK 844
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CRK 846
CRK 847
CRK 848

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      SUBROUTINE CALCIN (V, E, NTEST, ARP, ITN, AZ, Z, R, AJ, IK,
1      IZ, IM, MM, L, AR, TZZ, NX, M, ITZ, MN, IE)
CCALCIN ****SUBROUTINE CALCIN - N-LAYER ELASTIC SYSTEM ****
      COMMON /COECAL/ A(400, 5), B(400, 5), C(400, 5), D(400, 5)
      DIMENSION E(5), V(5), AZ(400), AJ(400), TEST(11), W(4), ANS(12),
1      XANS(12*11)
      COMMON /CAL/ IW, IDX(20)
      COMMON /PRIN/ AN1(13, 5, 20), AN2(13, 5, 20), AN3(13, 5, 20),
1      OSP(12, 3), N18, LOAD(20), MAT(5), MONTH(12)
10   REAL LOAD
      W(1) = 0.34785485
      W(2) = 0.65214515
      W(3) = W(2)
      W(4) = W(1)
      VL = 2.0*V(L)
      EL = (1.0+V(L))/E(L)
      VLL = 1.0-VL
      CSZ = 0.0
      CST = 0.0
      CSF = 0.0
      CTF = 0.0
      COM = 0.0
      CWL = 0.0
      NTS1 = NTEST*1
      ITS = 1
      DO 80 I = 1, ITN
C      INITIALIZE THE SUB-INTEGRALS
      RSZ = 0.0
      RST = 0.0
      RSF = 0.0
      RTF = 0.0
      ROM = 0.0
      RML = 0.0
C      COMPLETE THE SUB-INTEGRALS
      K = 4*(I-1)
      DO 20 J = 1, 4
      J1 = K+J
      P = AZ(J1)
      EP = EXP(P*Z)
      T1 = B(J1, L)*EP
      T2 = D(J1, L)/EP
      T1F = T1-T2
      T1M = T1-T2
      T1 = (A(J1, L)+B(J1, L)*Z)*EP
      T2 = (C(J1, L)+D(J1, L)*Z)/EP
      T2F = P*(T1+T2)
      T2M = P*(T1-T2)
      WA = AJ(J1)*W(J)
C      SPECIAL ROUTINE FOR R = ZERO
      PP = P*P
      RSZ = RSZ+WA*PP*(VL)*T1P-T2M)
      ROM = ROM+WA*EL*P*(2.0*VL)*T1M-T2P)
      RST = RST+WA*PP*(VL*0.5)*T1P+0.5*T2M)
      RSF = RST
20   CONTINUE
C
      SF = (AZ(K+4)-AZ(K+1))/1.722*726
      CSZ = CSZ+RSZ*SF

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CRK 849
CRK 850
CRK 851
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CRK 905
CRK 906

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CST = CST+RSZ*SF
CSR = CSR+RSZ*CF
CTR = CTR+RTH*SF
COM = COM+ROM*SF
CML = CMU+RMU*SF
RSZ = 2.0*RSZ*AR*SF
TESTH = ABS(RSZ)-10.0**(-4)
30 IF (ITS-NTS1) 30,40,40
CONTINUE
TEST(ITS) = TESTH
ITS = ITS+1
GO TO 80
40 CONTINUE
TEST(NTS1) = TESTH
DO 70 J = 1, NTEST
IF (TESTH-TEST(J)) 50,60,60
50 CONTINUE
TESTH = TEST(J)
60 CONTINUE
TEST(J) = TEST(J+1)
70 CONTINUE
IF (TESTH) 90,90,80
80 CONTINUE
CSZ = CSZ*ARP
CST = CST*ARP
CTR = CTR*ARP
CSR = CSR*ARP
COM = COM*ARP
CML = CMU*ARP
RSTS = CSZ+CST+CSR
RSTN = (CSR-V(L))*(CST+CSZ)/E(L)
TSTN = (CST-V(L))*(CSR+CSZ)/E(L)
STRESSES -- CSZ VERTICAL CST TANGENTIAL
CSR RADIAL CTR SHEAR BSTS BULK
STRAINS RSTN RADIAL TSTN TANGENTIAL
CMU SHEAR
COM VERTICAL DISPLACEMENT
IF (IZ-1) 100,100,170
IZ = 1 PART ONE CRACKING INDEX
IZ NOT 1 PART TWO RUT DEPTH
100 ANS(MM) = TSTN
IF (TZZ) 120,120,110
110 Z = -Z
120 CONTINUE
IF (MM-1K) 130,140,140
130 RETURN
140 IW = IW+1
CALL IOBIN (GMWRITER, 2, ANS(I), IW, IDX(IW))
IF (IW-1) 150,150,155
150 IF (IE.EQ.1) PRINT 330, Z, ANS(I)
IF (IE.NE.1) PRINT 335, Z, ANS(I)
RETURN
155 IF (IE-1) 160, 160, 165
160 PRINT 340, ANS
RETURN
165 PRINT 345, ANS

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CRK 907
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CRK 963
CRK 964

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RETURN
C
C PART TWO
170 CONTINUE
C
C M LOAD GROUP INDEX
LTZ = TZZ
ITZL = ITZ - LTZ
AN1(MM, ITZL, M) = AN1(MM, ITZL, M)+CSZ/2.0
AN3(MM, ITZL, M) = AN3(MM, ITZL, M)+CSR/2.0
C
C IK EITHER 1 OR 12
IK = 1 NO MONTHLY VARIATIONS ON E
IK GT 1 MONTHLY VARIATIONS MUST BE ACCUMULATED
MM INDEX VARYING FROM 1 TO IK
C
C MM = IK CALCULATIONS COMPLETE
IZ NUMBER OF LAYERS MINUS j
IF ((IX.EQ. 1) .AND. (M.EQ. N1R)) GO TO 180
GO TO 190
180 DSP(MM, 1) = COM
DSP(MM, 2) = CSZ
DSP(MM, 3) = CSR
190 CONTINUE
DSP(MONTH, I) VALUE AT SUBGRADE DUE TO 18-KIP LOAD
C I = 1 VERTICAL DISPLACEMENT
C I = 2 VERTICAL STRESS
C I = 3 RADIAL STRESS
C
C IF (MM-1K) 200,200,200
CONTINUE
C NO MONTHLY VARIATIONS IN ELASTIC MODULUS
IF (IK-1) 210,210,260
MM = MM+1
ANS(MM) = CSZ
IF (NX-1) 220,250,220
220 CONTINUE
ANS(MM+2) = CSR
IF (MM-2) 230,240,240
230 RETURN
240 MM = 0
PRINT 350, LOAD(M), MAT(ITZ), (ANS(I), I = 1, 4)
RETURN
C
C ANS(1) = VERTICAL STRESS AT TOP OF LAYER
C ANS(3) = AT BOTTOM OF LAYER
C ANS(2) = RADIAL STRESS AT TOP OF LAYER
C ANS(4) = AT BOTTOM OF LAYER
C
C SUBGRADE (NO BOTTOM OF LAYER)
250 ANS(MM+1) = CSR
MM = 0
PRINT 360, LOAD(M), MAT(ITZ), ANS(1), ANS(2)
RETURN
C
C MONTHLY VARIATIONS IN ELASTIC MODULUS
IF (MM-1) 270,270,280
260 FIRST MONTH INITIALIZATION
C

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```

CRK 965
CRK 966
CRK 967
CRK 968
CRK 969
CRK 970
CRK 971
CRK 972
CRK 973
CRK 974
CRK 975
CRK 976
CRK 977
CRK 978
CRK 979
CRK 980
CRK 981
CRK 982
CRK 983
CRK 984
CRK 985
CRK 986
CRK 987
CRK 988
CRK 989
CRK 990
CRK 991
CRK 992
CRK 993
CRK 994
CRK 995
CRK 996
CRK 997
CRK 998
CRK 999
CRK1000
CRK1001
CRK1002
CRK1003
CRK1004
CRK1005
CRK1006
CRK1007
CRK1008
CRK1009
CRK1010
CRK1011
CRK1012
CRK1013
CRK1014
CRK1015
CRK1016
CRK1017
CRK1018
CRK1019
CRK1020
CRK1021
CRK1022

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```

270      II = 0
280      II = II+1
      MN = MN+1

      XANS(II, MN) = CSZ
      IF (NX .EQ. 1) XANS(II, MN+1) = CSR
      XANS(II, MN+2) = CSR
      IF ((II .EQ. 12) .AND. (NX .EQ. 1)) GO TO 290
      RETURN
290      CONTINUE

      VALUES ARE PRINTED IN BLOCKS OF SIX MONTHS

      J = 0
      IZ1 = IZ-1
      DO 310 I = 1, IZ1
      J = J+1
      JP = J+3
      DO 300 K = 1, 12, 6
      KK = K+5
      PRINT 370, (MONTH(KI), KI = K, KK), MAT(I+1)
      PRINT 380, ((XANS(NI, MX), NI = K, KK), MX = J, JP)
300      CONTINUE
      J = JP
310      CONTINUE

      PRINT SUBGRADE VALUES
      J = J+1
      J1 = J+1
      DO 320 K = 1, 12, 6
      KK = K+5
      PRINT 370, (MONTH(KI), KI = K, KK), MAT(IZ+1)
      PRINT 390, ((XANS(NI, MX), NI = K, KK), MX = J, J1)
320      CONTINUE
      RETURN

330      FORMAT ( 10X, *MEAN VALUE* 4X, F6.2, 8X, E10.3 )
335      FORMAT ( 10X, *CONF VALUE* 4X, F6.2, 8X, E10.3 )
340      FORMAT ( 15X, *MEAN VALUES*/10X, 6E10.3/15X, 6E10.3 )
345      FORMAT ( 15X, *CONFIDENCE VALUES*/10X, 6E10.3/15X, 6E10.3 )
350      FORMAT ( 14X, F4.0, 4X, A10, 6X, 2E10.2, 3X, 2E10.2 )
360      FORMAT ( 14X, F4.0, 4X, A10, 6X, E10.2, 13X, E10.2 )
370      FORMAT ( 24X, 6A10 / 13X, A10 )
380      FORMAT ( 19X, *VERT* / 18X, *TOP* 4X, 6E10.3 / 18X, *BOTTOM*
      6E10.3 / 19X, *RADIAL*/ 18X, *TOP* 4X, 6E10.3 / 18X
      *BOTTOM* 6E10.3 / )
390      FORMAT ( 19X, *VERT* 2X, 6E10.3 / 19X, *RAD * 6E10.3// )
      END

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CRK1023
CRK1024
CRK1025
CRK1026
CRK1027
CRK1028
CRK1029
CRK1030
CRK1031
CRK1032
CRK1033
CRK1034
CRK1035
CRK1036
CRK1037
CRK1038
CRK1039
CRK1040
CRK1041
CRK1042
CRK1043
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CRK1059
CRK1060
CRK1061
CRK1062
CRK1063
CRK1064
CRK1065
CRK1066
CRK1067
CRK1068
CRK1069
CRK1070

SUBROUTINE PART (NTEST, AZ, ITN, A0)
DIMENSION AZ(400), RZ(400)
*****SUBROUTINE PART - 5-LAYER ELASTIC SYSTEM *****
** COMPLETE ZEROS OF J1(X) AND J0(X). SET UP GAUSS CONSTANTS **
10      RZ(1) = 0.0
      RZ(2) = 1.0
      RZ(3) = 2.4048
      RZ(4) = 3.8917
      RZ(5) = 5.5201
      RZ(6) = 7.0156
      K = ITN+1
      DO 20 I = 7, K, 2
      Y = 1/2
      TD = 4.0*Y-1.0
      RZ(I) = 3.1415927*(Y-0.25+0.050661/TD+0.053041/TD
      **3*0.262051/ID**5)
20      1      DO 30 I = 8, ITN, 2
      Y = (I-2)/2
      TD = 4.0*Y+1.0
      RZ(I) = 3.1415927*(Y+0.25-0.151982/TD+0.015399/TD
      **3*0.25270/ID**5)
30      1      G1 = 0.86113631
      G2 = 0.33998104
      ZF = AR
      NTEST = 2
      ** COMPLETE POINTS FOR LEGENDRE-GAUSS INTEGRATION **
      K = 1
      ZF = 2.0*ZF
      SZ2 = 0.0
      DO 40 I = 1, ITN
      SZ1 = SZ2
      SZ2 = RZ(I+1)/ZF
      SF = SZ2-SZ1
      PM = SZ2+SZ1
      SG1 = SF*G1
      SG2 = SF*G2
      AZ(K) = PM-SG1
      AZ(K+1) = PM-SG2
      AZ(K+2) = PM-SG2
      AZ(K+3) = PM-SG1
      K = K+4
40      CONTINUE
      RETURN
      END

```

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CRK1071
CRK1072
CRK1073
CRK1074
CRK1075
CRK1076
CRK1077
CRK1078
CRK1079
CRK1080
CRK1081
CRK1082
CRK1083
CRK1084
CRK1085
CRK1086
CRK1087
CRK1088
CRK1089
CRK1090
CRK1091
CRK1092
CRK1093
CRK1094
CRK1095
CRK1096
CRK1097
CRK1098
CRK1099
CRK1100
CRK1101
CRK1102
CRK1103
CRK1104
CRK1105
CRK1106
CRK1107
CRK1108
CRK1109
CRK1110
CRK1111
CRK1112
CRK1113
CRK1114

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FUNCTION POLY (CM, A, K, DVS)
NS      NUMBER OF MATERIALS
K       MATERIAL NUMBER UNDER CONSIDERATION
A       DESIGN VERTICAL STRESS
CM      CUMULATIVE EQUIVALENT 18-KIP AXLE APPLICATIONS
DVS     DESIGN RADIAL STRESS

GO TO (423,413,403), K
SUBGRADE
S1 = -ASALN = ALOG10(CM)
E = 0.3*561*S1-0.04064*S1*ALN-0.06511*S1*S1+0.00283
  *S1*3.0+0.00744*S1*S1*ALN
POLY = AMAX1(E, 0.0)
RETURN

BASE
CONTINUE
S1 = -ASS3 = -DVS
ALA = ALOG10(CM)*ALN2 = ALN**2
E = 0.57852-0.20640*S3+0.07854*S1-0.01464*S3*ALN
  -0.00121*S1*ALN-0.00408*S1*S3+0.03846*ALN2-0.00093
  *S1*S1-0.00062*ALN*S3*S3-0.00292*ALN*ALN2+0.00204
  *S3*3+0.0001*S1*3-0.0004*S3*S3*S1+0.00006
  *S1*S1*S3+0.00046*S1*S3*ALN
POLY = AMAX1(E, 0.0)
RETURN

423 RETURN
S1 = -ASS3 = -DVS
ALA = ALOG10(CM)*ALN2 = ALN*ALN
E = -0.75465+0.25605*ALN+0.17009*S1-0.14433*ALN*S3
  +0.01187*ALN*S1+0.01139*S1*S3+0.04947*S3*S3
  -0.01132*S1*S1+0.03340*ALN*S3*S3+0.00115*ALN
  *S1*S1-0.01885*S3*3+0.00025*S1*3+0.00367*S3
  *S3*S1-0.00072*S1*S1*S3-0.01018*S1*S3*ALN
POLY = AMAX1(E, 0.0)
RETURN
END

```

```

CRK1115
CRK1116
CRK1117
CRK1118
CRK1119
CRK1120
CRK1121
CRK1122
CRK1123
CRK1124
CRK1125
CRK1126
CRK1127
CRK1128
CRK1129
CRK1130
CRK1131
CRK1132
CRK1133
CRK1134
CRK1135
CRK1136
CRK1137
CRK1138
CRK1139
CRK1140
CRK1141
CRK1142
CRK1143
CRK1144
CRK1145
CRK1146
CRK1147
CRK1148
CRK1149
CRK1150
CRK1151

REAL MU
FUNCTION XNOHM (X, MU, SIGMA)
Z = X*0.005
A = 0.0
A = A*0.01*EXP(-Z*Z/2.)/(2.*3.1415926)**0.5
Z = Z*0.01
IF (Z-5.0) 10,10,20
XNCRM = A
RETURN
END

```

```

CRK1152
CRK1153
CRK1154
CRK1155
CRK1156
CRK1157
CRK1158
CRK1159
CRK1160
CRK1161

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```

FUNCTION ALPHA (CONF)
A = 0.5-CONF
C = 0.0
DELT = 0.0001
Z = 0.00005
C = C+DELT*EXP(-Z*Z/2.)/(SQRT(2.*3-1415926))
10 IF (C-ABS(A)) 20,30,30
20 Z = Z+DELT
GO TO 10
30 AA = DELT/2.0+Z
IF (A) 50,40,40
40 ALPHA = AA
RETURN
50 ALPHA = -AA
RETURN
END

```

```

CRK1162
CRK1163
CRK1164
CRK1165
CRK1166
CRK1167
CRK1168
CRK1169
CRK1170
CRK1171
CRK1172
CRK1173
CRK1174
CRK1175
CRK1176
CRK1177

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```

SUBROUTINE COEE (MIN, N, E, V, W, NS, P)
*****SUHDOUTINE COEE - 5-LAYER ELASTIC SYSTEM *****
CCOEE
COMMON /COECAL/ A(400, 5), B(400, 5), C(400, 5), D(400, 5)
DIMENSION E(5), V(5), H(4), X(5,4), SC(4), PH(4,4,4), FM(2,2)
DIMENSION SV1(4,2), CV1(2,1), SV2(4,4), CV2(2,2), SV3(4,8), CV3(2,
1,1), SV4(4,16), CV4(2,8), T(B), NT(4)
LC = KIN
CS=MK SET UP MATRIX X = 0I*MI*KI*K*W*D
C COMPUTE THE MATRICES X(K)
DO 10 K = 1, A
T1 = E(K)*(1.0-V(K+1))/(E(K+1)*(1.0+V(K)))
T1P = T1-1.0
PH = P*H(K)
PH2 = PH*2.0
VK2 = 2.0*V(K)
VKP2 = 2.0*V(K+1)
VK4 = 2.0*VK2
VKP4 = 2.0*VKP2
VKK8 = 8.0*V(K)*V(K+1)
X(K, 1, 1) = VK4-3.0-T1
X(K, 2, 1) = 0.0
X(K, 3, 1) = T1P*(PH2-VK4-1.0)
X(K, 4, 1) = -2.0*T1P*P
T3 = PH2*(VK2-1.0)
T4 = VKK8+1.0-3.0*VKP2
T5 = PH2*(VKP2-1.0)
T6 = VKK8+1.0-3.0*VK2
X(K, 1, 2) = (T3+T4-T1*(T5+T6))/P
X(K, 2, 2) = T1*(VKP4-3.0)-1.0
X(K, 4, 2) = T1P*(1.0-PH2-VK4)
X(K, 3, 4) = (T3-T4-T1*(T5-T6))/P
T3 = P*2*PH-VKK8+1.0
T4 = P*2*(VK2-VKP2)
X(K, 1, 4) = (T3+T4+VKP2-T1*(T3+T4+VK2))/P
X(K, 3, 2) = (-T3-T4-VKP2+T1*(T3-T4+VK2))/P
X(K, 1, 3) = T1P*(1.0-PH2-VK4)
X(K, 2, 3) = 2.0*T1P*P
X(K, 3, 3) = VK4-3.0-T1
X(K, 4, 3) = 0.0
X(K, 2, 4) = T1P*(PH2-VK4+1.0)
X(K, 4, 4) = T1*(VKP4-3.0)-1.0
K = K
10 CONTINUE
C COMPUTE THE PRODUCT MATRICES PM
SC(N) = 4.0*(V(N)-1.0)
IF (N=2) 40,20,20
LC 30 K1 = 2, N
.. = NS-K1
SC(M) = SC(M+1)+4.0*(V(M)-1.0)
30 CONTINUE

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CRK1178
CRK1179
CRK1180
CRK1181
CRK1182
CRK1183
CRK1184
CRK1185
CRK1186
CRK1187
CRK1188
CRK1189
CRK1190
CRK1191
CRK1192
CRK1193
CRK1194
CRK1195
CRK1196
CRK1197
CRK1198
CRK1199
CRK1200
CRK1201
CRK1202
CRK1203
CRK1204
CRK1205
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CRK1207
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CRK1223
CRK1224
CRK1225
CRK1226
CRK1227
CRK1228
CRK1229
CRK1230
CRK1231
CRK1232
CRK1233
CRK1234
CRK1235

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```

40 CONTINUE
C
K = N
DO 60 I = 1, 4
DO 50 J = 1, 2
50 SV1(I, J) = X(K, I, J+2)
60 CONTINUE
CV1(1, 1) = -2.0*P*M(K)
CV1(2, 1) = 0.0
K = K-1
IF (K) 200,200,70

70 CONTINUE
DO 90 J = 1, 2
J1 = J+J
T(1) = SV1(1, J)
T(2) = SV1(2, J)
T(3) = SV1(3, J)
T(4) = SV1(4, J)
DO 80 I = 1, 4
80 SV2(I, J1-1) = X(K, I, 1)*T(1)+X(K, I, 2)*T(2)
90 SV2(I, J1) = X(K, I, 3)*T(3)+X(K, I, 4)*T(4)
CONTINUE
T(1) = CV1(1, 1)
T(2) = -2.0*P*M(K)
CV2(1, 1) = T(1)
CV2(1, 2) = T(2)
CV2(2, 1) = T(1)-T(2)
CV2(2, 2) = 0.0
K = K-1
IF (K) 200,200,100

100 CONTINUE
DO 140 J = 1, 4
J1 = J
IF (J1-2) 120,120,110
110 J1 = J1+2
120 CONTINUE
T(1) = SV2(1, J)
T(2) = SV2(2, J)
T(3) = SV2(3, J)
T(4) = SV2(4, J)
DO 130 I = 1, 4
130 SV3(I, J1) = X(K, I, 1)*T(1)+X(K, I, 2)*T(2)
140 SV3(I, J1+2) = X(K, I, 3)*T(3)+X(K, I, 4)*T(4)
CONTINUE
T(1) = -2.0*P*M(K)
DO 150 J = 1, 2
CV3(1, J) = CV2(1, J)
CV3(2, J) = CV2(1, J)-T(1)
CV3(1, J+2) = CV2(2, J)*T(1)
CV3(2, J+2) = CV2(2, J)
150 CONTINUE
K = K-1
IF (K) 200,200,160

C
160 CONTINUE
DO 180 J = 1, 4

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CRK1236
CRK1237
CRK1238
CRK1239
CRK1240
CRK1241
CRK1242
CRK1243
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CRK1247
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CRK1289
CRK1290
CRK1291
CRK1292
CRK1293

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```

T(1) = SV3(1, J)
T(2) = SV3(2, J)
T(3) = SV3(3, J)
T(4) = SV3(4, J)
T(5) = SV3(1, J+4)
T(6) = SV3(2, J+4)
T(7) = SV3(3, J+4)
T(8) = SV3(4, J+4)
DO 170 I = 1, 4
SV4(1, J) = X(K, I, 1)*T(1)+X(K, I, 2)*T(2)
SV4(I, J+4) = X(K, I, 3)*T(3)+X(K, I, 4)*T(4)
SV4(I, J+8) = X(K, I, 1)*T(5)+X(K, I, 2)*T(6)
SV4(I, J+12) = X(K, I, 3)*T(7)+X(K, I, 4)*T(8)
170 CONTINUE
180 T(1) = -2.0*P*M(K)
DO 190 J = 1, 4
CV4(1, J) = CV3(1, J)
CV4(2, J) = CV3(1, J)-T(1)
CV4(1, J+4) = CV3(2, J)+T(1)
CV4(2, J+4) = CV3(2, J)
190 CONTINUE
C
200 CONTINUE
NT(1) = 1
DO 210 K = 2, N
NT(K) = NT(K-1)+NT(K-1)
DO 370 K = 1, N
K1 = NS-K
DO 220 I = 1, 4
PM(K1, I, 1) = 0.0
PM(K1, I, 2) = 0.0
220 CONTINUE
I1 = NT(K)
DO 370 I = 1, I1
I2 = I+I1
GO TO (230,240,250,260), K
230 CONTINUE
T(3) = CV1(1, I)
T(4) = CV1(2, I)
GO TO 270
CONTINUE
T(3) = CV2(1, I)
T(4) = CV2(2, I)
GO TO 270
CONTINUE
T(2) = CV3(1, I)
T(4) = CV3(2, I)
GO TO 270
CONTINUE
T(3) = CV4(1, I)
T(4) = CV4(2, I)
270 CONTINUE
T(1) = 0.0
T(2) = 0.0
IF (T(3)+68.0) 290,280,280
280 T(1) = EXP(T(3))
290 IF (T(4)+68.0) 310,300,300
300 T(2) = EXP(T(4))

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CRK1294
CRK1295
CRK1296
CRK1297
CRK1298
CRK1299
CRK1300
CRK1301
CRK1302
CRK1303
CRK1304
CRK1305
CRK1306
CRK1307
CRK1308
CRK1309
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CRK1348
CRK1349
CRK1350
CRK1351

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```

310 CONTINUE
    DO 370 J = 1, 2
    GO TO (320,330,340,350), K
320 CONTINUE
    T(3) = SV1(J, 1)
    T(4) = SV1(J, 12)
    T(5) = SV1(J*2, 1)
    T(6) = SV1(J*2, 12)
330 GO TO 360
    T(3) = SV2(J, 1)
    T(4) = SV2(J, 12)
    T(5) = SV2(J*2, 1)
    T(6) = SV2(J*2, 12)
340 GO TO 360
    T(3) = SV3(J, 1)
    T(4) = SV3(J, 12)
    T(5) = SV3(J*2, 1)
    T(6) = SV3(J*2, 12)
350 GO TO 360
    T(3) = SV4(J, 1)
    T(4) = SV4(J, 12)
    T(5) = SV4(J*2, 1)
    T(6) = SV4(J*2, 12)
360 CONTINUE
    PM(K1, J, 1) = PM(K1, J, 1)*T(1)*T(3)
    PM(K1, J, 2) = PM(K1, J, 2)*T(1)*T(4)
    PM(K1, J*2, 1) = PM(K1, J*2, 1)*T(2)*T(5)
    PM(K1, J*2, 2) = PM(K1, J*2, 2)*T(2)*T(6)
370 CONTINUE
    SOLVE FOR C(NS) AND D(NS)
    V2 = 2.0*V(1)
    V21 = V2-1.0
    DO 380 J = 1, 2
    FM(1, J) = P*PM(1, 1, J)+V2*PM(1, 2, J)+P*PM(1, 3,
    J)+V2*PM(1, 4, J)
    FM(2, J) = P*PM(1, 1, J)+V2*PM(1, 2, J)+P*PM(1,
    3, J)+V21*PM(1, 4, J)
    DFAC = SC(1)/(FM(1, 1)*FM(2, 2)-FM(2, 1)*FM(1, 2))
    A(ILC, NS) = 0.0
    B(ILC, NS) = 0.0
    C(ILC, NS) = -FM(1, 2)*DFAC
    D(ILC, NS) = FM(1, 1)*DFAC
C BACKSOLVE FOR THE OTHER A+B+C+D
    DO 390 K1 = 1, N
    A(ILC, K1) = (PM(K1, 1, 1)*C(ILC, NS)+PM(K1, 1, 2)
    *D(ILC, NS))/SC(K1)
    B(ILC, K1) = (PM(K1, 2, 1)*C(ILC, NS)+PM(K1, 2, 2)
    *D(ILC, NS))/SC(K1)
    C(ILC, K1) = (PM(K1, 3, 1)*C(ILC, NS)+PM(K1, 3, 2)
    *D(ILC, NS))/SC(K1)
    D(ILC, K1) = (PM(K1, 4, 1)*C(ILC, NS)+PM(K1, 4, 2)
    *D(ILC, NS))/SC(K1)
390 RETURN
END

```

```

CRK1362
CRK1363
CRK1364
CRK1365
CRK1366
CRK1367
CRK1368
CRK1369
CRK1370
CRK1371
CRK1372
CRK1373
CRK1374
CRK1375
CRK1376
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CRK1378
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CRK1389
CRK1390
CRK1391
CRK1392
CRK1393
CRK1394
CRK1395
CRK1396
CRK1397
CRK1398
CRK1399
CRK1400
CRK1401
CRK1402
CRK1403
CRK1404
CRK1405
CRK1406
CRK1407

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```

SUBROUTINE BESSEL (NI, XI, Y)
CHESSSEL *****SUBROUTINE BESSEL - 5-LAYER FLASTIC SYSTEM *****
DIMENSION PZ(6), QZ(6), P1(6), Q1(6), O(20)
C
C
10 PZ(1) = 1.0
    PZ(2) = -1.125E-4
    PZ(3) = 2.8710938E-7
    PZ(4) = -2.3449658E-9
    PZ(5) = 3.9806841E-11
    PZ(6) = -1.1536133E-12
C
    QZ(1) = -5.0E-3
    QZ(2) = 4.6875E-6
    QZ(3) = -2.3256859E-8
    QZ(4) = 2.8307087E-10
    QZ(5) = -6.3912096E-12
    QZ(6) = 2.3124704E-13
C
    P1(1) = 1.0
    P1(2) = 1.875E-4
    P1(3) = -3.6914063E-7
    P1(4) = 2.7713232E-9
    P1(5) = -4.5114421E-11
    P1(6) = 1.2750463E-12
C
    Q1(1) = 1.5E-2
    Q1(2) = -6.5625E-6
    Q1(3) = 2.8423828E-8
    Q1(4) = -6.5625E-6
    Q1(5) = 2.8443828E-8
    Q1(6) = -3.2662024E-10
    Q1(7) = 7.1431166E-12
    Q1(8) = -2.5327056E-13
C
    PI = 3.1415927
    P12 = 2.0*PI
C
    N = NI
    K = XI
    IF (X-7.0) 20,20,100
20 X2 = X/2.0
    FAC = -X2*X2
    IF (N) 30,30,60
30 C = 1.0
    V = C
50 I = 1, 34
    T = 1
    C = FAC*C/(T*T)
    TEST = ABS(C)-10.0**(-8)
    IF (TEST) 90,90,40
40 Y = Y*C
50 CONTINUE
60 C = X2

```

```

CRK1408
CRK1409
CRK1410
CRK1411
CRK1412
CRK1413
CRK1414
CRK1415
CRK1416
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CRK1457
CRK1458
CRK1459
CRK1460
CRK1461
CRK1462
CRK1463
CRK1464
CRK1465

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```

      Y = C
EQ 80 I = 1, 34
      T = 1
      C = FAC*C/(T*(T+1.0))
      TEST = ABS(C)-10*0**(-8)
      IF (TEST) 90,90,70
      Y = Y+C
70 CONTINUE
80 RETURN
90 IF (N) 110,110,130
C
110 DO 120 I = 1, 6
      D(I) = P2(I)
      D(I+10) = QZ(I)
120 CONTINUE
      GO TO 150
C
130 DO 140 I = 1, 6
      D(I) = P1(I)
      D(I+10) = Q1(I)
140 CONTINUE
150 CONTINUE
      T1 = 25.0/X
      T2 = T1*T1
      P = U(6)*T2*D(5)
DO 160 I = 1, 4
      J = 5-I
      P = P+T2*D(J)
160 CONTINUE
      Q = U(16)*T2*D(15)
DO 170 I = 1, 4
      J = 5-I
      Q = Q+T2*D(J+10)
170 CONTINUE
      Q = Q*T1
C
      T4 = SQRT(X*PI)
      T6 = SIN(X)
      T7 = COS(X)
C
      IF (N) 180,180,190
C
180 TS = ((P-Q)*T6+(P+Q)*T7)/T4
      GO TO 200
190 TS = ((P+Q)*T6-(P-Q)*T7)/T4
200 Y = TS
RETURN
EAD

```

```

CRK1466
CRK1467
CRK1468
CRK1469
CRK1470
CRK1471
CRK1472
CRK1473
CRK1474
CRK1475
CRK1476
CRK1477
CRK1478
CRK1479
CRK1480
CRK1481
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CRK1507
CRK1508
CRK1509
CRK1510
CRK1511
CRK1512
CRK1513
CRK1514

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APPENDIX 4.3

GUIDE FOR DATA INPUT



CRAKDX GUIDE FOR DATA INPUT

with supplementary notes

extract from

FLEXIBLE PAVEMENT SYSTEM - SECOND GENERATION,  
INCORPORATING FATIGUE AND STOCHASTIC CONCEPTS

by

Surendra Prakash Jain

December 1971

CRAKDX GUIDE FOR DATA INPUT

PROGRAM DESCRIPTION (one card)

Alphanumeric 8A10
-------------------

MATERIAL PARAMETERS (one card)

NL	NM	LDDF	LNDF	Y	GR	Qo	Q1	Q2	TEST
I 10	I 10	F 10.0	F 10.0	F 10.0	F 10.0	A2	A2	A2	F510
1	11	21	31	41	51	61 62	66 67	71 72	76 80

- NL     Number of load groups.
- NM     Number of materials (maximum of five).
- LDDF   Load distribution factor, ratio.
- LNDF   Lane distribution factor, ratio.
- Y       Design period, years.
- GR     Traffic growth rate, ratio. Leave blank if actual traffic data is given.
- Qo     Provide NO if the actual traffic data for each month is not given.
- Q1     Provide NO if the monthly traffic percentage does not vary.
- Q2     Provide NO if elastic modulus values of the different materials do not vary monthly.
- TEST   Leave blank if calculations for both cracking index and rut depth.  
           Provide 1.0 for cracking index only.  
           Provide 2.0 for rut depth only.

WHEEL LOAD DATA (NL cards)

L1	L2	N	TIRE
F 10.0	F 10.0	F 10.0	F 10.0

L1 - L2 Axle load range, kips.

N Number of axle applications in wheel load group for first year. Leave blank if actual monthly traffic data for each year is given.

TIRE Tire pressure, PSI.

MATERIAL DATA (NM cards if Q2 = NO , NM x 2 cards if Q2 is not NO)

MAT	TMIN	NU	CONF	SIG	E(1)	E(2)	E(3)	E(4)
A 10	F 10.0	F5.0	F5.0	F 10.0	F 10.0	F 10.0	F 10.0	F 10.0
E(5)	E(6)	E(7)	E(8)	E(9)	E(10)	E(11)	E(12)	
F 10.0	F 10.0	F 10.0	F 10.0	F 10.0	F 10.0	F 10.0	F 10.0	

MAT Material identification

TMIN Thickness of material, inches. Leave blank for subgrade.

NU Poisson's ratio.

CONF Confidence level for elastic modulus.

E Elastic modulus mean value. If Q2 = NO , provide one constant value. If Q2 is not NO provide one value for each month.

SIG Standard deviation of elastic modulus, expressed as a percent of mean modulus, i.e., coefficient of variation, percent.

FATIGUE CURVE DATA

A	B	log SD	LN
F 10.0	F 10.0	F 10.0	F 10.0

A Constant of fatigue curve  $N = A \left( \frac{1}{\sigma} \right)^B$ .

B Slope of fatigue curve.

log SD Log standard deviation of fatigue life.

LN Confidence level for N.

MONTHLY TRAFFIC PERCENTIAGES (one card if Q0 is NO and Q1 is not NO)

TRAF (1)	TRAF (2)	TRAF (3)	TRAF (4)	TRAF (5)	TRAF (6)	TRAF (7)	TRAF (8)	TRAF (9)	TRAF (10)	TRAF (11)	TRAF (12)
F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0	F 5.0

TRAF(I) Traffic percentage for Month I. If Q1 = NO, a value of 0.0833 will be assumed for each month. No card is needed if actual traffic data for each month is given.

ACTUAL TRAFFIC DATA (number of cards = number of design years x number of load groups if Q0 is not NO)

YR	TR(1)	TR(2)	TR(3)	TR(4)	TR(5)	TR(6)	TR(7)	TR(8)	TR(9)	TR(10)	TR(11)	TR(12)
A8	I6	I6	I6	I6	I6	I6	I6	I6	I6	I6	I6	I6

YR Year identification.

TR(1) Monthly traffic from January to December.

Provide a full set of traffic data for each load group separately and arrange the sets in the same sequence as in wheel load group.

DENSITY AND PRESSURE PARAMETERS (NM/2 CARDS)

R(1)	EM(1)	DRC(1)	R(2)	EM(2)	DRC(2)	
F10.0	F10.0	F10.0	F10.0	F10.0	F10.0	
1	10	20	30	40	50	60

R(I) Unit weight of material I (lb/cu in)

EM(I) Parameter to calculate correct radial pressure for given curve for base and subbase.  
 Leave blank if regression equation in N, Z1, Z3, and E is given. Leave blank for subgrade.

DRC(I) Radial pressure for which curves are given for material I.

APPENDIX 4.4

INPUT DATA SAMPLE

APPENDIX 4.4. INPUT DATA SAMPLE

IDENTIFICATION A4.4 INPUT DATA SAMPLE CODED BY JAIN DATE JULY 30,71 PAGE 1 OF 1

1	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80
CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM																
	1		4		1.0		1.0		3.0							
	6.0		6.0				42.3									
A. CONCRETE		1.0	0.3	0.25		25.0	160000.0	142000.0	137000.0	90000.0						
	60000.0	40000.0	35000.0	30000.0	42000.0	65000.0	83000.0	131000.0								
BASE		3.0	0.4	0.25		25.0	2400.0	2400.0	2400.0	15600.0						
	1800.0	1960.0	2160.0	2320.0	2400.0	2400.0	2400.0	2400.0	2400.0	2400.0						
SUBBASE		4.0	0.45	0.25		25.0	1320.0	1320.0	1320.0	7500.0						
	860.0	980.0	1080.0	1160.0	1220.0	1240.0	1280.0	1320.0								
SUBGRADE			0.50	0.25		25.0	660.0	660.0	660.0	3600.0						
	430.0	490.0	540.0	580.0	610.0	620.0	640.0	660.0								
	6.5E-07		3.18	0.25		0.05										
FIRST YR								400	4700	14600	16900	23200	17700			
SECOND YR	21900	17100	32900	33700	39900	38600	37000	37700	37500	31500	73400	85200				
THIRD YR	73500	74000	73000	67100	66600	63700	74100	54800								
	0.08				0.08											
	0.08				0.07					-3.5						

APPENDIX 4.5

CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM



CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM

AXLE LOAD RANGE,KIPS 6 - 6  
 AXLE LOAD MEAN,KIPS 6.00  
 TIRE PRESSURE 42.30  
 N INITIAL AXLE APPL -0

LOAD DISTRIBUTION FACTOR, RATIO 1.00  
 LANE DISTRIBUTION FACTOR, RATIO 1.00  
 DESIGN PERIOD, YEARS 3

MATERIAL PARAMETERS

MATERIAL	A.CONCRETE BASE	SUB BASE	SUBGARDE
THICKNESS	1.00	3.00	4.00
POISSONS RATIO	.30	.40	.45
COP VAR,PERCENT	25.00	25.00	25.00

VARIATIONS OF E VALUES IN SPACE AND TIME

MATERIAL	CONF	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE
		JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
A.CONCRETE	.25	1330200	1109553	1138984	748238	498825	332550
		290981	249413	3491775	540394	690041	1089101
		MEAN 1600000	1420000	1370000	900000	600000	400000
		350000	300000	4200000	650000	830000	1310000
BASE	.25	19953	19953	19953	12969	14965	16295
		17958	19288	19953	19953	19953	19953
		MEAN 24000	24000	24000	15600	18000	19600
		21600	23200	24000	24000	24000	24600
SUB BASE	.25	10974	10974	10974	6235	7150	8147
		8979	9694	10143	10309	10642	10974
		MEAN 13200	13200	13200	7500	8600	9800
		10800	11660	12200	12400	12800	13200
SUBGARDE	.25	5487	5487	5487	2993	3575	4074
		4489	4822	5071	5155	5321	5487
		MEAN 6600	6600	6600	3600	4300	4900
		5400	5800	6100	6200	6400	6600

CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM

N = 6.5E-07 \* (1/E) \*\* 3.18  
 LOG STANDARD DEVIATION OF .25  
 AND CONFIDENCE LEVEL .05

TANGENTIAL STRAIN

LOAD	6	DEPTH	1	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	NOVEMBER	DECEMBER
MEAN VALUES				2.027E-04	2.076E-04	2.089E-04	3.365E-04	3.073E-04	2.666E-04		
CONFIDENCE VALUES				2.284E-04	1.948E-04	1.514E-04	2.232E-04	2.221E-04	2.105E-04		
				2.438E-04	2.497E-04	2.513E-04	4.048E-04	3.696E-04	3.207E-04		
				2.748E-04	2.343E-04	1.822E-04	2.685E-04	2.671E-04	2.532E-04		

N TABLE - ACTUAL FROM TRAFFIC DATA INPUT

	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE
	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
FIRST YR	0	0	0	0	0	0
SECONDYR	16900	23200	17700	21900	17100	32900
3RD YEAR	31500	73400	88200	73500	74000	73000
	67100	68600	63700	74100	54800	0

N TABLE - THEORETICAL

LOAD	6	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE
CONF LEVEL		JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
.050		77798	72095	74632	15516	20709	32529
MEAN		53184	80304	196567	57224	58188	68955
		246635	409496	911550	265370	269841	319767

CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM

CRACKING INDEX AND RUT DEPTH INDEX EXAMPLE PROBLEM

LOAD YEAR	CONF	M ACTUAL / N THEORETICAL											
		JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
1	.054	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	MEAN	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2	.054	2.172E-01	3.218E-01	2.506E-01	1.411E+00	8.257E-01	1.011E+00	6.336E-01	4.518E-01	1.944E-01	6.466E-01	6.479E-01	5.438E-01
	MEAN	4.684E-02	6.939E-02	5.404E-02	3.044E-01	1.781E-01	2.181E-01	1.366E-01	9.744E-02	4.235E-02	1.394E-01	1.397E-01	1.173E-01
3	.054	4.049E-01	1.018E+00	1.206E+00	4.737E+00	3.573E+00	2.244E+00	1.282E+00	7.769E-01	3.241E-01	1.295E+00	9.418E-01	0.
	MEAN	8.731E-02	2.195E-01	2.601E-01	1.022E+00	7.705E-01	4.839E-01	2.721E-01	1.675E-01	6.988E-02	2.792E-01	2.031E-01	0.

MONTH	CONF	(N/A)T	LOG(N/N)T	LOG SD	K	A	CI
1	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
2	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
3	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
4	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
5	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
6	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
7	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
8	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
9	.0539	0	-	-	-	0	0
	MEAN	0	-	-	-	0	0
10	.0539	6.990E-03	-2.156E+00	4.143E-01	6.81E+00	3.26E-13	3.26E-10
	MEAN	1.5073E-03	-2.8218E+00				
11	.0539	6.776E-02	-1.057E+00	4.143E-01	4.16E+00	1.57E-05	1.57E-02
	MEAN	1.8925E-02	-1.7230E+00				
12	.0539	2.995E-01	-5.236E-01	4.143E-01	2.87E+00	2.04E-03	2.04E+00
	MEAN	6.4583E-02	-1.1899E+00				
13	.0539	5.167E-01	-2.867E-01	4.143E-01	2.30E+00	1.07E-02	1.07E+01
	MEAN	1.1143E-01	-9.5301E-01				
14	.0539	8.385E-01	-7.649E-02	4.143E-01	1.79E+00	3.65E-02	3.65E+01
	MEAN	1.8082E-01	-7.4276E-01				
15	.0539	1.089E+00	3.707E-02	4.143E-01	1.52E+00	6.44E-02	6.44E+01
	MEAN	2.3486E-01	-6.2920E-01				
16	.0539	2.501E+00	3.980E-01	4.143E-01	6.47E-01	2.59E-01	2.59E+02
	MEAN	5.3922E-01	-2.6823E-01				

MONTH	CONF	(N/A)T	LOG(N/N)T	LOG SD	K	A	CI
17	.0539	3.324E+00	5.220E-01	4.143E-01	3.48E-01	3.64E+01	3.64E+02
	MEAN	7.1728E-01	-1.4431E-01				
18	.0539	4.338E+00	6.373E-01	4.143E-01	7.00E-02	4.72E+01	4.72E+02
	MEAN	9.3538E-01	-2.9012E-02				
19	.0539	4.971E+00	6.965E-01	4.143E-01	-7.29E-02	5.29E+01	5.29E+02
	MEAN	1.0720E+00	2.0203E-02				
20	.0539	5.423E+00	7.343E-01	4.143E-01	-1.64E-01	5.65E+01	5.65E+02
	MEAN	1.1695E+00	6.7984E-02				
21	.0539	5.620E+00	7.497E-01	4.143E-01	-2.01E-01	5.80E+01	5.80E+02
	MEAN	1.2118E+00	8.3432E-02				
22	.0539	6.266E+00	7.970E-01	4.143E-01	-3.16E-01	6.24E+01	6.24E+02
	MEAN	1.3512E+00	1.3073E-01				
23	.0539	6.914E+00	8.397E-01	4.143E-01	-4.19E-01	6.62E+01	6.62E+02
	MEAN	1.4909E+00	1.7346E-01				
24	.0539	7.458E+00	8.726E-01	4.143E-01	-4.98E-01	6.91E+01	6.91E+02
	MEAN	1.6082E+00	2.0634E-01				
25	.0539	7.863E+00	8.956E-01	4.143E-01	-5.53E-01	7.10E+01	7.10E+02
	MEAN	1.6955E+00	2.2930E-01				
26	.0539	8.881E+00	9.485E-01	4.143E-01	-6.81E-01	7.52E+01	7.52E+02
	MEAN	1.9151E+00	2.8218E-01				
27	.0539	1.009E+01	1.004E+00	4.143E-01	-8.15E-01	7.92E+01	7.92E+02
	MEAN	2.1792E+00	3.3750E-01				
28	.0539	1.482E+01	1.171E+00	4.143E-01	-1.22E+00	8.88E+01	8.88E+02
	MEAN	3.1967E+00	9.0470E-01				
29	.0539	1.840E+01	1.265E+00	4.143E-01	-1.44E+00	9.26E+01	9.26E+02
	MEAN	3.9672E+00	5.9849E-01				
30	.0539	2.064E+01	1.319E+00	4.143E-01	-1.57E+00	9.41E+01	9.41E+02
	MEAN	4.4912E+00	6.4847E-01				
31	.0539	2.190E+01	1.341E+00	4.143E-01	-1.63E+00	9.48E+01	9.48E+02
	MEAN	4.7232E+00	6.7424E-01				
32	.0539	2.268E+01	1.356E+00	4.143E-01	-1.66E+00	9.52E+01	9.52E+02
	MEAN	4.8907E+00	6.8938E-01				
33	.0539	2.300E+01	1.362E+00	4.143E-01	-1.68E+00	9.53E+01	9.53E+02
	MEAN	4.9606E+00	6.9554E-01				
34	.0539	2.430E+01	1.386E+00	4.143E-01	-1.74E+00	9.89E+01	9.89E+02
	MEAN	5.2399E+00	7.1932E-01				
35	.0539	2.524E+01	1.402E+00	4.143E-01	-1.78E+00	9.62E+01	9.62E+02
	MEAN	5.4429E+00	7.3583E-01				
36	.0539	2.524E+01	1.402E+00	4.143E-01	-1.78E+00	9.62E+01	9.62E+02
	MEAN	5.4429E+00	7.3583E-01				

LOAD MATERIAL	RADIAL PRESSURE (INPUT)	..... JANUARY .....			..... FEBRUARY .....		
		VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS
6							
BASE		-24.278	-24.278	-1.562	-24.996	-24.996	-1.641
SUB BASE		-12.854	-12.854	-1.000	-13.149	-13.149	-1.000
SUBGARDE	-3.500	-8.436	-10.303	-1.633	-8.572	-10.441	-1.632

..... MARCH .....			..... APRIL .....			..... MAY .....		
VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS
-25.206	-25.206	-1.667	-24.304	-24.304	-1.000	-27.234	-27.234	-1.000
-13.236	-13.236	-1.000	-12.416	-12.416	-1.000	-13.711	-13.711	-1.000
-8.612	-10.481	-1.631	-8.071	-10.027	-1.544	-8.754	-10.679	-1.575

..... JUNE .....			..... JULY .....			..... AUGUST .....		
VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS
-29.428	-29.428	-1.328	-30.121	-30.121	-1.726	-30.692	-30.692	-2.267
-14.848	-14.848	-1.000	-15.225	-15.225	-1.000	-15.579	-15.579	-1.000
-9.315	-11.180	-1.635	-9.504	-11.339	-1.666	-9.680	-11.476	-1.704

..... SEPTEMBER .....			..... OCTOBER .....			..... NOVEMBER .....		
VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS	VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS
-17.572	-17.572	-1.000	-28.575	-28.575	-1.451	-27.686	-27.686	-1.658
-9.911	-9.911	-1.000	-14.517	-14.517	-1.900	-14.202	-14.202	-1.000
-6.959	-6.936	-1.623	-9.169	-11.045	-1.624	-9.038	-10.913	-1.625

..... DECEMBER .....		
VERTICAL MEAN	STRESS DESIGN	RADIAL STRESS
-25.466	-25.466	-1.700
-13.343	-13.343	-1.000
-8.662	-10.531	-1.631

YEAR 1	JANUARY		FEBRUARY		MARCH		APRIL		MAY		JUNE	
	HI-LO-REPT											
BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUB BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUBGARDE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
CUMULATIVE												
BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUB BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUBGARDE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
STRAIN TO BEGINNING OF MONTH												
BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUB BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUBGARDE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
STRAIN THROUGH MONTH												
BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUB BASE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SUBGARDE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
DEPTH												
BASE		3.0										
SUB BASE		4.0										
SUBGARDE												
VERT. DSP.	1.625E+02	1.638E+02	1.642E+02	2.901E+02	2.534E+02	2.298E+02						
Y STRESS	-7.796E+00	-7.932E+00	-7.972E+00	-7.431E+00	-8.114E+00	-8.675E+00						
R STRESS	-9.927E-01	-9.915E-01	-9.914E-01	-9.644E-01	-9.352E-01	-9.948E-01						
DEFORMATION AT MONTH N												
BASE	0.	0.	0.	0.	0.	0.						
SUB BASE	0.	0.	0.	0.	0.	0.						
SUBGARDE	0.	0.	0.	0.	0.	0.						
DEFORMATION DUE TO REPTIONS THROUGH MONTH (N-1)												
BASE	0.	0.	0.	0.	0.	0.						
SUB BASE	0.	0.	0.	0.	0.	0.						
SUBGARDE	0.	0.	0.	0.	0.	0.						
DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1												
BASE	0.	0.	0.	0.	0.	0.						
SUB BASE	0.	0.	0.	0.	0.	0.						
SUBGARDE	0.	0.	0.	0.	0.	0.						
CUMULATIVE DEFORMATION THROUGH MONTH N												
BASE	0.	0.	0.	0.	0.	0.						
SUB BASE	0.	0.	0.	0.	0.	0.						
SUBGARDE	0.	0.	0.	0.	0.	0.						
TOTAL CUMULATIVE DEFORMATION												
BASE	0.	0.	0.	0.	0.	0.						
SUB BASE	0.	0.	0.	0.	0.	0.						
SUBGARDE	0.	0.	0.	0.	0.	0.						
RUT DEPTH												

YEAR 1						
	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
HI-LO-REPT						
BASE	0.	0.	0.	4.000E+02	4.700E+03	1.460E+04
SUB BASE	0.	0.	0.	4.000E+02	4.700E+03	1.460E+04
SUBGARDE	0.	0.	0.	4.000E+02	4.700E+03	1.460E+04
CUMMULATIVE						
BASE	0.	0.	0.	4.000E+02	5.100E+03	1.970E+04
SUB BASE	0.	0.	0.	4.000E+02	5.100E+03	1.970E+04
SUBGARDE	0.	0.	0.	4.000E+02	5.100E+03	1.970E+04
STRAIN TO BEGINNING OF MONTH						
BASE	0.	0.	0.	0.	2.626E+00	2.933E+00
SUB BASE	0.	0.	0.	0.	1.575E+00	1.637E+00
SUBGARDE	0.	0.	0.	0.	7.020E-01	1.061E+00
STRAIN THROUGH MONTH						
BASE	0.	0.	0.	3.000E+00	3.567E+00	3.353E+00
SUB BASE	0.	0.	0.	1.603E+00	1.717E+00	1.720E+00
SUBGARDE	0.	0.	0.	7.293E-01	1.191E+00	1.294E+00
DEPTH						
BASE		3.0				
SUB BASE		4.0				
SUBGARDE						
VERT. DSP.	2.108E-02	1.983E-02	1.598E-02	1.801E-02	1.733E-02	1.640E-02
V STRESS	-8.864E+00	-9.040E+00	-8.318E+00	-8.529E+00	-8.398E+00	-8.022E+00
R STRESS	-1.026E+00	-1.064E+00	-9.831E-01	-9.840E-01	-9.852E-01	-9.913E-01
DEFORMATION AT MONTH N						
BASE	0.	0.	0.	8.999E-02	1.070E-01	1.006E-01
SUB BASE	0.	0.	0.	6.412E-02	6.867E-02	6.881E-02
SUBGARDE	0.	0.	0.	1.079E-01	1.782E-01	2.000E-01
DEFORMATION DUE TO REPITIONS THROUGH MONTH (N-1)						
BASE	0.	0.	0.	0.	7.877E-02	8.800E-02
SUB BASE	0.	0.	0.	0.	6.298E-02	6.548E-02
SUBGARDE	0.	0.	0.	0.	1.050E-01	1.640E-01
DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1						
BASE	0.	0.	0.	8.999E-02	2.824E-02	1.250E-02
SUB BASE	0.	0.	0.	6.412E-02	5.690E-03	3.334E-03
SUBGARDE	0.	0.	0.	1.079E-01	7.318E-02	3.601E-02
CUMULATIVE DEFORMATION THROUGH MONTH N						
BASE	0.	0.	0.	8.999E-02	1.182E-01	1.308E-01
SUB BASE	0.	0.	0.	6.412E-02	6.981E-02	7.314E-02
SUBGARDE	0.	0.	0.	1.079E-01	1.811E-01	2.171E-01
TOTAL CUMULATIVE DEFORMATION						
BASE	0.	0.	0.	2.620E+01	3.692E+01	4.211E+01
SUB BASE	0.	0.	0.			
SUBGARDE	0.	0.	0.			
RUT DEPTH						

YEAR 2						
	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE
HI-LO-REPT						
BASE	1.690E+04	2.320E+04	1.770E+04	2.190E+04	1.710E+04	3.290E+04
SUB BASE	1.690E+04	2.320E+04	1.770E+04	2.190E+04	1.710E+04	3.290E+04
SUBGARDE	1.690E+04	2.320E+04	1.770E+04	2.190E+04	1.710E+04	3.290E+04
CUMMULATIVE						
BASE	3.660E+04	5.980E+04	7.750E+04	9.940E+04	1.165E+05	1.494E+05
SUB BASE	3.660E+04	5.980E+04	7.750E+04	9.940E+04	1.165E+05	1.494E+05
SUBGARDE	3.660E+04	5.980E+04	7.750E+04	9.940E+04	1.165E+05	1.494E+05
STRAIN TO BEGINNING OF MONTH						
BASE	3.191E+00	3.481E+00	3.661E+00	4.334E+00	5.509E+00	5.950E+00
SUB BASE	1.673E+00	1.741E+00	1.779E+00	1.655E+00	1.803E+00	1.941E+00
SUBGARDE	1.206E+00	1.363E+00	1.464E+00	1.201E+00	1.572E+00	1.895E+00
STRAIN THROUGH MONTH						
BASE	3.377E+00	3.631E+00	3.741E+00	4.427E+00	5.581E+00	6.065E+00
SUB BASE	1.714E+00	1.771E+00	1.795E+00	1.671E+00	1.813E+00	1.956E+00
SUBGARDE	1.306E+00	1.446E+00	1.508E+00	1.238E+00	1.600E+00	1.946E+00
DEPTH						
BASE		3.0				
SUB BASE		4.0				
SUBGARDE						
VERT. DSP.	1.625E-02	1.638E-02	1.642E-02	2.901E-02	2.534E-02	2.298E-02
V STRESS	-7.796E+00	-7.932E+00	-7.972E+00	-7.431E+00	-8.114E+00	-8.675E+00
R STRESS	-9.927E-01	-9.915E-01	-9.914E-01	-9.044E-01	-9.352E-01	-9.948E-01
DEFORMATION AT MONTH N						
BASE	1.013E-01	1.089E-01	1.122E-01	1.328E-01	1.674E-01	1.820E-01
SUB BASE	6.856E-02	7.086E-02	7.182E-02	6.683E-02	7.251E-02	7.823E-02
SUBGARDE	2.090E-01	2.251E-01	2.340E-01	1.982E-01	2.429E-01	2.853E-01
DEFORMATION DUE TO REPITIONS THROUGH MONTH (N-1)						
BASE	9.574E-02	1.044E-01	1.098E-01	1.300E-01	1.653E-01	1.785E-01
SUB BASE	6.701E-02	6.964E-02	7.118E-02	6.622E-02	7.213E-02	7.763E-02
SUBGARDE	1.902E-01	2.123E-01	2.271E-01	1.923E-01	2.386E-01	2.778E-01
DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1						
BASE	9.585E-03	4.520E-03	2.404E-03	2.806E-03	2.159E-03	3.477E-03
SUB BASE	1.548E-03	1.220E-03	6.393E-04	6.122E-04	3.852E-04	5.954E-04
SUBGARDE	1.574E-02	1.284E-02	6.839E-03	5.890E-03	4.337E-03	7.531E-03
CUMULATIVE DEFORMATION THROUGH MONTH N						
BASE	1.394E-01	1.409E-01	1.433E-01	1.461E-01	1.483E-01	1.518E-01
SUB BASE	7.459E-02	7.591E-02	7.655E-02	7.716E-02	7.755E-02	7.814E-02
SUBGARDE	2.329E-01	2.457E-01	2.525E-01	2.584E-01	2.628E-01	2.703E-01
TOTAL CUMULATIVE DEFORMATION						
BASE	4.440E+01	4.625E+01	4.724E+01	4.817E+01	4.886E+01	5.002E+01
SUB BASE						
SUBGARDE						
RUT DEPTH						

YEAR 2	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
HI-LO-REPT						
BASE	3.370E+04	3.990E+04	3.860E+04	3.700E+04	3.770E+04	3.750E+04
SUB BASE	3.370E+04	3.990E+04	3.860E+04	3.700E+04	3.770E+04	3.750E+04
SUBGARDE	3.370E+04	3.990E+04	3.860E+04	3.700E+04	3.770E+04	3.750E+04
CUMMULATIVE						
BASE	1.831E+05	2.230E+05	2.616E+05	2.986E+05	3.363E+05	3.738E+05
SUB BASE	1.831E+05	2.230E+05	2.616E+05	2.986E+05	3.363E+05	3.738E+05
SUBGARDE	1.831E+05	2.230E+05	2.616E+05	2.986E+05	3.363E+05	3.738E+05
STRAIN TO BEGINNING OF MONTH						
BASE	5.705E+00	5.194E+00	2.932E+00	5.739E+00	5.072E+00	4.233E+00
SUB BASE	2.008E+00	2.067E+00	1.978E+00	1.985E+00	1.965E+00	1.893E+00
SUBGARDE	2.059E+00	2.202E+00	9.905E-01	2.021E+00	1.974E+00	1.784E+00
STRAIN THROUGH MONTH						
BASE	5.788E+00	5.266E+00	2.969E+00	5.795E+00	5.116E+00	4.266E+00
SUB BASE	2.020E+00	2.078E+00	1.988E+00	1.993E+00	1.972E+00	1.899E+00
SUBGARDE	2.103E+00	2.246E+00	1.006E+00	2.047E+00	1.996E+00	1.802E+00
DEPTH						
BASE	3.0					
SUB BASE	4.0					
SUBGARDE						
VERT. DSP.	2.108E-02	1.983E-02	1.598E-02	1.801E-02	1.733E-02	1.646E-02
V STRESS	-8.864E+00	-9.040E+00	-6.319E+00	-8.529E+00	-8.398E+00	-8.022E+00
R STRESS	-1.028E+00	-1.064E+00	-9.831E-01	-9.840E-01	-9.858E-01	-9.913E-01
DEFORMATION AT MONTH N						
BASE	1.736E-01	1.580E-01	8.907E-02	1.739E-01	1.535E-01	1.286E-01
SUB BASE	8.078E-02	8.313E-02	6.351E-02	7.970E-02	7.886E-02	7.596E-02
SUBGARDE	3.054E-01	3.238E-01	1.838E-01	3.030E-01	2.986E-01	2.765E-01
DEFORMATION DUE TO REPITIONS THROUGH MONTH (N-1)						
BASE	1.711E-01	1.558E-01	8.797E-02	1.722E-01	1.522E-01	1.270E-01
SUB BASE	8.030E-02	8.267E-02	6.313E-02	7.940E-02	7.859E-02	7.572E-02
SUBGARDE	2.991E-01	3.176E-01	1.810E-01	2.991E-01	2.952E-01	2.756E-01
DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1						
BASE	2.592E-03	2.157E-03	1.102E-03	1.677E-03	1.319E-03	9.843E-04
SUB BASE	4.811E-04	4.609E-04	3.814E-04	3.055E-04	2.729E-04	2.421E-04
SUBGARDE	6.360E-03	6.340E-03	2.810E-03	3.901E-03	3.418E-03	2.817E-03
CUMULATIVE DEFORMATION THROUGH MONTH N						
BASE	1.543E-01	1.565E-01	1.576E-01	1.593E-01	1.606E-01	1.616E-01
SUB BASE	7.862E-02	7.909E-02	7.947E-02	7.977E-02	8.004E-02	8.029E-02
SUBGARDE	2.767E-01	2.830E-01	2.858E-01	2.897E-01	2.931E-01	2.959E-01
TOTAL CUMULATIVE DEFORMATION						
BASE	5.095E-01	5.186E-01	5.229E-01	5.288E-01	5.338E-01	5.378E-01

YEAR 3	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE
HI-LO-REPT						
BASE	3.150E+04	7.340E+04	8.520E+04	7.350E+04	7.400E+04	7.300E+04
SUB BASE	3.150E+04	7.340E+04	8.520E+04	7.350E+04	7.400E+04	7.300E+04
SUBGARDE	3.150E+04	7.340E+04	8.520E+04	7.350E+04	7.400E+04	7.300E+04
CUMMULATIVE						
BASE	4.053E+05	4.787E+05	5.639E+05	6.374E+05	7.114E+05	7.844E+05
SUB BASE	4.053E+05	4.787E+05	5.639E+05	6.374E+05	7.114E+05	7.844E+05
SUBGARDE	4.053E+05	4.787E+05	5.639E+05	6.374E+05	7.114E+05	7.844E+05
STRAIN TO BEGINNING OF MONTH						
BASE	4.076E+00	4.219E+00	4.303E+00	5.080E+00	6.351E+00	6.791E+00
SUB BASE	1.854E+00	1.886E+00	1.903E+00	1.773E+00	1.911E+00	2.044E+00
SUBGARDE	1.681E+00	1.767E+00	1.817E+00	1.495E+00	1.906E+00	2.269E+00
STRAIN THROUGH MONTH						
BASE	4.100E+00	4.270E+00	4.394E+00	5.126E+00	6.401E+00	6.836E+00
SUB BASE	1.859E+00	1.895E+00	1.912E+00	1.780E+00	1.917E+00	2.050E+00
SUBGARDE	1.694E+00	1.795E+00	1.845E+00	1.513E+00	1.926E+00	2.289E+00
DEPTH						
BASE	3.0					
SUB BASE	4.0					
SUBGARDE						
VERT. DSP.	1.625E-02	1.638E-02	1.642E-02	2.901E-02	2.534E-02	2.298E-02
V STRESS	-7.794E+00	-7.932E+00	-7.972E+00	-7.431E+00	-8.114E+00	-8.675E+00
R STRESS	-9.927E-01	-9.915E-01	-9.914E-01	-9.044E-01	-9.352E-01	-9.948E-01
DEFORMATION AT MONTH N						
BASE	1.230E-01	1.281E-01	1.306E-01	1.538E-01	1.920E-01	2.051E-01
SUB BASE	7.436E-02	7.581E-02	7.649E-02	7.118E-02	7.669E-02	8.199E-02
SUBGARDE	2.671E-01	2.795E-01	2.863E-01	2.421E-01	2.923E-01	3.356E-01
DEFORMATION DUE TO REPITIONS THROUGH MONTH (N-1)						
BASE	1.223E-01	1.266E-01	1.291E-01	1.524E-01	1.905E-01	2.037E-01
SUB BASE	7.418E-02	7.543E-02	7.612E-02	7.091E-02	7.645E-02	8.178E-02
SUBGARDE	2.650E-01	2.752E-01	2.820E-01	2.392E-01	2.893E-01	3.326E-01
DEFORMATION AT MONTH N - DEFORMATION AT MONTH N-1						
BASE	7.295E-04	1.532E-03	1.516E-03	1.382E-03	1.494E-03	1.363E-03
SUB BASE	1.846E-04	3.758E-04	3.650E-04	2.712E-04	2.389E-04	2.091E-04
SUBGARDE	2.056E-03	4.354E-03	4.321E-03	2.899E-03	3.001E-03	2.958E-03
CUMULATIVE DEFORMATION THROUGH MONTH N						
BASE	1.623E-01	1.638E-01	1.654E-01	1.667E-01	1.682E-01	1.695E-01
SUB BASE	8.047E-02	8.085E-02	8.121E-02	8.148E-02	8.172E-02	8.193E-02
SUBGARDE	2.980E-01	3.024E-01	3.067E-01	3.096E-01	3.126E-01	3.155E-01
TOTAL CUMULATIVE DEFORMATION						
BASE	5.408E-01	5.471E-01	5.533E-01	5.578E-01	5.625E-01	5.671E-01

VZE3872

YEAR 1	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
MI-LD-REPT						
BASE	6.710E+04	6.860E+04	6.370E+04	7.410E+04	5.480E+04	0.
SUB BASE	6.710E+04	6.860E+04	6.370E+04	7.410E+04	5.480E+04	0.
SUBGARDE	6.710E+04	6.860E+04	6.370E+04	7.410E+04	5.480E+04	0.
CUMMULATIVE						
BASE	8.515E+05	9.201E+05	9.838E+05	1.058E+06	1.113E+06	1.113E+06
SUB BASE	8.515E+05	9.201E+05	9.838E+05	1.058E+06	1.113E+06	1.113E+06
SUBGARDE	8.515E+05	9.201E+05	9.838E+05	1.058E+06	1.113E+06	1.113E+06
STRAIN TC BEGINNING OF MONTH						
BASE	6.406E+00	5.755E+00	3.259E+00	6.299E+00	5.539E+00	4.605E+00
SUB BASE	2.101E+00	2.153E+00	1.659E+00	2.058E+00	2.034E+00	1.959E+00
SUBGARDE	2.416E+00	2.545E+00	1.127E+00	2.265E+00	2.217E+00	1.990E+00
STRAIN THROUGH MONTH						
BASE	6.441E+00	5.783E+00	3.274E+00	6.330E+00	5.958E+00	4.605E+00
SUB BASE	2.106E+00	2.157E+00	1.663E+00	2.062E+00	2.037E+00	1.959E+00
SUBGARDE	2.434E+00	2.562E+00	1.133E+00	2.299E+00	2.226E+00	1.990E+00
DEPTH						
BASE	3.0					
SUB BASE	4.0					
SUBGARDE						
VERT. OSP.	2.108E-02	1.983E-02	1.598E-02	1.801E-02	1.733E-02	1.646E-02
V STRESS	-8.854E+00	-9.040E+00	-6.319E+00	-8.529E+00	-8.398E+00	-8.022E+00
R STRESS	-1.028E+00	-1.064E+00	-9.831E-01	-9.840E-01	-9.852E-01	-9.913E-01
DEFORMATION AT MONTH N						
BASE	1.932E-01	1.735E-01	9.822E-02	1.899E-01	1.667E-01	1.381E-01
SUB BASE	8.423E-02	8.627E-02	6.652E-02	8.248E-02	8.148E-02	7.834E-02
SUBGARDE	3.535E-01	3.695E-01	2.071E-01	3.403E-01	3.330E-01	3.075E-01
DEFORMATION DUE TO REPETITIONS THROUGH MONTH (N-1)						
BASE	1.922E-01	1.726E-01	9.779E-02	1.890E-01	1.662E-01	1.381E-01
SUB BASE	8.406E-02	8.611E-02	6.638E-02	8.233E-02	8.137E-02	7.834E-02
SUBGARDE	3.509E-01	3.670E-01	2.060E-01	3.381E-01	3.316E-01	3.075E-01
DEFORMATION AT MONTH N = DEFORMATION AT MONTH N-1						
BASE	1.046E-03	8.479E-04	4.622E-04	9.206E-04	5.602E-04	0.
SUB BASE	1.738E-04	1.624E-04	1.443E-04	1.511E-04	1.046E-04	0.
SUBGARDE	2.567E-03	2.492E-03	1.178E-03	2.141E-03	1.452E-03	0.
CUMULATIVE DEFORMATION THROUGH MONTH N						
BASE	1.706E+01	1.715E+01	1.720E+01	1.729E+01	1.734E+01	1.734E+01
SUB BASE	8.211E+02	8.227E+02	8.241E+02	8.256E+02	8.267E+02	8.267E+02
SUBGARDE	3.181E+01	3.206E+01	3.218E+01	3.239E+01	3.254E+01	3.254E+01
TOTAL CUMULATIVE DEFORMATION						
RUT DEPTH	5.709E-01	5.744E-01	5.761E-01	5.794E-01	5.815E-01	5.815E-01

16.02.50 CRK:107,117000,77.CEOC015+.JAIN.  
 16.02.50 BLOWUP(CRKDEX)  
 16.02.50 RUN (S)  
 16.48.22 CTIME 012.100 SEC. RUN\* LEVEL 60\*  
 16.48.22 LGO.  
 16.48.30 LOADER UNUSED STORAGE 024+20.  
 16.48.30 = 72000CM 8,569CP 9+5MS DMT  
 16.52.10 END = CRKDEX  
 16.52.11 MS 1231 PRU.  
 16.52.11 CP 58.072 SEC.  
 16.52.11 PP 28.178 SEC.  
 16.52.11 TM 62.996 SEC. 77 (OCTAL)

APPENDIX 4.6

REGRESSION ANALYSIS FOR CRACKING-PATCHING VERSUS ROUGHNESS INDEX

STEP01 - STEPWISE REGRESSION - VERSION OF OCT. 15, 1968  
 THE UNIVERSITY OF TEXAS CENTER FOR HIGHWAY RESEARCH

REGRESSION EQUATION FOR LSVSV1 AND LCP

PROBLEM CODE	LRSVLC
NUMBER OF CASES	95
NUMBER OF ORIGINAL VARIABLES	7
NUMBER OF VARIABLES ADDED	3
TOTAL NUMBER OF VARIABLES	10
NUMBER OF SUB-PROBLEMS	1

INPUT DATA

PROBLEM CARD  
 ROBLM LRSVLC 95 7 3 3 1 1 10 YES YES YES 1

TRANSGENERATION CARDS  
 TRNGEN 022 1 0.000  
 TRNGEN 910 2 3.000  
 TRNGEN 1010 2 2.000

LABEL CARD  
 LAE 1LRSVSV 2 LCP 3 01 4 D2 5 03 6 WT 7 NXL  
 LAE BARCTAN 9 CBLCP 10 SGLCP -0 -0 -0 -0

VARIABLE FORMAT CARD  
 SF7.1,28X,2F7.1)

0.	0.	3.0000000E+00	0.	8.0000000E+00	3.2000000E+01	2.0000000E+00
3.0000000E-01	2.0000000E+00	3.0000000E+00	0.	2.0000000E+00	3.2000000E+01	2.0000000E+00
2.4500000E-01	2.4300000E+00	3.0000000E+00	0.	2.0000000E+00	3.2000000E+01	2.0000000E+00
0.	0.	4.0000000E+00	0.	1.2000000E+01	3.2000000E+01	2.0000000E+00
6.1500000E-01	2.1200000E+00	4.0000000E+00	0.	1.2000000E+01	3.2000000E+01	2.0000000E+00
0.	0.	3.0000000E+00	3.0000000E+00	2.0000000E+00	3.2000000E+01	2.0000000E+00
5.9000000E-01	2.2600000E+00	3.0000000E+00	3.0000000E+00	2.0000000E+00	3.2000000E+01	2.0000000E+00
0.	0.	4.0000000E+00	3.0000000E+00	4.0000000E+00	1.8000000E+01	1.0000000E+00
5.9000000E-01	2.4000000E+00	4.0000000E+00	3.0000000E+00	4.0000000E+00	1.8000000E+01	1.0000000E+00
0.	0.	3.0000000E+00	0.	2.0000000E+00	1.8000000E+01	1.0000000E+00
3.5000000E-01	2.0000000E+00	3.0000000E+00	0.	2.0000000E+00	1.8000000E+01	1.0000000E+00
6.0000000E-01	2.1400000E+00	3.0000000E+00	0.	2.0000000E+00	1.8000000E+01	1.0000000E+00
0.	0.	2.0000000E+00	3.0000000E+00	2.0000000E+00	2.4000000E+01	2.0000000E+00
6.2000000E-01	2.2500000E+00	2.0000000E+00	3.0000000E+00	2.0000000E+00	2.4000000E+01	2.0000000E+00
5.5000000E-01	2.2500000E+00	2.0000000E+00	3.0000000E+00	2.0000000E+00	2.4000000E+01	2.0000000E+00
0.	0.	3.0000000E+00	6.0000000E+00	4.0000000E+00	2.4000000E+01	2.0000000E+00
3.2000000E-01	1.2100000E+00	3.0000000E+00	6.0000000E+00	4.0000000E+00	2.4000000E+01	2.0000000E+00
0.	0.	2.0000000E+00	0.	4.0000000E+00	2.0000000E+00	1.0000000E+00
3.6000000E-01	1.1100000E+00	2.0000000E+00	0.	4.0000000E+00	2.0000000E+00	1.0000000E+00
5.1000000E-01	2.0000000E+00	2.0000000E+00	0.	4.0000000E+00	2.0000000E+00	1.0000000E+00
6.2000000E-01	2.1000000E+00	2.0000000E+00	0.	4.0000000E+00	2.0000000E+00	1.0000000E+00
0.	0.	1.0000000E+00	3.0000000E+00	0.	2.0000000E+00	1.0000000E+00
1.7500000E-01	1.3200000E+00	1.0000000E+00	3.0000000E+00	0.	2.0000000E+00	1.0000000E+00
4.1000000E-01	2.0000000E+00	1.0000000E+00	3.0000000E+00	0.	2.0000000E+00	1.0000000E+00
0.	0.	1.0000000E+00	3.0000000E+00	4.0000000E+00	6.0000000E+00	1.0000000E+00
2.5000000E-01	2.0800000E+00	1.0000000E+00	3.0000000E+00	4.0000000E+00	6.0000000E+00	1.0000000E+00
4.1000000E-01	2.1600000E+00	1.0000000E+00	3.0000000E+00	4.0000000E+00	6.0000000E+00	1.0000000E+00
4.4000000E-01	2.2300000E+00	1.0000000E+00	3.0000000E+00	4.0000000E+00	6.0000000E+00	1.0000000E+00
0.	0.	4.0000000E+00	3.0000000E+00	4.0000000E+00	1.2000000E+01	1.0000000E+00
4.6000000E-01	2.2600000E+00	4.0000000E+00	3.0000000E+00	4.0000000E+00	1.2000000E+01	1.0000000E+00
5.9000000E-01	2.3600000E+00	4.0000000E+00	3.0000000E+00	4.0000000E+00	1.2000000E+01	1.0000000E+00
0.	0.	1.0000000E+00	6.0000000E+00	4.0000000E+00	6.0000000E+00	1.0000000E+00





9.90000000E-01 3.00000000E+00 3.00000000E+00 6.00000000E+00 1.20000000E+01 3.20000000E+01 2.00000000E+00

VARIABLE	MEAN	STANDARD DEVIATION
LRSV 1	.3702E	.29609
LCP 2	1.6675E	1.14127
D1 3	2.76842	1.04630
D2 4	3.25263	2.1731E
D3 5	4.75789	3.96988
4T 6	14.58947	10.05463
NXL 7	1.2315E	.4240E
ARCTAN 8	.33184	.25355
CBLCP 9	10.18614	8.66436
SGLCP 10	4.06961	3.09206

COVARIANCE MATRIX

VARIABLE NUMBER	1	2	3	4	5	6	7	8	9	10
1	.088									
2	.313	.088								
3	1.303	.061	.022							
4		1.095	.022	.121						
5			.321	.175	.177					
6			.729	1.028	5.542	-.020				
7			4.723	1.977	4.764	.075	.004			
8				15.760	23.974	.069	.075	.075	2.421	.473
9					101.096	.674	.138	.275	9.132	3.436
10						3.649	.112	.019	.945	.281
							.180	.001	3.253	1.070
								.064	6.154	1.796
									7.291	1.509
									2.071	.756
									75.071	26.421
										9.561

CORRELATION MATRIX

VARIABLE NUMBER	1	2	3	4	5	6	7	8	9	10
1	1.000	.926	.072	.187	.149	.059	.028	.996	.944	.953
2		1.000	.051	.129	.092	-.006	-.042	.950	.923	.974
3			1.000	.321	.248	.527	.170	.071	.104	.087
4				1.000	.229	.218	.074	.180	.173	.159
5					1.000	.601	.400	.137	.179	.146
6						1.000	.856	.844	.084	.049
7							1.000	.011	.013	-.010
8								1.000	.943	.964
9									1.000	.986
10										1.000

SUE PROBLEM CARD

UBFR0 8 -0 -0 -0 -0 YES YES YES

SLB-PROBLM 1  
 DEPENDENT VARIABLE 8  
 MAXIMUM NUMBER OF STEPS 20  
 F-LEVEL FOR INCLUSION .010000  
 F-LEVEL FOR DELETION .005000  
 TOLERANCE LEVEL .001000

CONTROL-DELETE CARDS  
 ONCE1\*\*\*\*\*

STEP NUMBER 1  
 VARIABLE ENTERED 10

MULTIPLE R .9638  
 STD. ERROR FOR RESIDUALS .0680

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	1	5.613	5.613	1214.618
RESIDUAL	93	.430	.005	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	.01021						
SULCP 10	.07903	.00227	1214.6179	LRSV 1	.96203	.0909	1142.9149
				LCP 2	.19048	.0520	3.4638
				D1 3	-.04650	.9924	.1994
				D2 4	.10066	.9747	.9417
				D3 5	-.01351	.9786	.0168
				WT 6	-.01131	.9976	.0118
				NXL 7	.07872	.9999	.5737
				CRLCP 9	-.17712	.0274	2.9796

STEP NUMBER 2  
 VARIABLE ENTERED 2

MULTIPLE R .9651  
 STD. ERROR FOR RESIDUALS .0671

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	2	5.629	2.815	625.130
RESIDUAL	92	.414	.005	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER

(CONSTANT	.00006 )							
LCP 2	.04949	.02659	3,4638		LRSVSV 1	.98693	.0908	3412.3257
SQLCP 10	.06125	.00982	38,9363		D1 3	-.01876	.9705	.0320
					D2 4	.12551	.9620	1.4564
					D3 5	.03079	.9288	.0863
					WT 6	.03446	.9439	.1082
					NXL 7	.10878	.9798	1.0898
					CRLCP 9	.03525	.0015	.1132

STEP NUMBER 3  
VARIABLE ENTERED 4

MULTIPLE R .9657  
STD. ERROR FOR RESIDUALS .0669

ANALYSIS OF VARIANCE

REGRESSION	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
	3	5.636	1.879	419.306
RESIDUAL	91	.448	.004	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.01163 )			LRSVSV 1	.98677	.0895	3333.9556
LCP 2	.05317	.02670	3.9653	D1 3	-.09966	.8830	.3215
OP 4	.00391	.00324	1.4544	D3 5	.00698	.8949	.0044
SQLCP 10	.05949	.00990	36.1131	WT 6	.01048	.9086	.0099
				NXL 7	.10203	.9761	.9467
				CRLCP 9	.04363	.0015	.1716

STEP NUMBER 4  
VARIABLE ENTERED 7

MULTIPLE R .9660  
STD. ERROR FOR RESIDUALS .0670

ANALYSIS OF VARIANCE

REGRESSION	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
	4	5.640	1.410	314.532
RESIDUAL	90	.403	.004	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.03165 )			LRSVSV 1	.98671	.0883	3282.1747
LCP 2	.05669	.02695	4.4238	D1 3	-.07531	.8651	.5477
OP 4	.00371	.00325	1.3088	D3 5	-.03504	.7630	.1094
NXL 7	.01604	.01548	.9467				

SQLCP 10	.05827	.00998	34.0788	.	WT 6	-.15379	.2331	2.1559
				.	CRLCP 9	.04256	.0015	.1615

STEP NUMBER 5  
VARIABLE ENTERED 6

MULTIPLE R .9669  
STD. ERROR FOR RESIDUALS .0665

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	5	5.649	1.130	255.289
RESIDUAL	89	.394	.004	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.05548						
LCP 2	.04881	.02731	3.1938	LRSVSV 1	.98650	.0857	3192.6650
D2 4	.00505	.00335	2.2730	D1 3	.05140	.4177	.2331
WT 6	-.00208	.00141	2.1559	D3 5	.04785	.5761	.2019
NXL 7	.05697	.03233	3.1046	CRLCP 9	.03118	.0015	.0857
SQLCP 10	.06133	.01013	36.6210				

STEP NUMBER 6  
VARIABLE ENTERED 3

MULTIPLE R .9669  
STD. ERROR FOR RESIDUALS .0668

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	6	5.650	.942	210.946
RESIDUAL	88	.393	.004	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.07307						
LCP 2	.04801	.02748	3.0525	LRSVSV 1	.98662	.0856	3187.0087
D1 3	.00442	.01019	.2331	D3 5	.07365	.4894	.4745
D2 4	.00402	.00340	2.0048	CRLCP 9	.03711	.0015	.1200
WT 6	-.00278	.00204	1.8578				
NXL 7	.06932	.04134	2.8119				
SQLCP 10	.06103	.01020	36.5246				

STEP NUMBER 7

VARIABLE ENTERED 5

MULTIPLE R .9671  
STD. ERROR FOR RESIDUALS .0670

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	7	5.653	.808	179.799
RESIDUAL	87	.391	.004	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.08955						
LCP 2	.04922	.02762	3.1756	LRSVSV 1	.98661	.0853	3147.2078
D1 3	.00788	.01109	.5054	CRCLCP 9	.03998	.0015	.1377
D2 4	.00455	.00343	1.7548				
D3 5	.00171	.00249	.4745				
WT 6	-.00382	.00254	2.2602				
NXL 7	.08282	.04586	3.2613				
SQLCP 10	.06100	.01027	35.2851				

STEP NUMBER 8  
VARIABLE ENTERED 9

MULTIPLE R .9672  
STD. ERROR FOR RESIDUALS .0674

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE	F RATIO
REGRESSION	8	5.653	.707	155.782
RESIDUAL	86	.390	.005	

VARIABLES IN EQUATION				VARIABLES NOT IN EQUATION			
VARIABLE	COEFFICIENT	STD. ERROR	F TO REMOVE	VARIABLE	PARTIAL CORR.	TOLERANCE	F TO ENTER
(CONSTANT	-.09136						
LCP 2	.09108	.11616	.6147	LRSVSV 1	.99154	.0836	4958.9820
D1 3	.00837	.01122	.5559				
D2 4	.00458	.00345	1.7567				
D3 5	.00175	.00250	.4882				
WT 6	-.00386	.00255	2.2812				
NXL 7	.08325	.04610	3.2610				
CRCLCP 9	.00778	.02096	.1377				
SQLCP 10	.02445	.09904	.0609				

F-LEVEL OR TOLERANCE INSUFFICIENT FOR FURTHER COMPUTATION



SUMMARY TABLE

STEP NUMBER	VARIABLE		MULTIPLE		INCREASE IN RSQ	F VALUE TO ENTER OR REMOVE	NUMBER OF INDEPENDENT VARIABLES INCLUDED
	ENTERED	REMOVED	R	RSQ			
1	SGLCP	10	.9638	.9289	.9289	1214.6179	1
2	LCP	2	.9651	.9315	.0026	3.4638	2
3	D2	4	.9657	.9325	.0011	1.4564	3
4	NXL	7	.9660	.9332	.0007	.9467	4
5	WT	6	.9669	.9348	.0016	2.1559	5
6	D1	3	.9669	.9350	.0002	.2331	6
7	D3	5	.9671	.9353	.0004	.4745	7
8	CBLCP	9	.9672	.9354	.0001	.1377	8

LIST OF Y-VALUES, Y-ESTIMATES, AND RESIDUALS

CASE	Y-VALUE	Y-ESTIMATE	RESIDUAL
1	0.	-9.22772784E-03	9.22772784E-03
2	2.91456794E-01	3.32929214E-01	-4.14724196E-02
3	7.01584135E-01	6.63127003E-01	3.84571317E-02
4	0.	6.13447292E-03	-6.13487292E-03
5	5.51375965E-01	5.91236458E-01	-3.98604930E-02
6	0.	4.49928191E-03	-4.49928191E-03
7	5.33034110E-01	5.28349612E-01	4.68449846E-03
8	0.	-2.33655304E-02	2.33655304E-02
9	5.33034110E-01	6.36033369E-01	-1.02999258E-01
10	0.	-3.84628748E-02	3.84628748E-02
11	3.36644819E-01	3.03694067E-01	3.29807522E-02
12	5.46415500E-01	5.54607675E-01	-1.41881752E-02
13	0.	2.70088722E-02	-2.70088722E-02
14	5.97176058E-01	5.84742465E-01	1.24141928E-02
15	7.55762755E-01	7.08085558E-01	5.16771973E-02
16	0.	4.20982166E-02	-4.20982166E-02
17	4.09702745E-01	3.59428288E-01	-4.97253438E-02
18	0.	7.91064046E-03	-7.91064046E-03
19	3.45555581E-01	2.74022929E-01	7.15326512E-02
20	4.71615568E-01	3.50047582E-01	1.21547985E-01
21	5.97176058E-01	3.79005580E-01	2.18171072E-01
22	0.	6.27504945E-03	-6.27504945E-03
23	1.73245066E-01	1.86978397E-01	-1.37327302E-02
24	3.89097231E-01	3.48431991E-01	4.06652397E-02
25	0.	-2.16284648E-03	2.16284648E-03
26	2.44978063E-01	3.63028480E-01	-1.1849817E-01
27	3.85097231E-01	3.86996982E-01	-2.10024871E-03
28	4.14506875E-01	4.08754472E-01	5.75240248E-03
29	0.	-2.14486885E-04	2.14486885E-04
30	4.31138741E-01	4.20255970E-01	1.08827706E-02
31	5.33034110E-01	5.23635843E-01	9.39826725E-03
32	0.	1.15641633E-02	-1.15641633E-02
33	6.36577757E-01	6.66681358E-01	-3.61036048E-02
34	0.	1.32711825E-02	-1.32711825E-02
35	2.91456794E-01	4.33741640E-01	-1.42284845E-01
36	4.14506875E-01	5.29772064E-01	-1.15265190E-01
37	0.	-2.93040974E-02	2.93040974E-02
38	3.97627992E-01	5.09447526E-01	-1.11819536E-01
39	0.	6.20362122E-03	-6.20362122E-03
40	1.68340157E-01	1.86946498E-01	-1.85168112E-02
41	3.65745850E-01	2.72315910E-01	9.34499461E-02
42	4.31138741E-01	5.44955247E-01	-1.13816506E-01
43	4.85013570E-01	6.15424581E-01	-1.30411011E-01
44	5.79882132E-01	6.19586722E-01	-3.46245898E-02
45	6.24023053E-01	6.23606772E-01	4.16280936E-04
46	0.	-9.95128920E-03	9.95128920E-03
47	2.98779988E-01	3.43607574E-01	-4.48275855E-02
48	5.53549764E-01	5.99269671E-01	-4.57199067E-02
49	6.7474042E-01	6.28222370E-01	4.65185602E-02
50	0.	6.51638980E-03	-6.51638980E-03
51	3.64843489E-01	3.23846462E-01	4.10470274E-02
52	4.85013570E-01	5.01369240E-01	-1.63556701E-02
53	5.79882132E-01	5.52820609E-01	-2.70615233E-02
54	6.24023053E-01	5.68131550E-01	5.58914967E-02
55	0.	-7.92511851E-04	7.92511851E-04
56	4.06048048E-01	4.32631777E-01	-2.65337203E-02
57	5.10448322E-01	4.45835567E-01	6.4427544E-02
58	6.37070329E-01	4.66116450E-01	1.70953880E-01

59	7.04494064E-01	4.94060344E-01	2.10433720E-01
60	7.70170914E-01	7.02444345E-01	6.77265686E-02
61	0.	-2.23079643E-02	2.23079643E-02
62	3.44670029E-01	3.98162493E-01	-5.34924632E-02
63	0.	-6.15305391E-03	6.15305391E-03
64	2.70224043E-01	3.44941892E-01	-9.47172491E-02
65	2.65943158E-01	3.95349780E-01	-1.09405828E-01
66	0.	-3.76705651E-02	3.76705651E-02
67	2.26068388E-01	1.78033250E-01	4.80351305E-02
68	0.	7.57395584E-03	-7.57395584E-03
69	1.03627460E-01	1.88277303E-01	-8.46498430E-02
70	2.73008703E-01	3.22215120E-01	-4.92064178E-02
71	4.63647609E-01	4.40998246E-01	2.26493627E-02
72	5.33034110E-01	5.31424280E-01	1.60982452E-03
73	0.	-1.27205293E-03	1.27205293E-03
74	4.95133263E-01	5.75930895E-01	-8.07976315E-02
75	7.04494064E-01	6.36901612E-01	6.75924520E-02
76	0.	-1.10088553E-02	1.10088553E-02
77	2.91456794E-01	2.90393172E-01	1.06362234E-03
78	6.33982978E-01	5.35245364E-01	6.86876142E-02
79	0.	4.35374552E-03	-4.35374552E-03
80	5.10488322E-01	5.24520957E-01	-1.40326349E-02
81	6.12736551E-01	5.89455330E-01	2.32812205E-02
82	5.35946055E-01	5.43105371E-01	-7.10931559E-03
83	5.7358129E-01	5.65968912E-01	7.58921628E-03
84	6.3057757E-01	6.05463963E-01	2.51137942E-02
85	2.54368059E-01	3.46510687E-01	-9.21426289E-02
86	0.	1.95966263E-02	-1.95966263E-02
87	4.31138741E-01	4.14897336E-01	1.62414044E-02
88	5.33034110E-01	4.86505588E-01	4.65285224E-02
89	0.	-2.49685116E-03	2.49685116E-03
90	2.65575001E-01	2.98905176E-01	-3.33301756E-02
91	4.89697775E-01	5.90573699E-01	-1.00875924E-01
92	0.	2.52224247E-02	-2.52224247E-02
93	5.14286541E-01	4.65217178E-01	4.90693628E-02
94	6.56178718E-01	5.49072759E-01	1.07105963E-01
95	7.80373080E-01	7.28459284E-01	5.19137980E-02

FINISH CARD ENCOUNTERED  
PROGRAM TERMINATED

APPENDIX 4.7

COMPUTER PROGRAM AND CALCULATED VALUES OF ROUGHNESS  
INDEX AND PRESENT SERVICEABILITY INDEX

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PROGRAM PSI (INPUT, OUTPUT, TAPES=INPUT)
PRINT 60
50 FORMAT (1H) 5X, 36HCRK THICKNESSES AXLE NO. ROUGH
* 33HROUGH RUT CALC SECTION MONTH /
* 4X, 43HINDEX AC B SB WT AXLE IND-I IND-T
* 11H DEPTH PSI // )
20 CONTINUE
HEAD (5,30) CI, GO, UT, DTM, WT, NXL, SVX, ROI, NSECT, MONTH
30 FORMAT (5F4.1, 18, 2F8.0, 2A8 )
40 CONTINUE
*
* CI CRACKING INDEX
* GO THICKNESS OF ASPHALT CONCRATE
* UT THICKNESS OF BASE
* DTM THICKNESS OF SUBBASE
* WT AXLE LOAD
* NXL NUMBER OF AXLE
* 1 = SINGLE AXLE
* 2 = TANDEM AXLE
* SVX ROUGHNESS INDEX -- LOG (1+SLOPE VARIANCE) OF A
SECTION JUST AFTER CONSTRUCTION OR AT THE BEGINNING
OF THE ANALYSIS PERIOD = ROUGH IND-I.
* ROI RUT DEPTH INDEX
* NSECT SECTION NUMBER OF REFERENCE TO
* MONTH MONTH FOR WHICH PSI IS CALCULATED
*
SVZ = 10.0**SVX - 1.0
SVZ SLOPE VARIANCE OF THE SECTION AT THE BEGINNING OF THE
ANALYSIS PERIOD
*
B = -.09136 + 0.09108*ALOG10(1.0+CI) + 0.02445*
ALOG10(1.0+CI)**2.0 + 0.00778*ALOG10(1.0+CI)
**3.0 + 0.00837*GO + 0.00458*DT + 0.00175*
DTM - 0.00386*WT + 0.00325*NXL
*
SV = (10.0**TAN(B) - 1.0 + SVZ**0.5) ** 2.0
CALCULATED VALUE OF SLOPE VARIANCE AT ANY TIME
AFTER BEGINNING OF ANALYSIS PERIOD
*
SVL = ALOG10(1.0+SV)
CALCULATED VALUE OF ROUGHNESS INDEX-- LOG(1+SV) AT
ANY TIME AFTER BEGINNING OF ANALYSIS PERIOD =ROUGH IND-T.
*
PSIC = 5.03 - 1.91*ALOG10(1.0+SV) - 1.3R*ROI**2.0
- 0.01*CI**0.3
PSIC CALCULATED VALUE OF PSI
*
PRINT 50, CI, GO, UT, DTM, WT, NXL, SVX, SVL, ROI, PSIC,
NSECT, MONTH
50 FORMAT (5X, F3.0, 3F5.2, F7.1, 17, F6.2, F7.2, F6.2, F7.2,
A8, 1X, A8)
60 TO 20
90 CONTINUE
END

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CRK INDEX	THICKNESSES AC	B	SB	AXLE NO. WT	AXLE	ROUGH IND-I	ROUGH IND-T	RUT DEPTH	CALC PSI	SECTION	MONTH
0	4.00	3.00	4.00	18.0	1	.30	.28	0.00	4.50	627	R
40	4.00	3.00	4.00	18.0	1	.30	.58	.12	3.84	627	FEB
70	4.00	3.00	4.00	18.0	1	.30	.67	.13	3.64	627	MAR
230	4.00	3.00	4.00	18.0	1	.30	.97	.13	3.00	627	APR
0	3.00	6.00	8.00	18.0	1	.30	.29	0.00	4.48	623	R
90	3.00	6.00	8.00	18.0	1	.30	.75	.33	3.36	623	FEB
160	3.00	6.00	8.00	18.0	1	.30	.89	.34	3.05	623	MAR
400	3.00	6.00	8.00	18.0	1	.30	1.20	.35	2.37	623	APR
600	3.00	6.00	8.00	18.0	1	.30	1.38	.36	1.97	623	MAY
0	3.00	0.00	8.00	18.0	1	.60	.57	0.00	3.95	607	R
70	3.00	0.00	8.00	18.0	1	.60	.89	.15	3.22	607	DEC
270	3.00	0.00	8.00	18.0	1	.60	1.16	.16	2.61	607	FEB
350	3.00	0.00	8.00	18.0	1	.60	1.24	.16	2.44	607	MAR
670	3.00	0.00	8.00	18.0	1	.60	1.48	.65	1.36	607	APR
0	4.00	6.00	12.00	18.0	1	.50	.50	0.00	4.07	625	R
135	4.00	6.00	12.00	18.0	1	.50	1.00	.31	2.86	625	APR-1
290	4.00	6.00	12.00	18.0	1	.50	1.21	.32	2.40	625	MAY-1
790	4.00	6.00	12.00	18.0	1	.50	1.64	.33	1.47	625	JUL
940	4.00	6.00	12.00	18.0	1	.50	1.74	.34	1.25	625	DEC
940	4.00	6.00	12.00	18.0	1	.50	1.74	.53	1.02	625	MAR-2
960	4.00	6.00	12.00	18.0	1	.50	1.75	.53	.99	625	APR-2
970	4.00	6.00	12.00	18.0	1	.50	1.76	.53	.98	625	MAY-2
980	4.00	6.00	12.00	18.0	1	.50	1.76	.54	.95	625	JUN-2
985	4.00	6.00	12.00	18.0	1	.50	1.76	.54	.94	625	JUL-2
990	4.00	6.00	12.00	18.0	1	.50	1.77	.54	.94	625	NOV-2
0	5.00	6.00	4.00	18.0	1	.50	.50	0.00	4.08	615	R
40	5.00	6.00	4.00	18.0	1	.50	.78	.17	3.44	615	APR-1
120	5.00	6.00	4.00	18.0	1	.50	.97	.16	3.04	615	MAY-1
640	5.00	6.00	4.00	18.0	1	.50	1.51	.17	1.84	615	JUL
860	5.00	6.00	4.00	18.0	1	.50	1.67	.30	1.43	615	DEC
880	5.00	6.00	4.00	18.0	1	.50	1.68	.37	1.33	615	MAR-2
500	5.00	6.00	4.00	18.0	1	.50	1.69	.38	1.30	615	APR-2
0	5.00	0.00	4.00	18.0	1	.20	.20	0.00	4.65	629	R
10	5.00	0.00	4.00	18.0	1	.20	.35	.16	4.29	629	MAR-1
140	5.00	0.00	4.00	18.0	1	.20	.80	.16	3.35	629	APR-1
860	5.00	0.00	4.00	18.0	1	.20	1.57	.35	1.56	629	DEC-2
870	5.00	0.00	4.00	18.0	1	.20	1.58	.36	1.54	629	FEB-2
880	5.00	0.00	4.00	18.0	1	.20	1.59	.36	1.52	629	MAR-2
900	5.00	0.00	4.00	18.0	1	.20	1.60	.37	1.48	629	APR-2
0	2.00	3.00	8.00	12.0	1	.90	.89	0.00	3.32	159	R
250	2.00	3.00	8.00	12.0	1	.90	1.37	.33	2.11	159	FEB
340	2.00	3.00	8.00	12.0	1	.90	1.45	.34	1.92	159	MAR
670	2.00	3.00	8.00	12.0	1	.90	1.67	.35	1.41	159	APR
830	2.00	3.00	8.00	12.0	1	.90	1.76	.35	1.20	159	MAY
0	3.00	0.00	4.00	12.0	1	.50	.48	0.00	4.11	163	R
190	3.00	0.00	4.00	12.0	1	.50	1.04	.08	2.90	163	MAR
450	3.00	0.00	4.00	12.0	1	.50	1.31	.09	2.30	163	APR
700	3.00	0.00	4.00	12.0	1	.50	1.50	.10	1.88	163	MAY
0	4.00	3.00	4.00	12.0	1	.70	.70	0.00	3.69	151	R
80	4.00	3.00	4.00	12.0	1	.70	1.03	.27	2.87	151	APR
180	4.00	3.00	4.00	12.0	1	.70	1.19	.27	2.53	151	MAY
320	4.00	3.00	4.00	12.0	1	.70	1.51	.28	1.82	151	JUN
0	4.00	6.00	0.00	12.0	1	.70	.71	0.00	3.68	161	R
50	4.00	6.00	0.00	12.0	1	.70	.97	.26	3.00	161	APR
150	4.00	6.00	0.00	12.0	1	.70	1.16	.27	2.59	161	MAY
450	4.00	6.00	0.00	12.0	1	.70	1.47	.28	1.91	161	JUN
660	4.00	6.00	0.00	12.0	1	.70	1.62	.29	1.56	161	JUL
800	4.00	6.00	0.00	12.0	1	.70	1.72	.30	1.34	161	AUG



983	4.50	9.00	4.00	18.0	1	.43	1.76	.26	1.27	EXMP	AUG
989	4.50	9.00	4.00	18.0	1	.43	1.76	.26	1.26	EXMP	OCT
1	4.50	9.00	4.00	18.0	1	.43	.44	0.00	4.19	EXMP	JULY
67	4.50	9.00	4.00	18.0	1	.43	.82	.25	3.30	EXMP	SEP
176	4.50	9.00	4.00	18.0	1	.43	1.03	.26	2.85	EXMP	NOV
199	4.50	9.00	4.00	18.0	1	.43	1.07	.28	2.74	EXMP	JAN
239	4.50	9.00	4.00	18.0	1	.43	1.12	.29	2.61	EXMP	MAR
520	4.50	9.00	4.00	18.0	1	.43	1.42	.29	1.98	EXMP	MAY
803	4.50	9.00	4.00	18.0	1	.43	1.04	.30	1.50	EXMP	JULY
895	4.50	9.00	4.00	18.0	1	.43	1.70	.30	1.36	EXMP	SEP
915	4.50	9.00	4.00	18.0	1	.43	1.71	.30	1.33	EXMP	NOV
923	4.50	9.00	4.00	18.0	1	.43	1.72	.31	1.31	EXMP	JAN
929	4.50	9.00	4.00	18.0	1	.43	1.72	.31	1.30	EXMP	MAR
955	4.50	9.00	4.00	18.0	1	.43	1.74	.32	1.26	EXMP	MAY
981	4.50	9.00	4.00	18.0	1	.43	1.76	.32	1.22	EXMP	JULY
0	3.50	9.00	8.00	18.0	1	.43	.44	0.00	4.20	EXMP	OCT
3	3.50	9.00	8.00	18.0	1	.43	.50	.32	3.91	EXMP	DEC
26	3.50	9.00	8.00	18.0	1	.43	.88	.34	3.53	EXMP	FEB
185	3.50	9.00	8.00	18.0	1	.43	1.05	.36	2.72	EXMP	APR
665	3.50	9.00	8.00	18.0	1	.43	1.53	.36	1.67	EXMP	JUN
891	3.50	9.00	8.00	18.0	1	.43	1.69	.38	1.30	EXMP	AUG
932	3.50	9.00	8.00	18.0	1	.43	1.72	.39	1.23	EXMP	OCT
942	3.50	9.00	8.00	18.0	1	.43	1.73	.40	1.21	EXMP	DEC
948	3.50	9.00	8.00	18.0	1	.43	1.73	.40	1.20	EXMP	FEB
965	3.50	9.00	8.00	18.0	1	.43	1.74	.41	1.16	EXMP	APR
988	3.50	9.00	8.00	18.0	1	.43	1.76	.41	1.13	EXMP	JUN
995	3.50	9.00	8.00	18.0	1	.43	1.76	.42	1.11	EXMP	AUG
997	3.50	9.00	8.00	18.0	1	.43	1.76	.43	1.09	EXMP	OCT
0	5.00	6.00	12.00	30.0	1	.30	.27	0.00	4.52	305	B
50	5.00	6.00	12.00	30.0	1	.30	.80	.33	3.67	305	MAR
400	5.00	6.00	12.00	30.0	1	.30	1.14	.34	2.49	305	MAY
730	5.00	6.00	12.00	30.0	1	.30	1.48	.60	1.56	305	JUN
860	5.00	6.00	12.00	30.0	1	.30	1.51	.63	1.31	305	JULY
960	5.00	6.00	12.00	30.0	1	.30	1.57	.65	1.14	305	JAN
0	5.00	6.00	12.00	30.0	1	.25	.22	0.00	4.61	307	B
50	5.00	6.00	12.00	30.0	1	.25	.55	.33	3.75	307	MAR
400	5.00	6.00	12.00	30.0	1	.25	1.12	.34	2.54	307	MAY
730	5.00	6.00	12.00	30.0	1	.25	1.40	.60	1.60	307	JUN
860	5.00	6.00	12.00	30.0	1	.25	1.49	.63	1.35	307	JULY
960	5.00	6.00	12.00	30.0	1	.25	1.55	.65	1.17	307	JAN
0	4.00	6.00	12.00	30.0	1	.50	.46	0.00	4.15	323	B
50	4.00	6.00	12.00	30.0	1	.50	.75	.40	3.30	323	JAN
110	4.00	6.00	12.00	30.0	1	.50	.88	.41	3.01	323	FEB
160	4.00	6.00	12.00	30.0	1	.50	.96	.42	2.82	323	MAR
440	4.00	6.00	12.00	30.0	1	.50	1.26	.43	2.16	323	APR
870	4.00	6.00	12.00	30.0	1	.50	1.56	.44	1.49	323	JUN
0	6.00	9.00	12.00	30.0	1	.30	.29	0.00	4.48	311	B
10	6.00	9.00	12.00	30.0	1	.30	.44	.34	4.01	311	MAR
150	6.00	9.00	12.00	30.0	1	.30	.87	.35	3.08	311	MAY
470	6.00	9.00	12.00	30.0	1	.30	1.26	.36	2.22	311	JUN
670	6.00	9.00	12.00	30.0	1	.30	1.43	.36	1.85	311	JULY
890	6.00	9.00	12.00	30.0	1	.30	1.59	.37	1.50	311	JAN
920	6.00	9.00	12.00	30.0	1	.30	1.61	.60	1.15	311	APR
1000	6.00	9.00	12.00	30.0	1	.30	1.67	.61	1.02	311	NOV

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WE7917

10.45.38 JAIN.07.70400.17.CEDC0154.SURN.  
 10.45.38 R-H (S)  
 10.45.40 CTIME 000.301 SEC. RUN# LEVEL 60\*  
 10.45.40 LGO.  
 10.45.42 LOADER UNUSED STORAGE 055523.  
 10.45.42 = 11700CM 0.20BCP 48MS OMT  
 10.45.50 ENC - PSI  
 10.45.50 MS 114 PRU.  
 10.45.50 CP 1.688 SEC.  
 10.45.50 PP 3.987 SEC.  
 10.45.50 TM 2.144 SEC. 3 (OCTAL)

APPENDIX 5

FLEXIBLE PAVEMENT PERFORMANCE RECORD

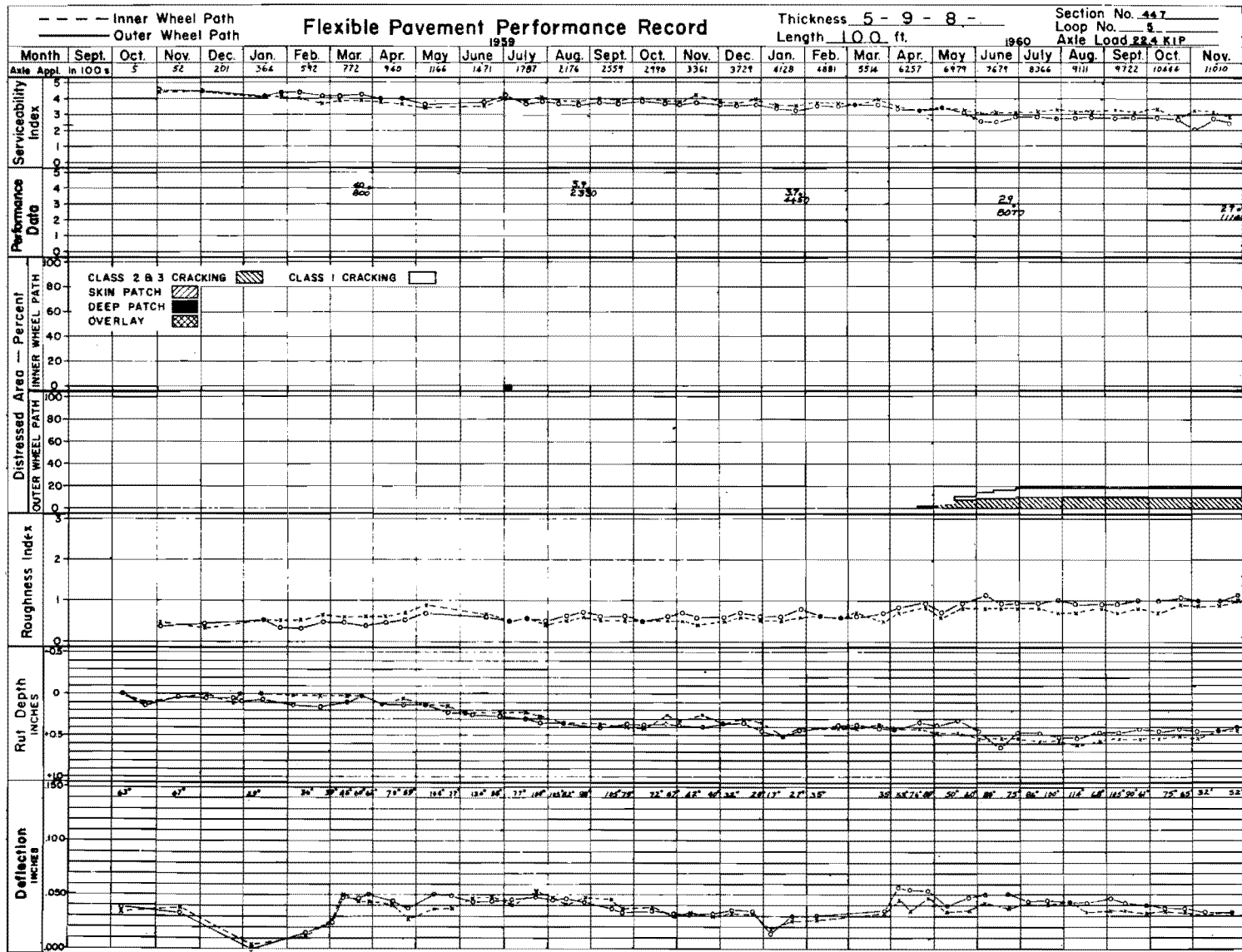


Fig A5.1. Typical AASHTO Road Test section history (Ref 70)



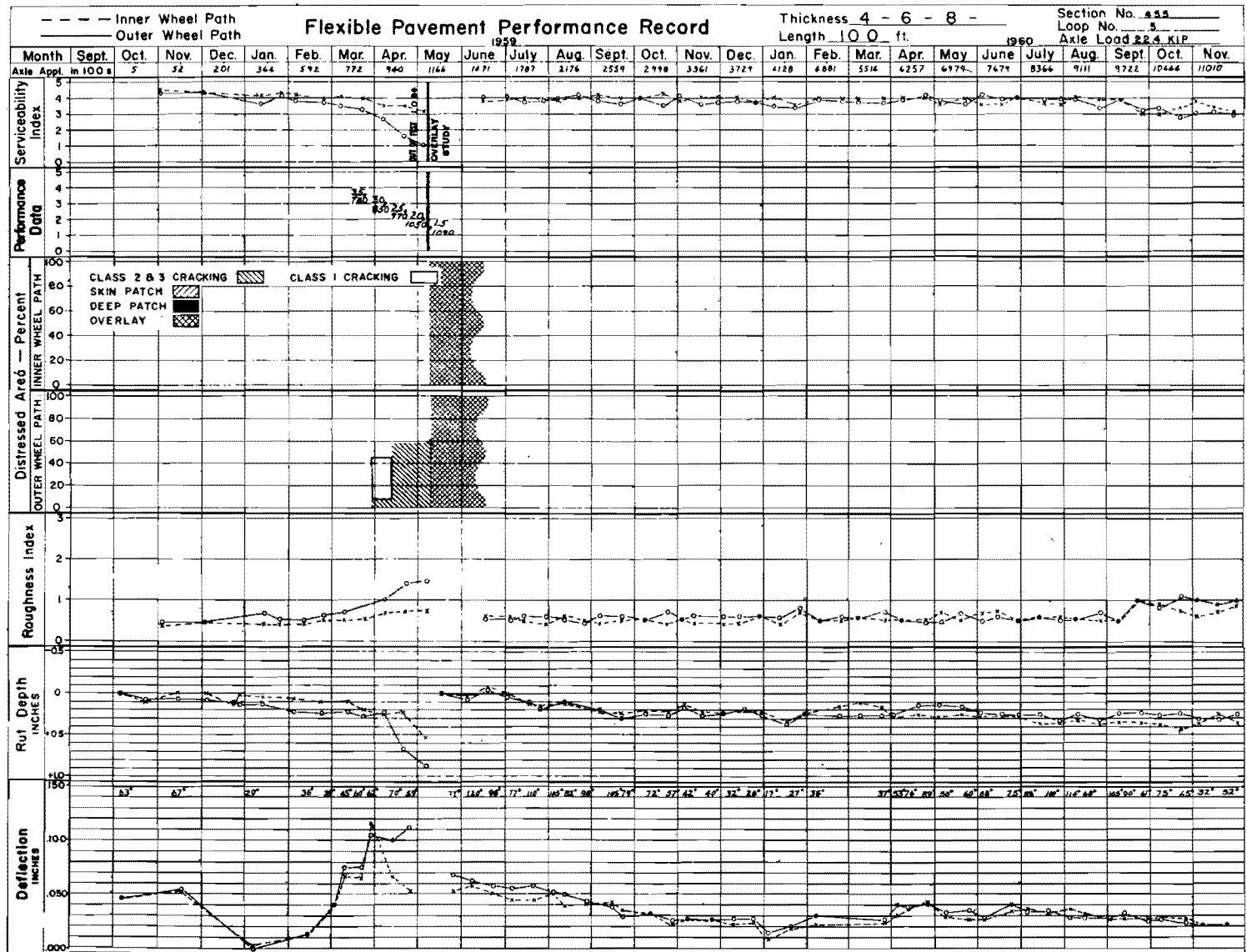


Fig A5.2. Typical AASHTO Road Test section history (Ref 70).

APPENDIX 6

NOMENCLATURE

## APPENDIX 6. NOMENCLATURE

This appendix gives the nomenclature for the fatigue tests used in this report (Ref 35).

### Flexural Test for Asphalt Concrete

The following nomenclature applies to the flexural fatigue tests on asphalt concrete.

$N_i$	The number of load applications of level $i$ to cause failure in simple loading.
$n_i$	The number of actual load applications of level $i$ .
$\epsilon$	Bending strain in flexural fatigue test.
A and B	Constants depending upon material characteristics.
E	Modulus of elasticity or stiffness.
$\mu$	Poisson's ratio.

### Repetitive Load-Deformation Tests on Base, Subbase, and Subgrade

The nomenclature used for repeated load tests on granular base and subbase and fine subgrade materials is given below. The nomenclature is also explained by Figs A6.1 and A6.2.

t	Total cumulative deformation when the maximum load is applied to the specimen.
p	Permanent cumulative deformation retained by the specimen between cyclic load applications.

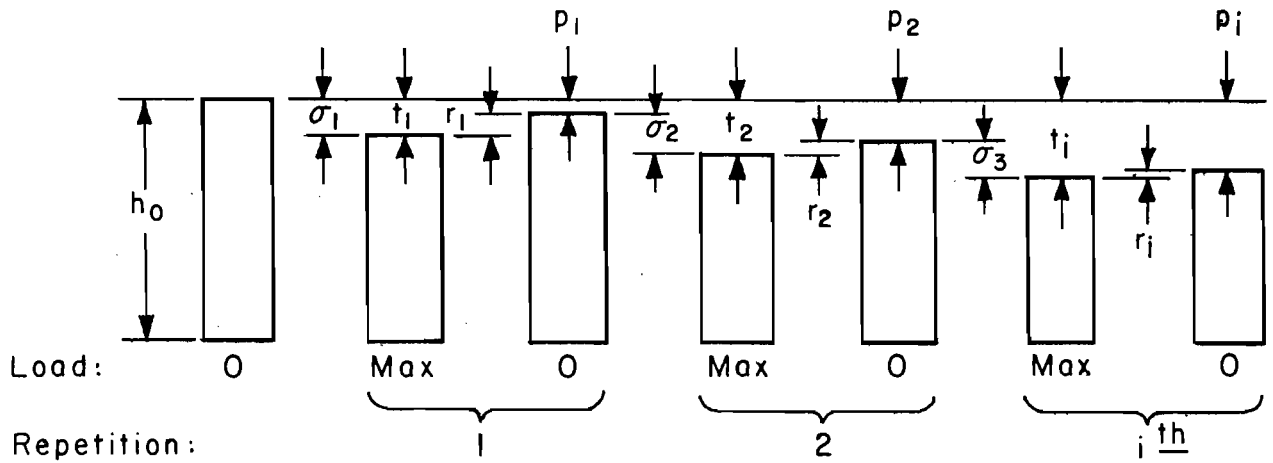


Fig A6.1. Diagrammatic representation of changes in specimen lengths during load and unload cycles (after Ref 35).

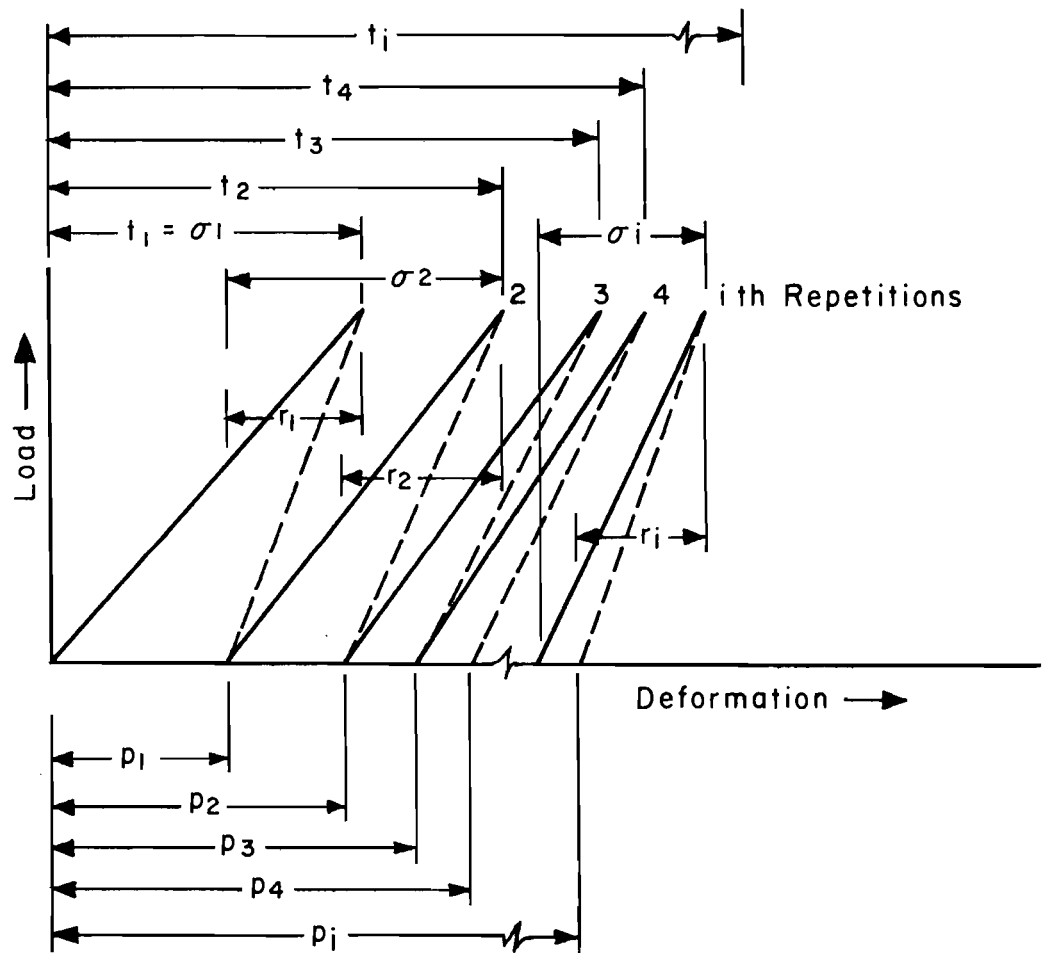


Fig A6.2. Hypothetical repetitive load-deformation relationship indicating suggested nomenclature for various types of deformations (after Ref 35).

- r Rebound non-cumulative deformation, which is equal to the difference between total and permanent deformation for any particular load application.
- $\sigma$  Transient non-cumulative deformation; deformation observed from zero to maximum stress for any particular load application.

Note:

- (1) Strains may be used instead of deformation.
- (2) Elastic refers to a condition where rebound or transient deformation approaches constant values over several stress repetitions.
- (3) Perfect resiliency is the state when continued loading produces no further total or permanent deformation, i.e., rebound equals the transient deformation.
- (4) Total strains in the triaxial load are analogous to the strain observed as a point in the roadway during application of the wheel load. Permanent strains may represent the net rut depth in the pavement.

APPENDIX 7

AASHO ROAD TEST AND PRESENT SERVICEABILITY CONCEPT

## APPENDIX 7. AASHO ROAD TEST AND PRESENT SERVICEABILITY CONCEPT

In this report the serviceability and performance concept using the performance data of the AASHO Road Test has been utilized to develop and verify the suitability of the suggested models. Thus, the purpose of this appendix is to discuss the AASHO Road Test and its performance concept.

### Introduction

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance and capabilities of highway pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. AASHO Road Test reports (Refs 67 - 73) contain complete information about this test. The principal objective of the road test was to determine the significant relationship between pavement behavior and the major variables of design and loading. The construction of the test facility was completed in 1958. Traffic started to move over it in November 1958 and continued through November 1960. A total of 1,114,000 axle load applications was accumulated. Based on the results of the AASHO Road Test an Interim Design Guide was published in 1962.

### Pavement Performance

The popular pavement service and performance concept was also developed at the AASHO Road Test in 1962. The failure of a pavement system is generally not a catastrophic occurrence, as is the case in some other structures. A pavement which has been designated as "failed" in some response may still be capable of carrying traffic at a reduced service level. It is clear that cracks will occur if a pavement is overstressed, but the question is how much they are going to affect the performance of the pavement. Cracks are undesirable but the degree of undesirability is not known. Comfort and convenience may be considered inherent manifestations of pavement performance. The performance of a pavement is influenced by many factors, including applied loads, tire pressure, number of load applications, and thickness and strength

characteristics of pavement layers and subgrade. Thus, functional pavement design should correlate these factors with desired performance characteristics.

To introduce the measure of pavement performance, certain terms used in the design methods were defined as below (Ref 70):

- (1) Present Serviceability Rating (PSR): The mean of the individual ratings made by the members of a specific panel of men selected for the purpose.
- (2) Present Serviceability Index (PSI): A mathematical combination of values obtained from certain physical measurements of a large number of pavements so formulated as to predict the PSR for those pavements within prescribed limits. This represents the ability of the pavement to serve high-speed, high-volume, mixed traffic in its existing condition. (The definition applies to the condition existing - on the date of rating, not to the condition assumed the next day or at any future or past date.)
- (3) Performance Index: A summary of PSI values over a period of time (See Fig A7.1).

#### Present Serviceability Index Equation

Based on regression analysis of measurement and panel ratings on 49 rigid and 74 flexible pavement test sections, the following index equations were suggested (Ref 70):

For flexible pavements,

$$PSI = 5.03 - 1.91 \log (1 + \overline{SV}) - 1.38 \overline{RD}^2 - 0.01 \sqrt{C + P} \quad (A7.1)$$

For rigid pavements,

$$PSI = 5.41 - 1.78 \log (1 + \overline{SV}) - 0.09 \sqrt{C + P} \quad (A7.2)$$

where slope variance  $\overline{SV}$  is the statistical measure of the variability of the slope of the pavement. It is a direct measurement of the longitudinal roughness of the pavement. A continuous analog trace of the pavement slope is obtained from the profilometer, and a point measurement of slope  $S_1$  at a one-foot interval is obtained. Then

$$SV = 10^6 \times \text{slope variance} = 10^6 \times \sigma_s^2$$



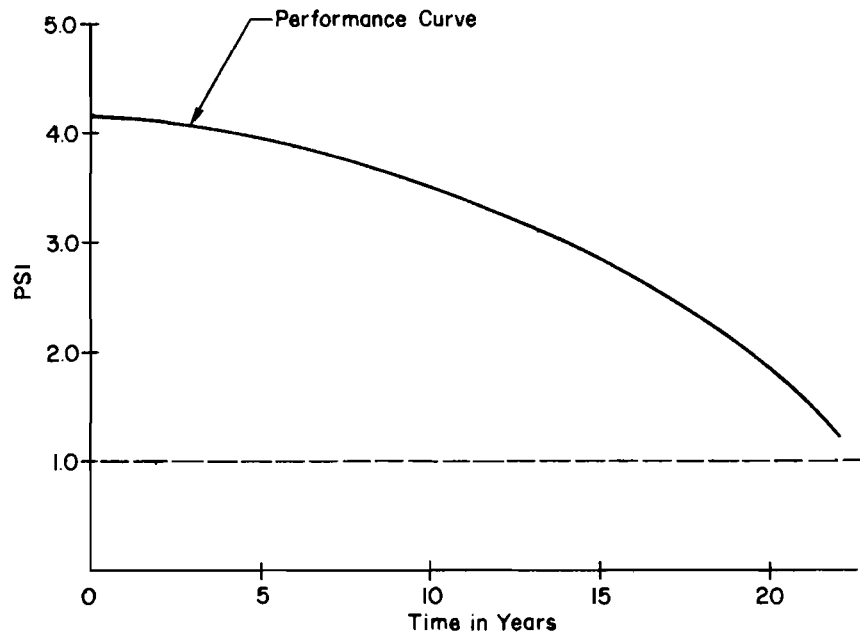


Fig A7.1. Performance curve.

$$= 10^6 \sum_{t=1}^n (S_t - \bar{S})^2 / (n-1) \quad (\text{A7.3})$$

where

$n$  = number of measurements

$\bar{S}$  = mean slope measurement

Rut depth  $\overline{RD}$  is the measurement of the amount of permanent deformation in the transverse profile of the pavement. It is measured in inches below the center of a 4-foot span placed across the wheel path. These measurements are made throughout the length of the section and then averaged.

Cracking and patching  $C + P$  is the measurement of the major cracking (classes 2 and 3) and patching in square feet per 1000-square-foot area of flexible pavement. In rigid pavement, class 2 and sealed cracks are measured in feet by 1000 square feet area of pavement.

As seen from the PSI equation, slope variance is the most important single variable influencing the PSI of the pavement. Rutting plays a secondary role while cracking and patching was found to have only a minor role in determination of the serviceability or riding quality of the pavement. However, it is emphasized that this does not mean that cracking is of minor importance as far as the design or structural behavior, pavement life, or even the serviceability is concerned, because cracking in the pavement is itself indicative of other forms of distress and is a direct indication of a structural inadequacy somewhere in the pavement. By the time enough cracks are developed in the pavement it is already rough in terms of slope variance. Therefore, it seems that the slope variance is the cause of detrimental effects of cracking.

#### AASHO Road Test Data

AASHO Road Test data are a good source of performance data. Performance data in the form of plots of cracking and patching, roughness index, PSI, and rut depth are available for each test section. Typical plots of this data are shown in Appendix 5. These plots were used in the development and verification of the models discussed in this report.

APPENDIX 8

COMPARISON OF DISTRESS MODELS - PLOTS

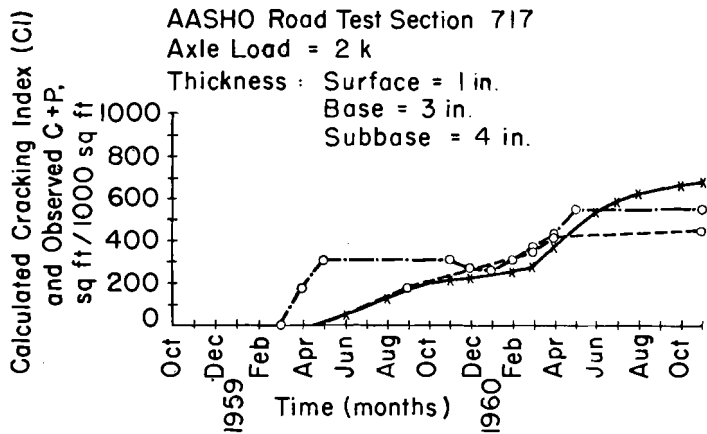


Fig A8.1. Computed cracking index (CI) versus observed cracking-patching (C + P).

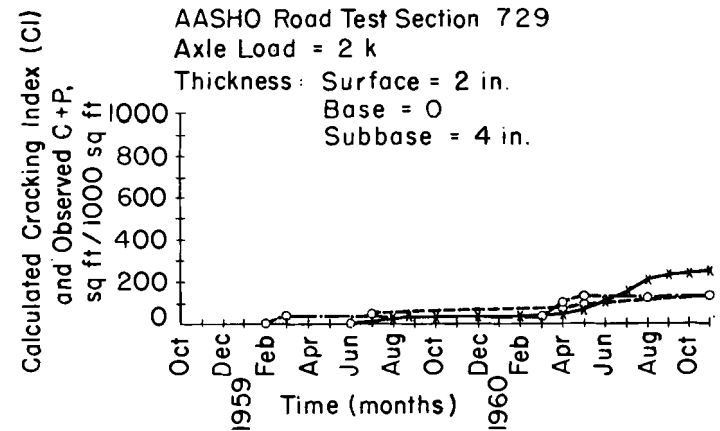


Fig A8.2. Computed cracking index (CI) versus observed cracking-patching (C + P).

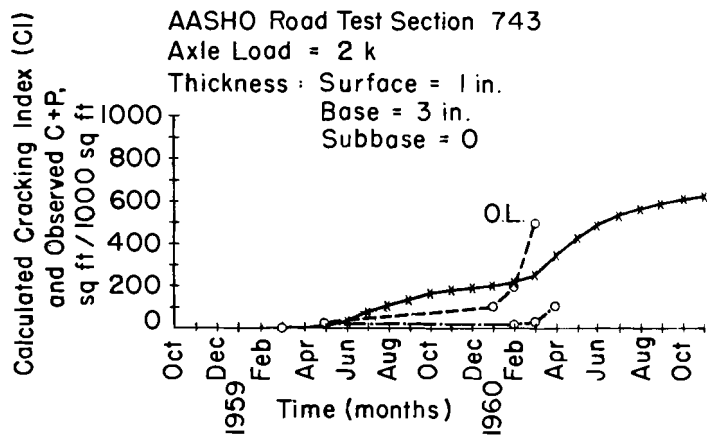


Fig A8.3. Computed cracking index (CI) versus observed cracking-patching (C + P).

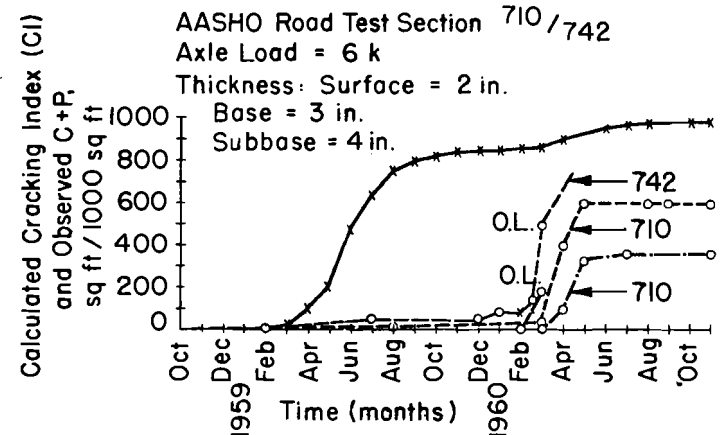


Fig A8.4. Computed cracking index (CI) versus observed cracking-patching (C + P).

Observed (inner wheel) ---o---  
 Observed (outer wheel) ---o---  
 Calculated Section Overlaid at Point ---x---  
 Section Overlaid at Point o O.L.

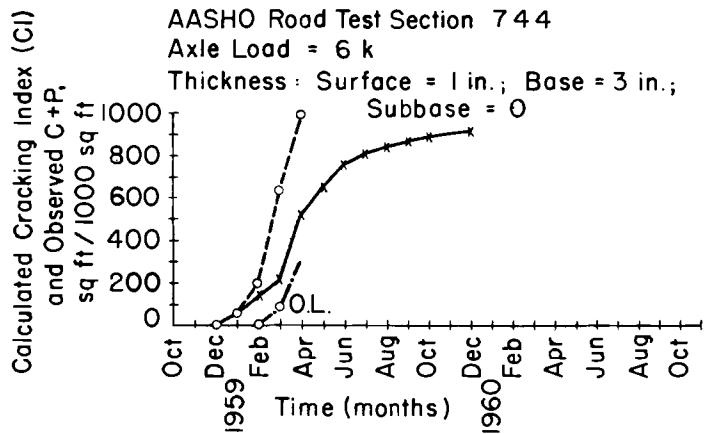


Fig A8.5. Computed cracking index (CI) versus observed cracking-patching (C + P).

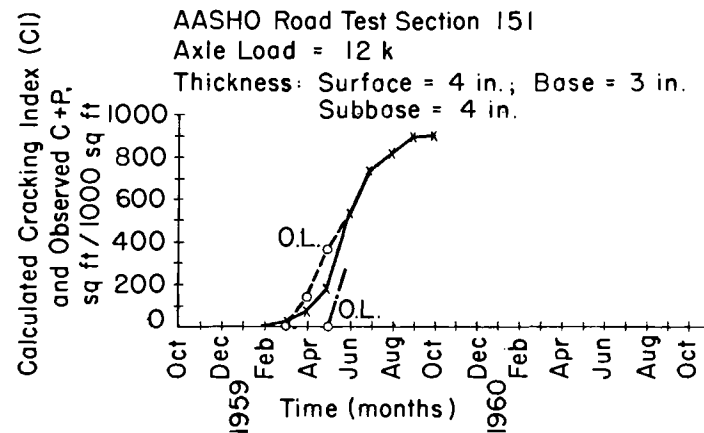


Fig A8.6. Computed cracking index (CI) versus observed cracking-patching (C + P).

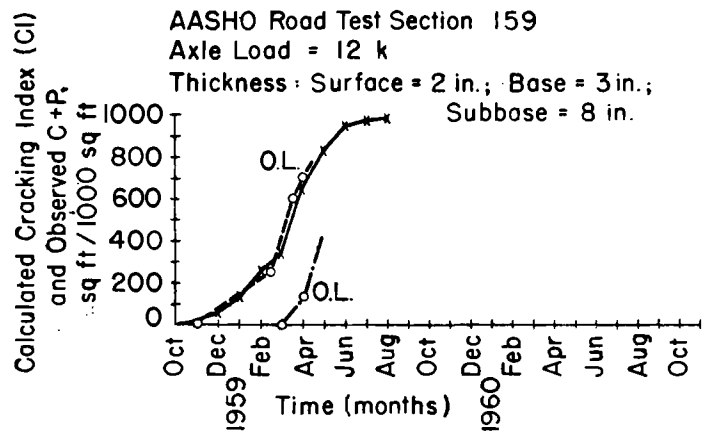


Fig A8.7. Computed cracking index (CI) versus observed cracking-patching (C + P).

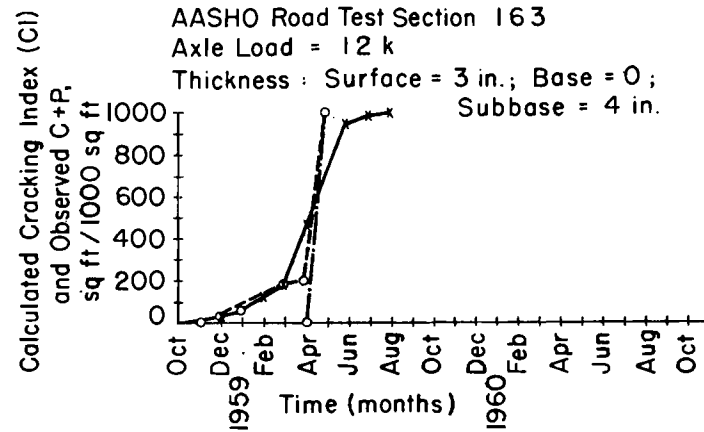


Fig A8.8. Computed cracking index (CI) versus observed cracking-patching (C + P).

Observed (inner wheel) —○—  
Observed (outer wheel) —○—  
Calculated —\*—  
Section Overload at Point —○—

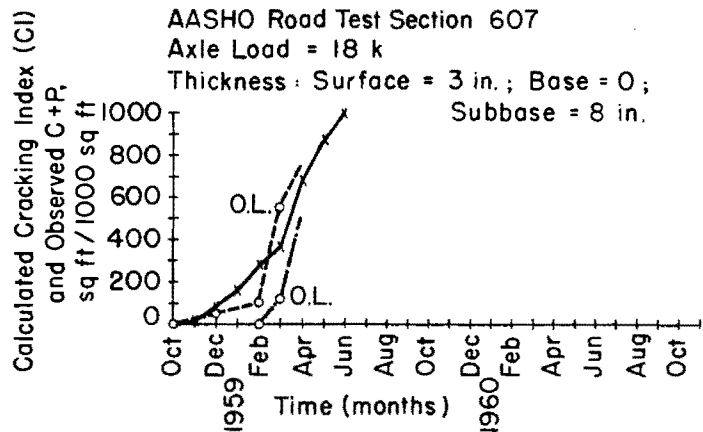


Fig A8.9. Computed cracking index (CI) versus observed cracking-patching (C + P).

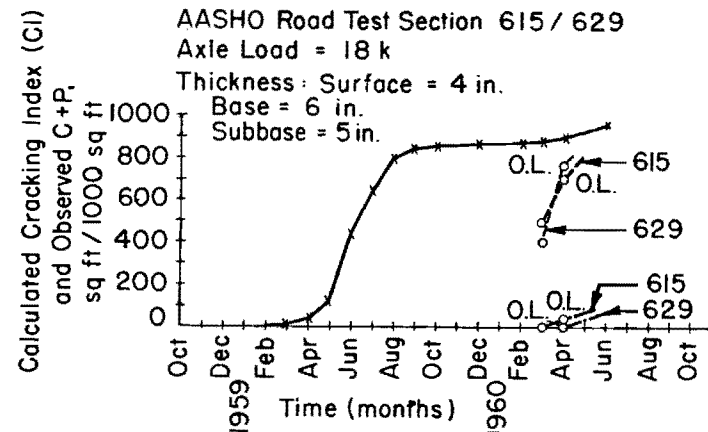


Fig A8.10. Computed cracking index (CI) versus observed cracking-patching (C + P).

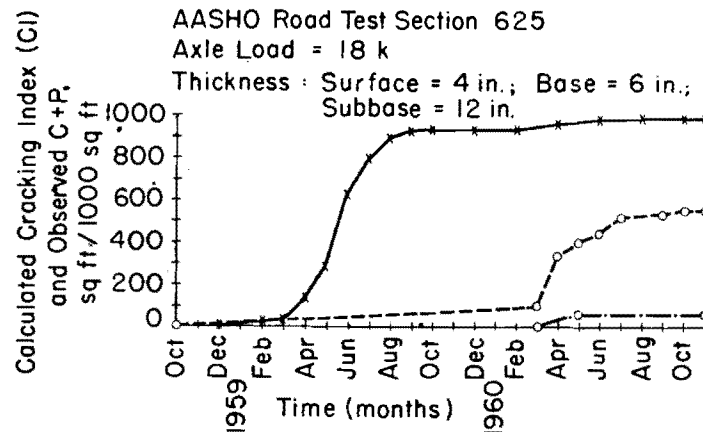


Fig A8.11. Computed cracking index (CI) versus observed cracking-patching (C + P).

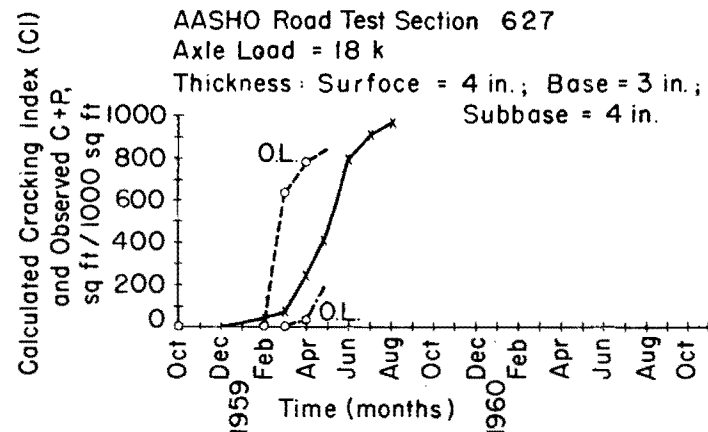


Fig A8.12. Computed cracking index (CI) versus observed cracking-patching (C + P).

Observed (inner wheel) —○—○—      Calculated      ×××××  
 Observed (outer wheel) —○—○—      Section Overload at Point      ○ O.L.

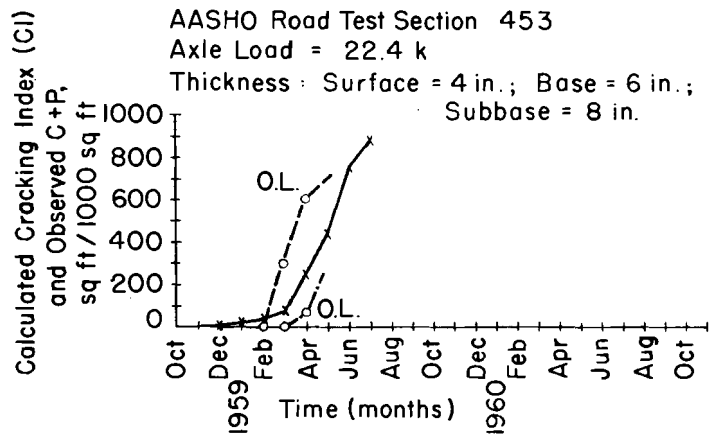


Fig A8.13. Computed cracking index (CI) versus observed cracking-patching (C + P).

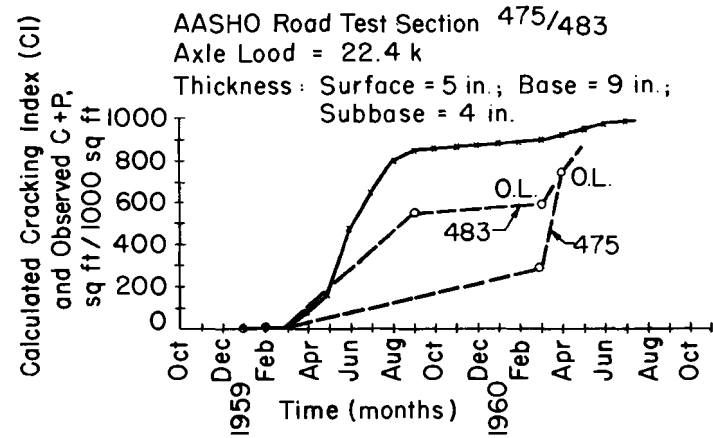


Fig A8.14. Computed cracking index (CI) versus observed cracking-patching (C + P).

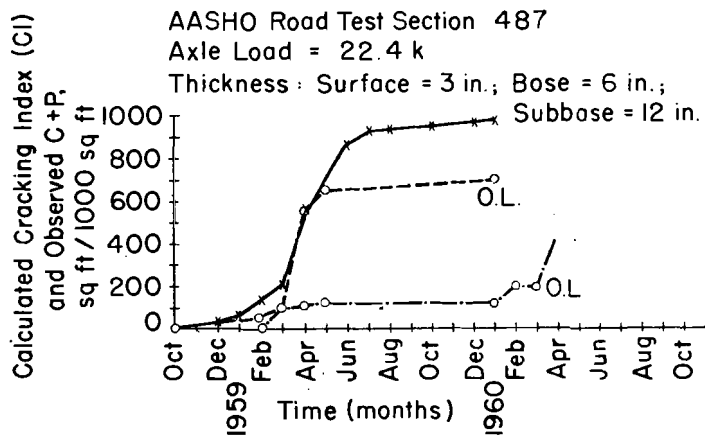


Fig A8.15. Computed cracking index (CI) versus observed cracking-patching (C + P).

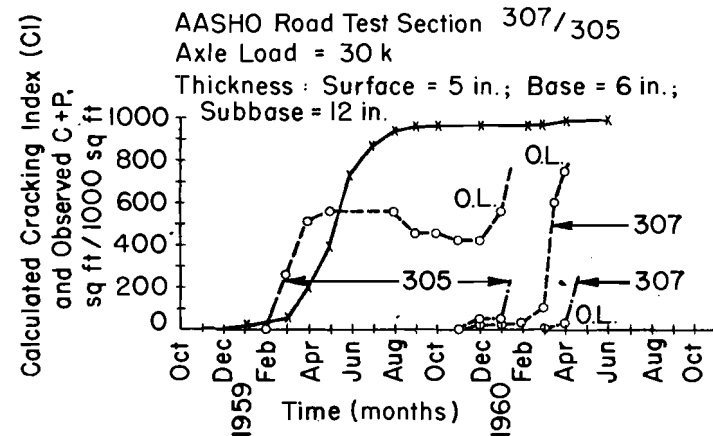


Fig A8.16. Computed cracking index (CI) versus observed cracking-patching (C + P).

Observed (inner wheel) —○—  
 Observed (outer wheel) - -○- -  
 Calculated —×—  
 Section Overlaid at Point ○ O.L.

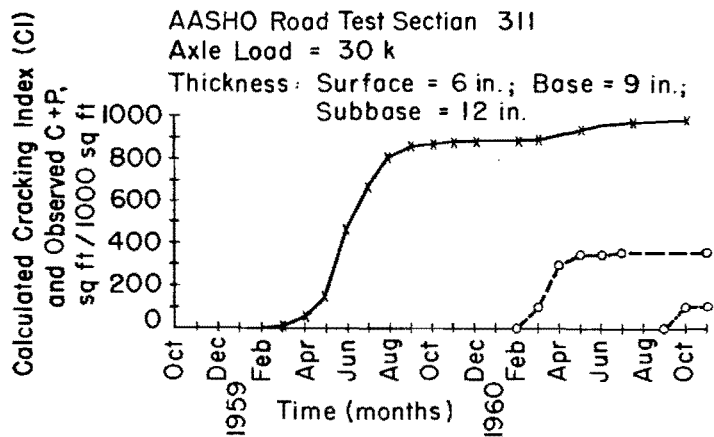


Fig A8.17. Computed cracking index (CI) versus observed cracking-patching (C + P).

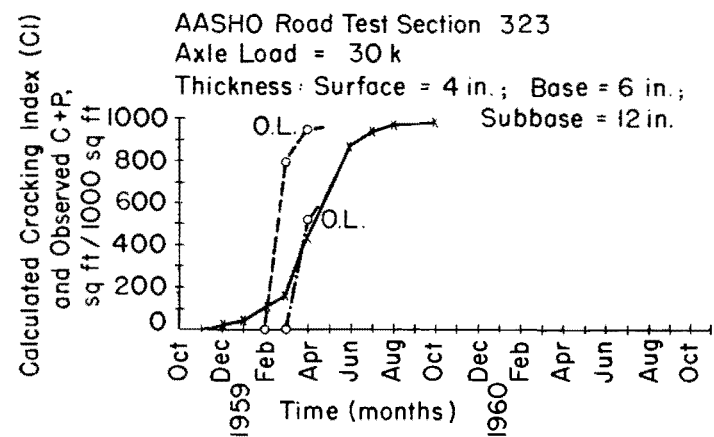


Fig A8.18. Computed cracking index (CI) versus observed cracking-patching (C + P).

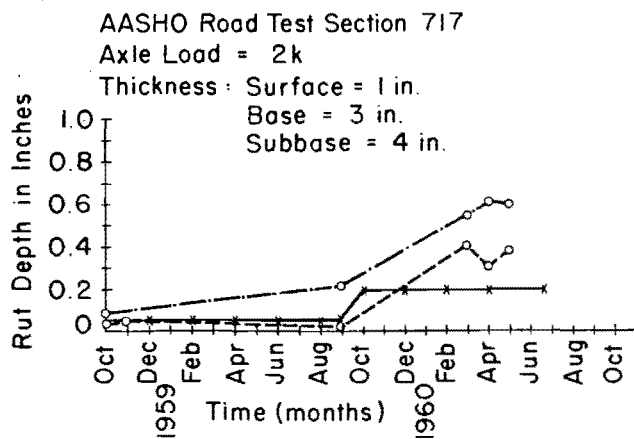


Fig A8.19. Observed versus calculated rut depth index.

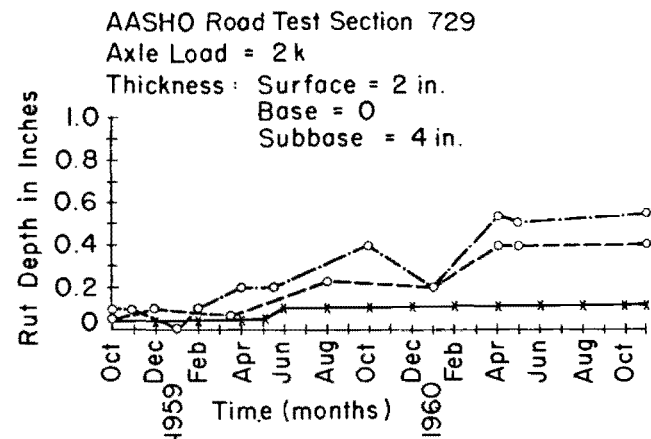


Fig A8.20. Observed versus calculated rut depth index.

Observed (inner wheel) —○—○—  
Observed (outer wheel) —○—○—  
Calculated —●—●—  
Section Overload at Point —OL—



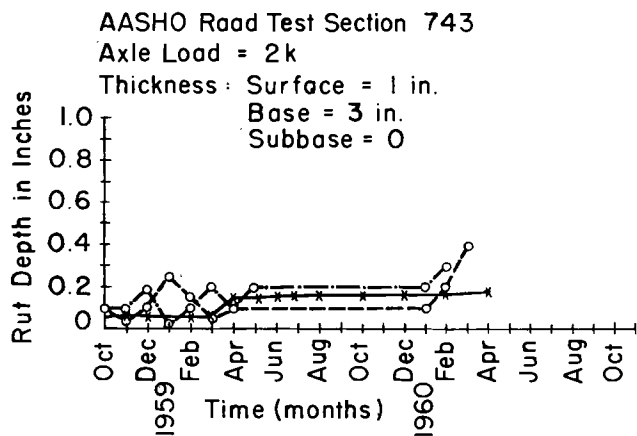


Fig A8.21. Observed versus calculated rut depth index.

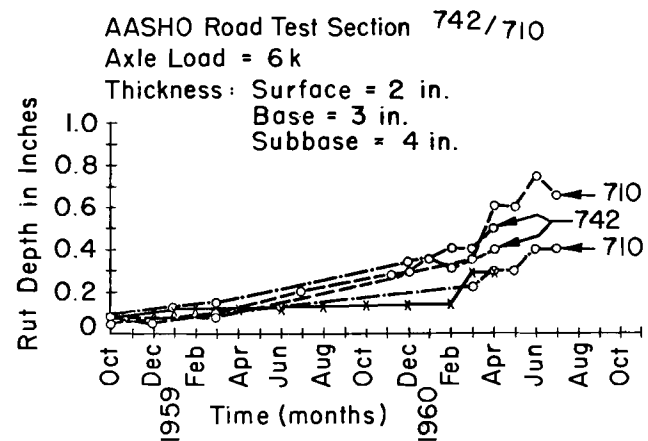


Fig A8.22. Observed versus calculated rut depth index.

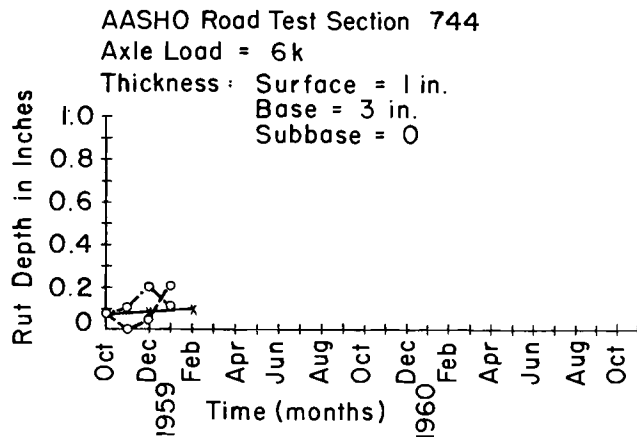


Fig A8.23. Observed versus calculated rut depth index.

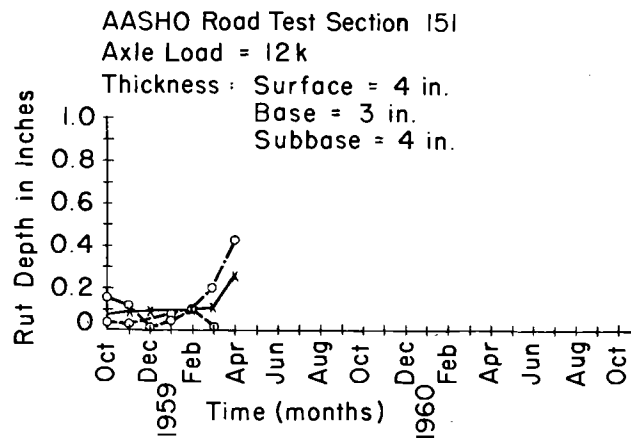


Fig A8.24. Observed versus calculated rut depth index.

Observed (inner wheel) —○—  
 Observed (outer wheel) —○—  
 Calculated —\*—

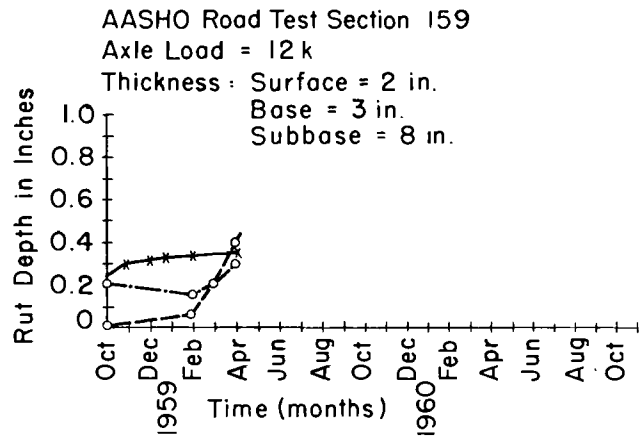


Fig A8.25. Observed versus calculated rut depth index.

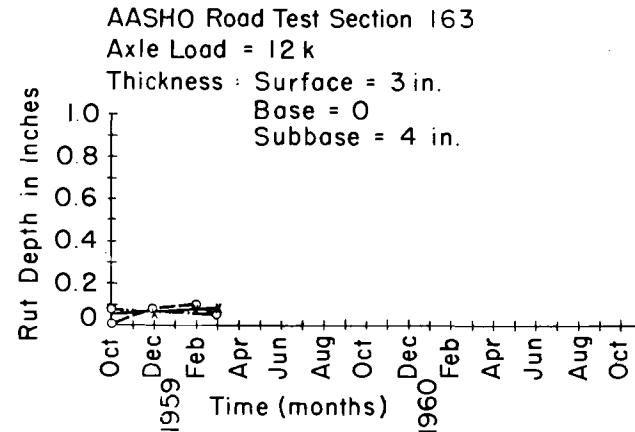


Fig A8.26. Observed versus calculated rut depth index.

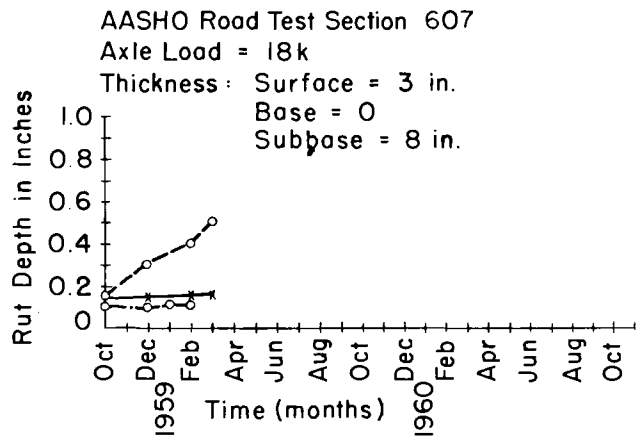


Fig A8.27. Observed versus calculated rut depth index.

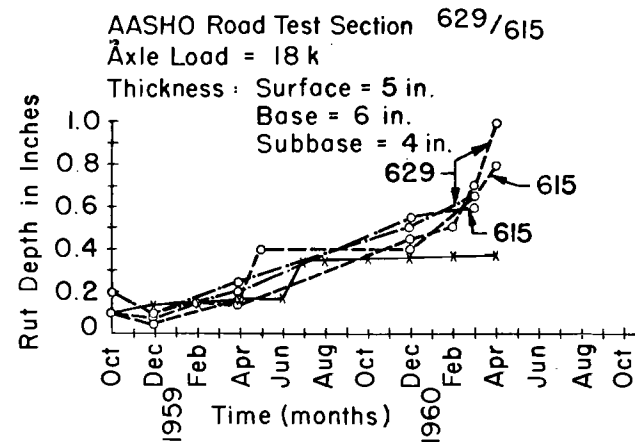


Fig A8.28. Observed versus calculated rut depth index.

Observed (inner wheel) —○—○—      Calculated ———  
 Observed (outer wheel) —×—×—

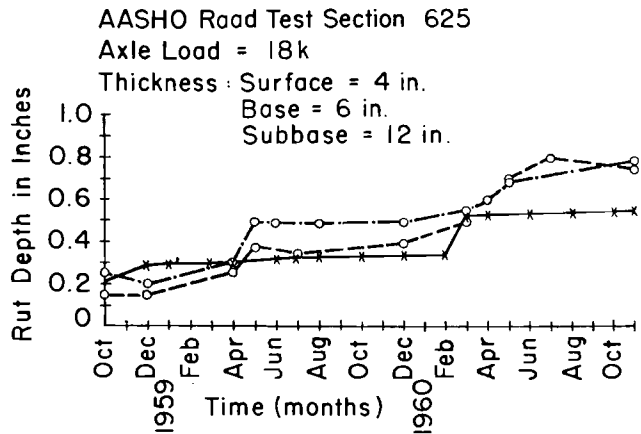


Fig A8.29. Observed versus calculated rut depth index.

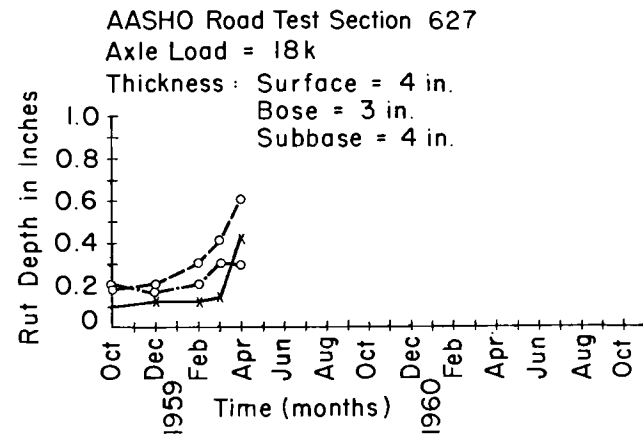


Fig A8.30. Observed versus calculated rut depth index.

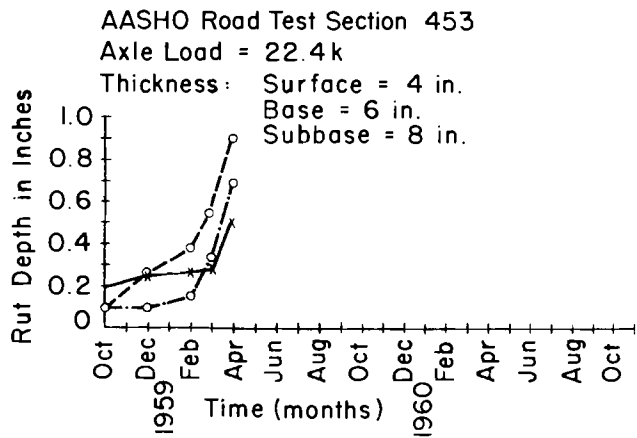


Fig A8.31. Observed versus calculated rut depth index.

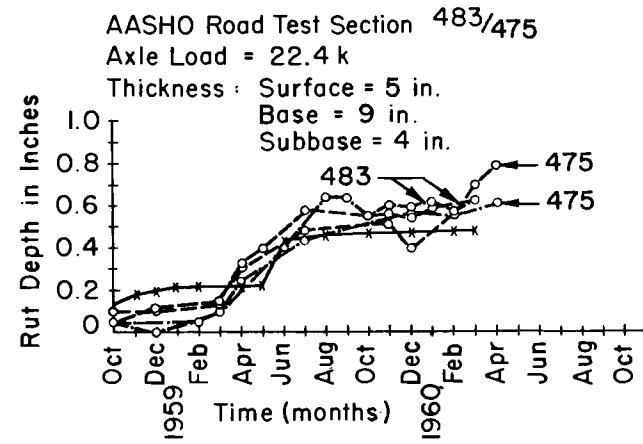


Fig A8.32. Observed versus calculated rut depth index.

Observed (inner wheel) —○—  
 Observed (outer wheel) - -○- -  
 Calculated —x—

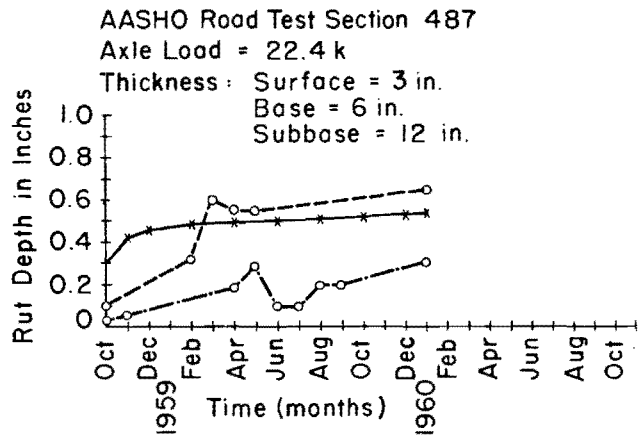


Fig A8.33. Observed versus calculated rut depth index.

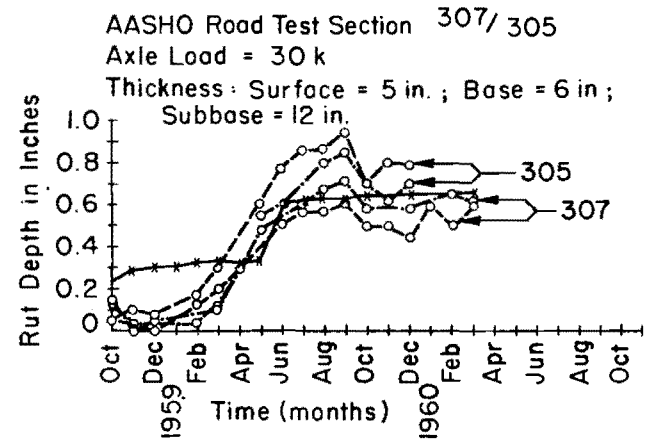


Fig A8.34. Observed versus calculated rut depth index.

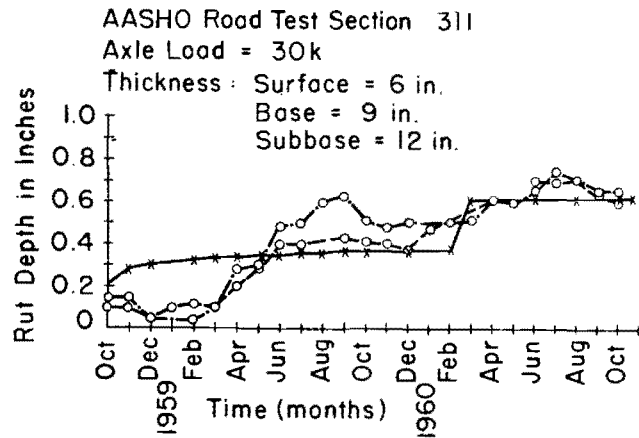


Fig A8.35. Observed versus calculated rut depth index.

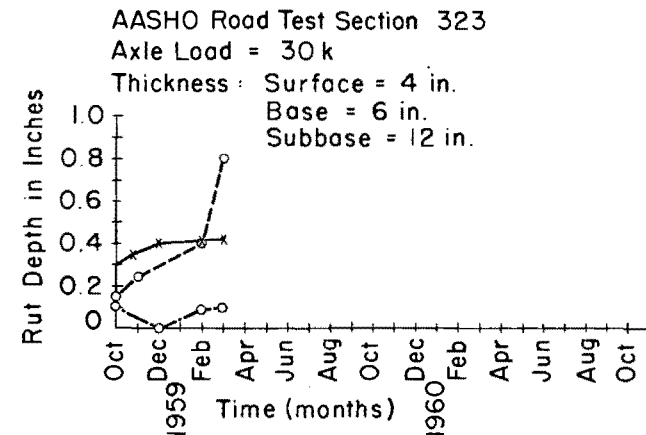


Fig A8.36. Observed versus calculated rut depth index.

Observed (inner wheel) —○— Calculated  
 Observed (outer wheel) —○—



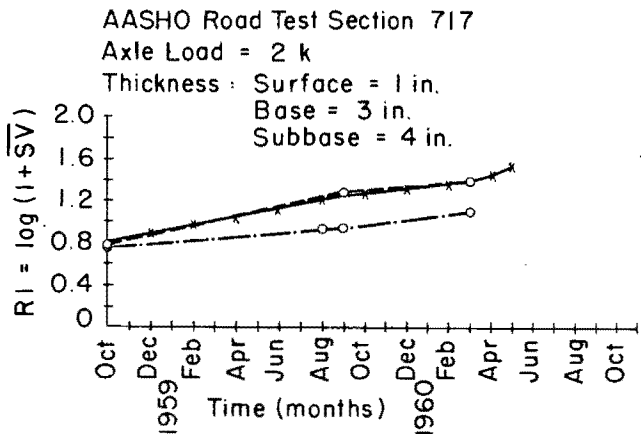


Fig A8.37. Observed versus calculated roughness index, RI.

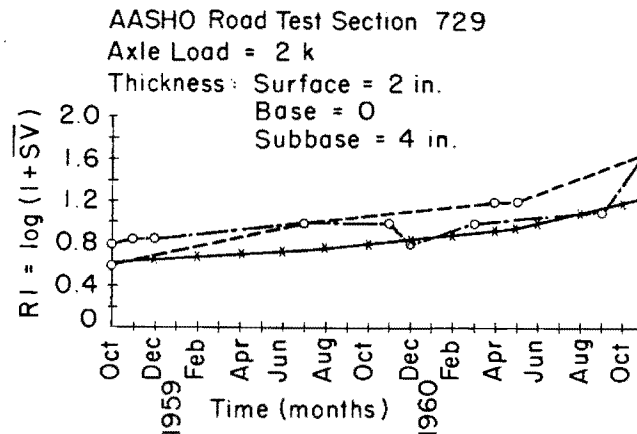


Fig A8.38. Observed versus calculated roughness index, RI.

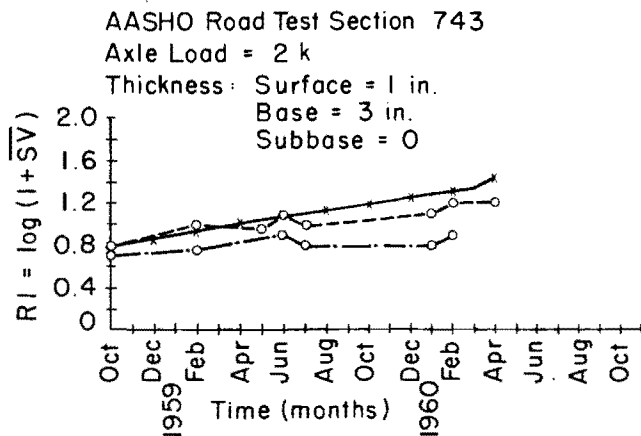


Fig A8.39. Observed versus calculated roughness index, RI.

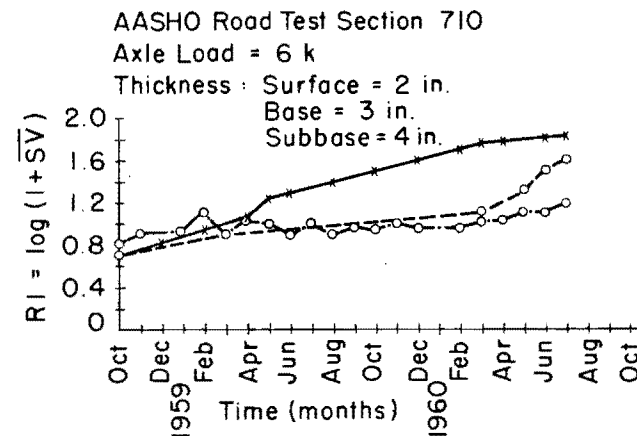


Fig A8.40. Observed versus calculated roughness index, RI.

Observed (inner wheel) —○—  
 Observed (outer wheel) - -○- -  
 Calculated —x—

AASHO Road Test Section 742

Axle Load = 6 k

Thickness: Surface = 2 in.

Base = 3 in.; Subbase = 4 in.

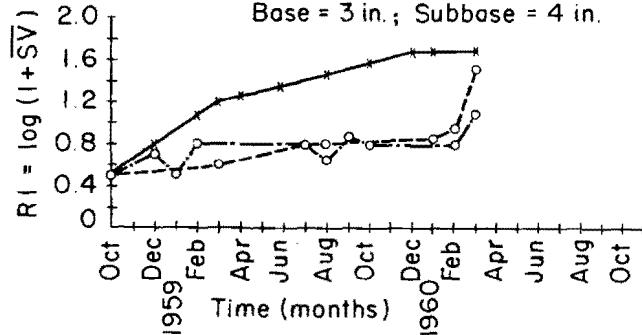


Fig A8.41. Observed versus calculated roughness index, RI.

AASHO Road Test Section 744

Axle Load = 6 k

Thickness: Surface = 1 in.

Base = 3 in.  
Subbase = 0

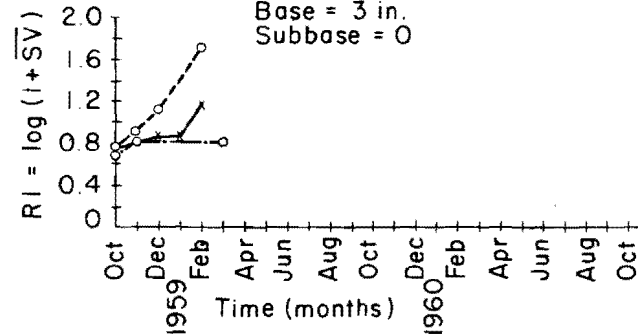


Fig A8.42. Observed versus calculated roughness index, RI.

AASHO Road Test Section 151

Axle Load = 12 k

Thickness: Surface = 4 in.

Base = 3 in.  
Subbase = 4 in.

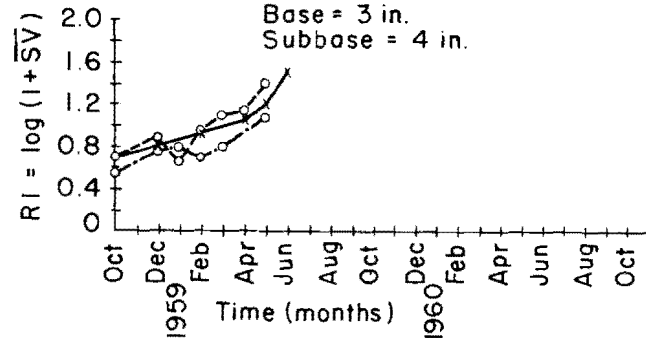


Fig A8.43. Observed versus calculated roughness index, RI.

AASHO Road Test Section 159

Axle Load = 12 k

Thickness: Surface = 2 in.

Base = 3 in.; Subbase = 8 in.

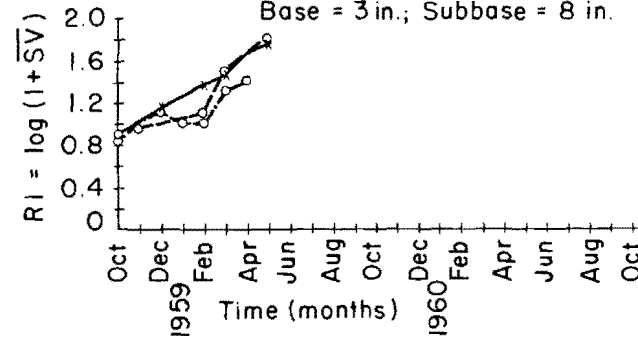
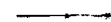


Fig A8.44. Observed versus calculated roughness index, RI.

Observed (inner wheel) —○—  
Observed (outer wheel) - -○- -  
Calculated —×—



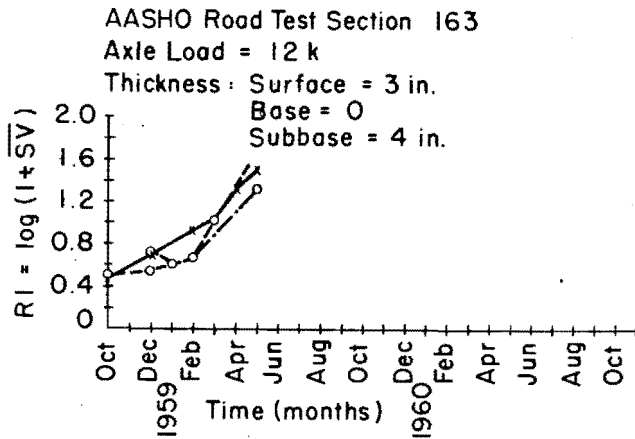


Fig A8.45. Observed versus calculated roughness index, RI.

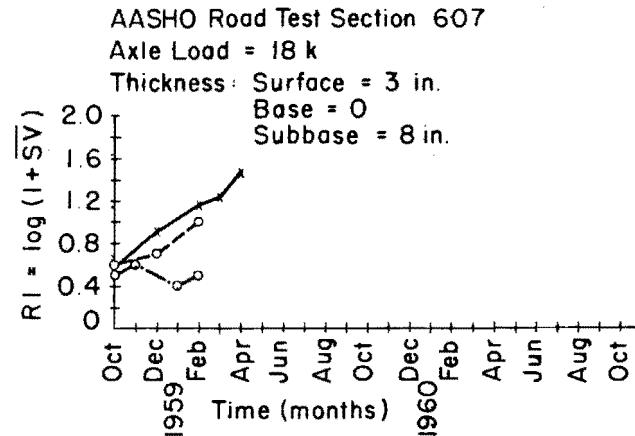


Fig A8.46. Observed versus calculated roughness index, RI.

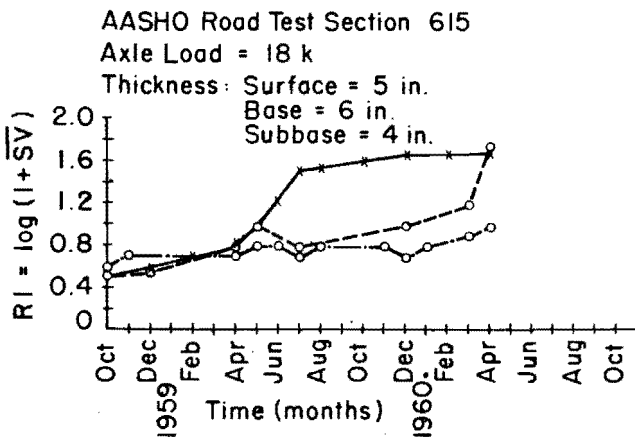


Fig A8.47. Observed versus calculated roughness index, RI.

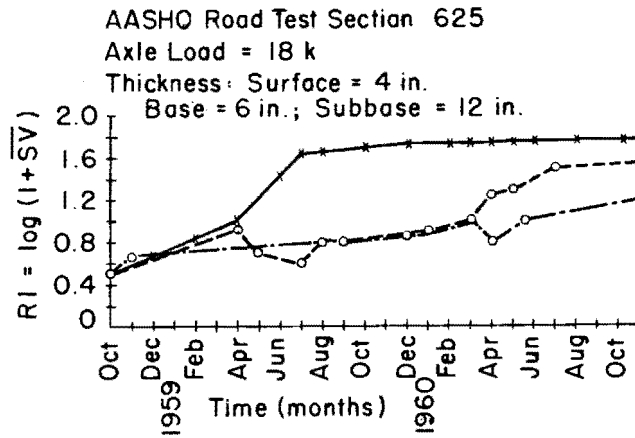
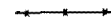


Fig A8.48. Observed versus calculated roughness index, RI.

Observed (inner wheel) —○— Calculated  
 Observed (outer wheel) - -○- -



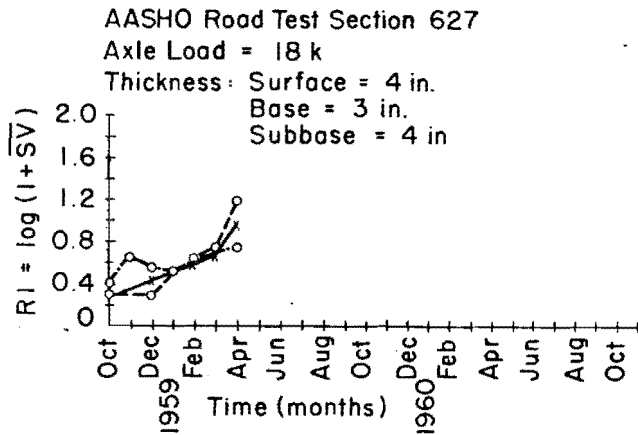


Fig A8.49. Observed versus calculated roughness index, RI.

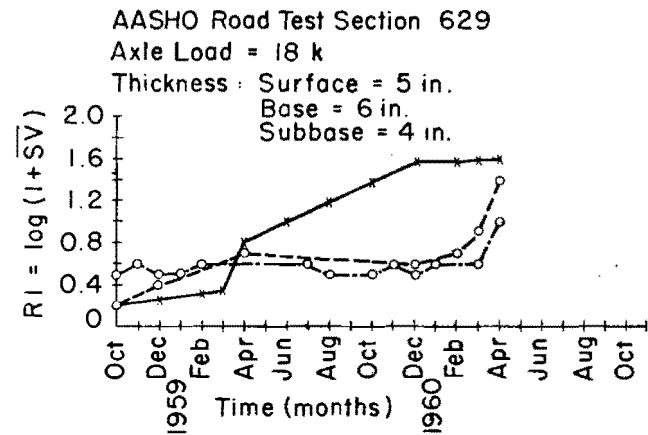


Fig A8.50. Observed versus calculated roughness index, RI.

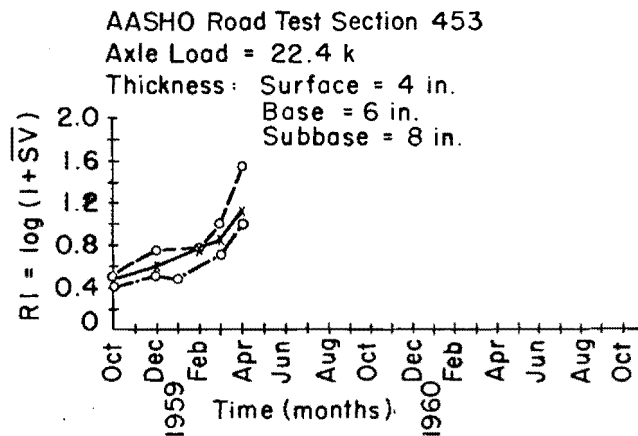


Fig A8.51. Observed versus calculated roughness index, RI.

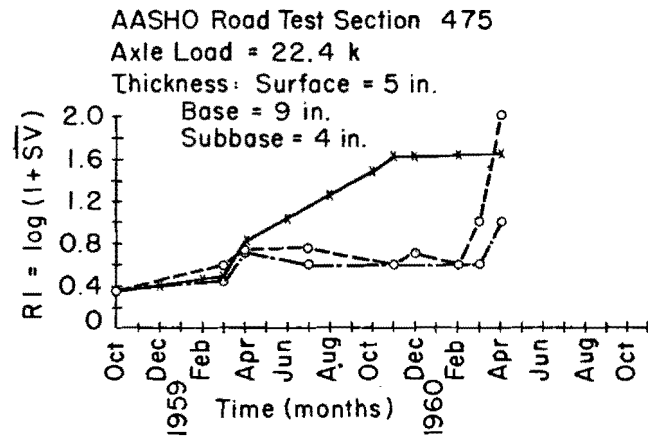


Fig A8.52. Observed versus calculated roughness index, RI.

Observed (inner wheel) —○—  
 Observed (outer wheel) - -○- -  
 Calculated —×—



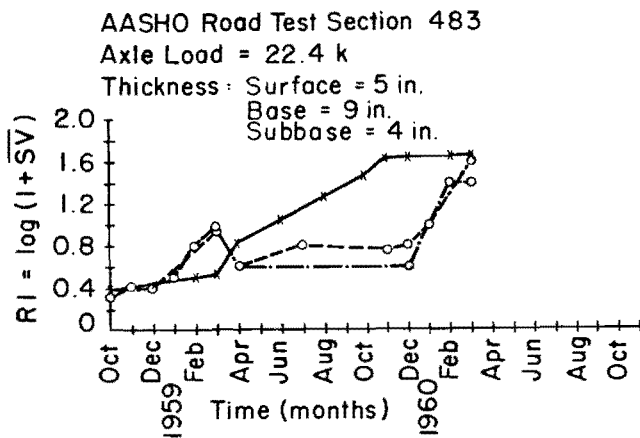


Fig A8.53. Observed versus calculated roughness index, RI.

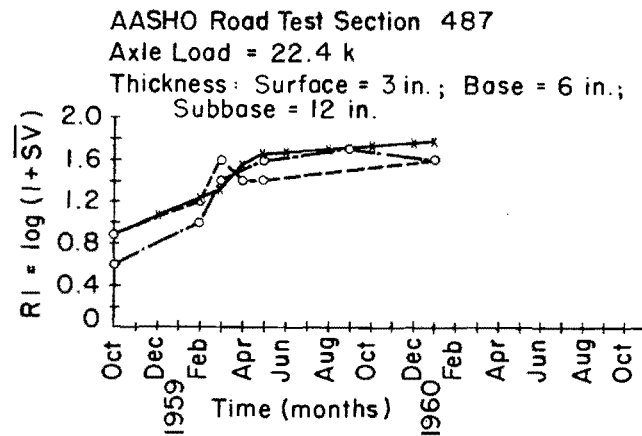


Fig A8.54. Observed versus calculated roughness index, RI.

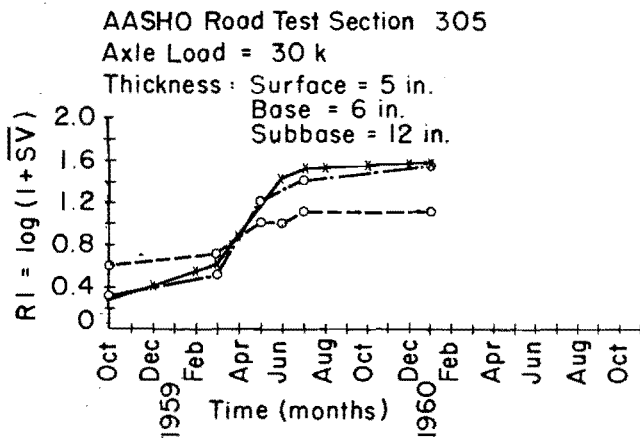


Fig A8.55. Observed versus calculated roughness index, RI.

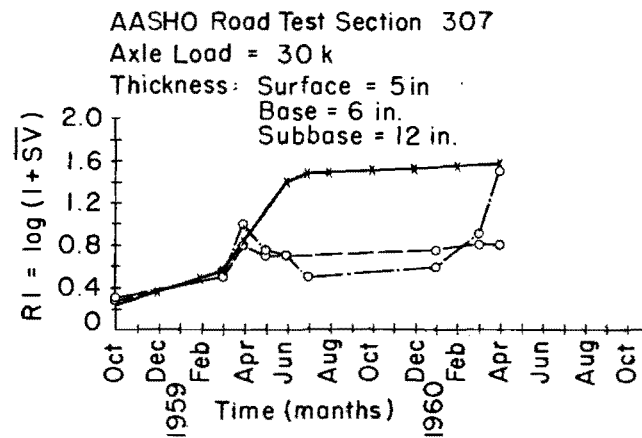


Fig A8.56. Observed versus calculated roughness index, RI.

Observed (inner wheel) —○—  
 Observed (outer wheel) - -○- -  
 Calculated —×—

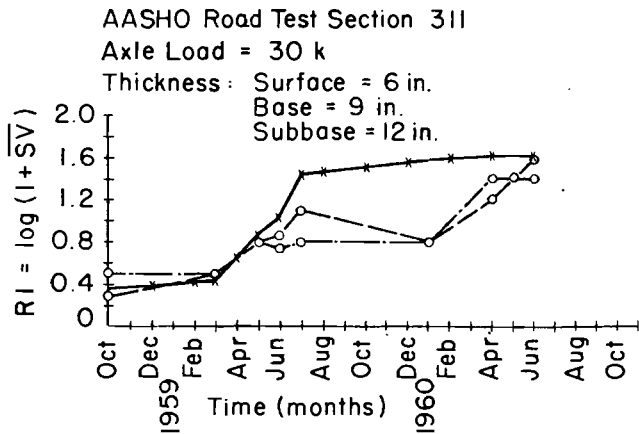


Fig A8.57. Observed versus calculated roughness index, RI.

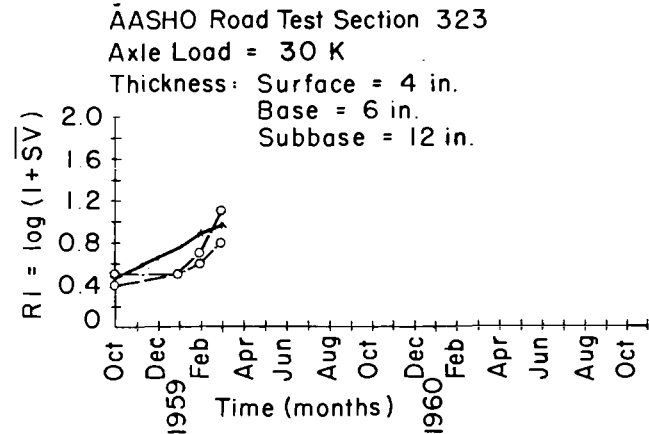


Fig A8.58. Observed versus calculated roughness index, RI.

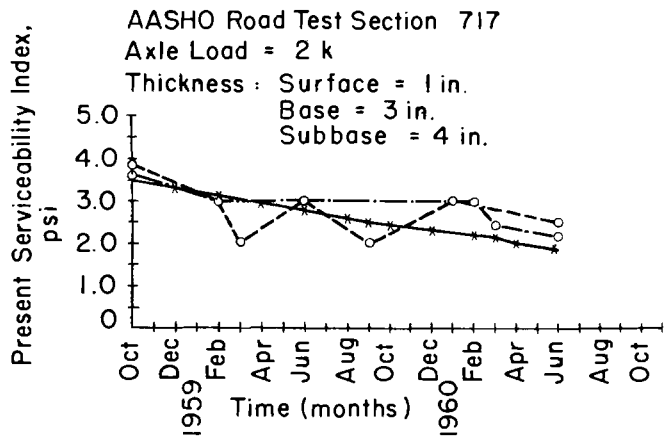


Fig A8.59. Observed versus calculated present serviceability index.

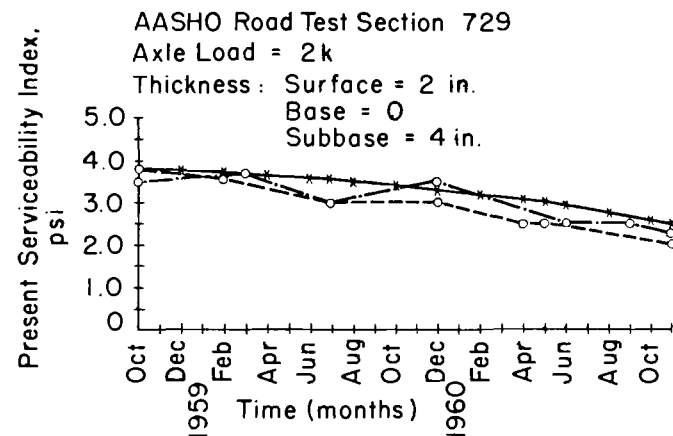


Fig A8.60. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—  
 Observed (outer wheel) —□—  
 Calculated —×—

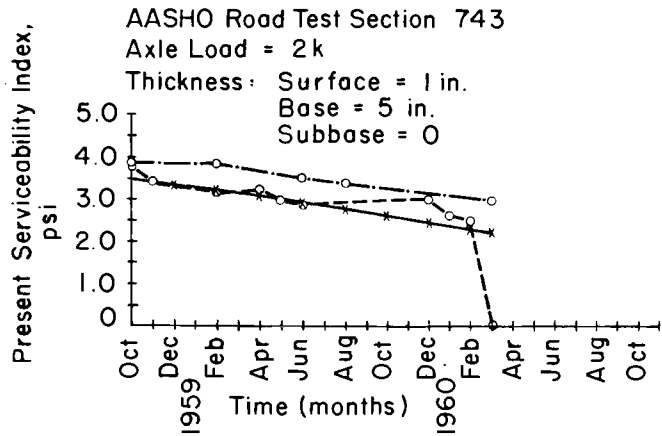


Fig A8.61. Observed versus calculated present serviceability index.

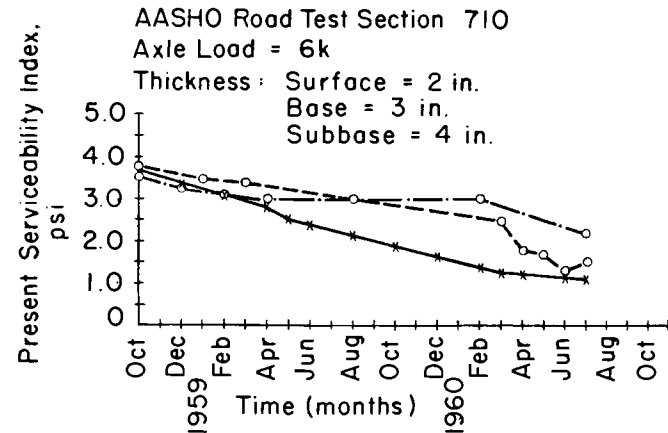


Fig A8.62. Observed versus calculated present serviceability index.

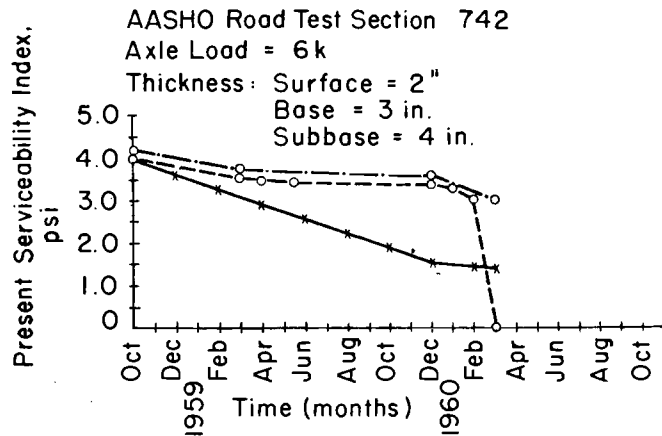


Fig A8.63. Observed versus calculated present serviceability index.

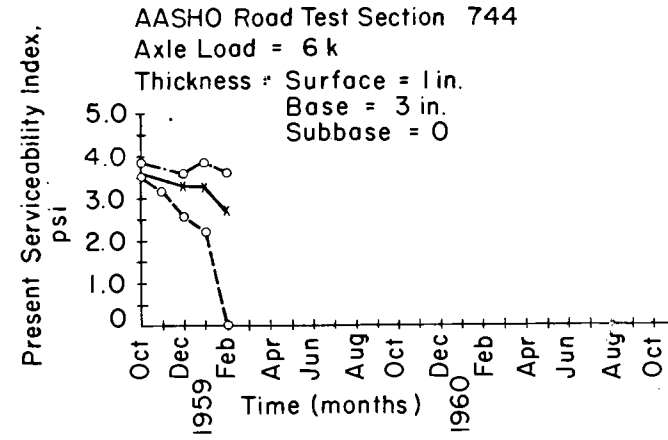


Fig A8.64. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—  
 Observed (outer wheel) —○—  
 Calculated —x—

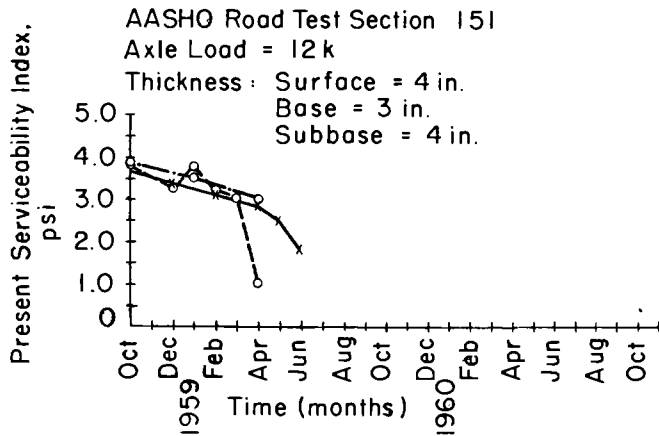


Fig A8.65. Observed versus calculated present serviceability index.

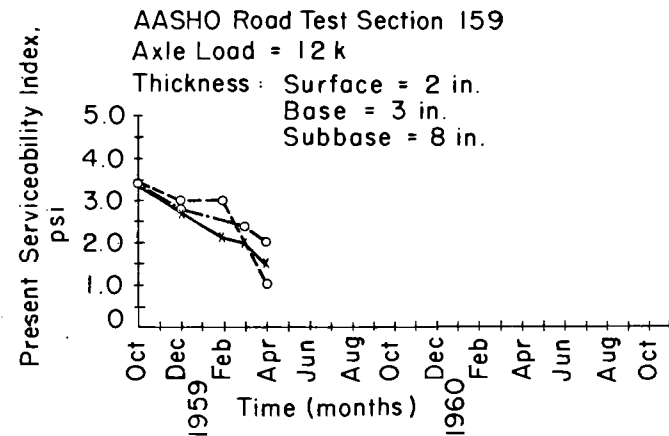


Fig A8.66. Observed versus calculated present serviceability index.

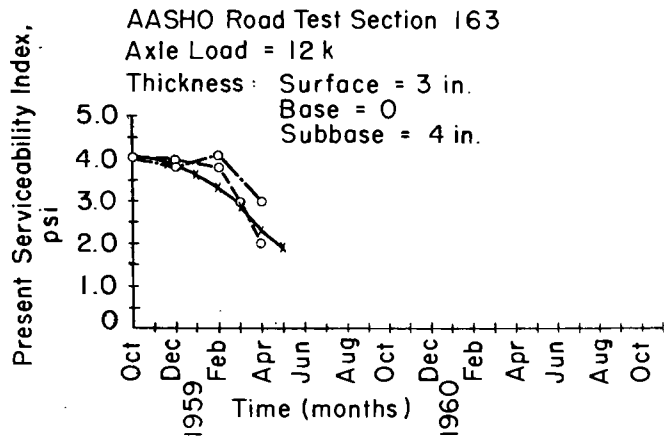


Fig A8.67. Observed versus calculated present serviceability index.

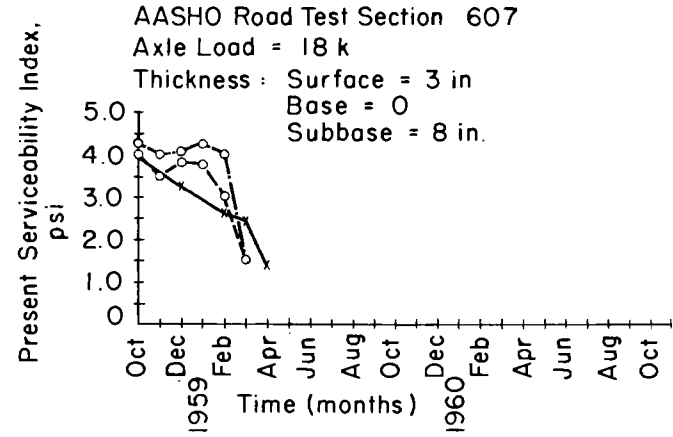


Fig A8.68. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—  
Observed (outer wheel) —○—  
Calculated —x—

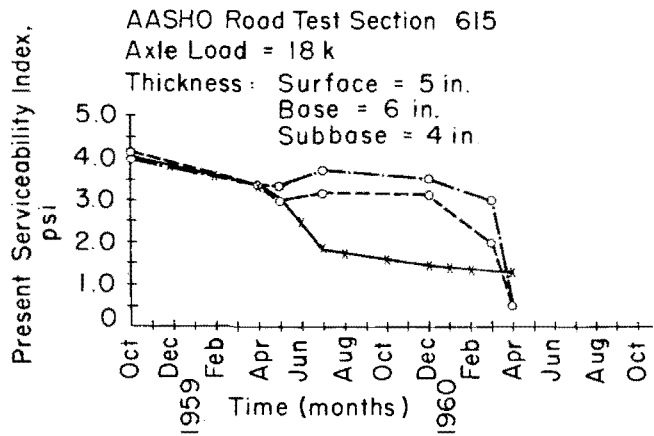


Fig A8.69. Observed versus calculated present serviceability index.

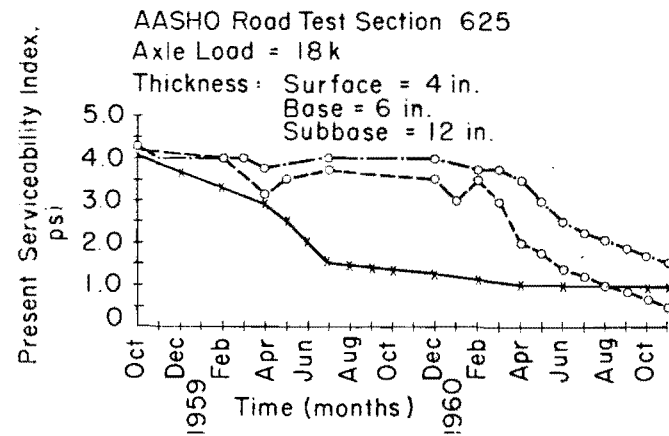


Fig A8.70. Observed versus calculated present serviceability index.

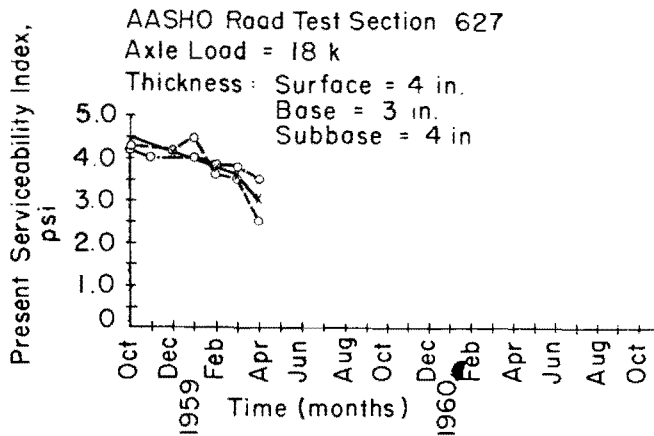


Fig A8.71. Observed versus calculated present serviceability index.

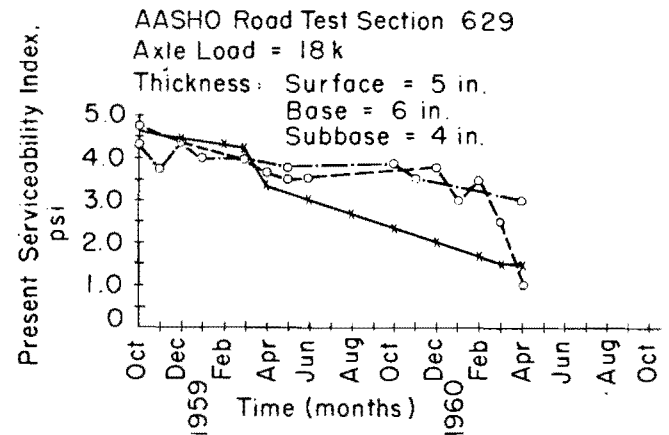


Fig A8.72. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—○—      Calculated  
 Observed (outer wheel) —○—○—

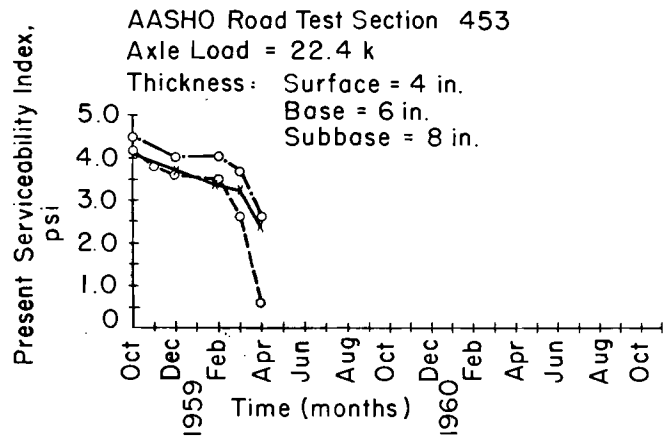


Fig A8.73. Observed versus calculated present serviceability index.

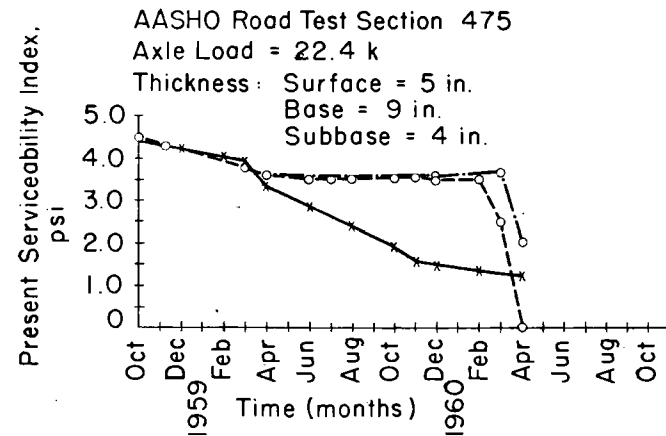


Fig A8.74. Observed versus calculated present serviceability index.

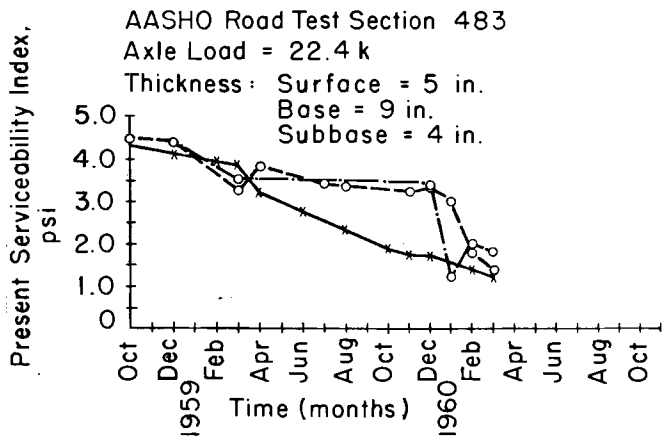


Fig A8.75. Observed versus calculated present serviceability index.

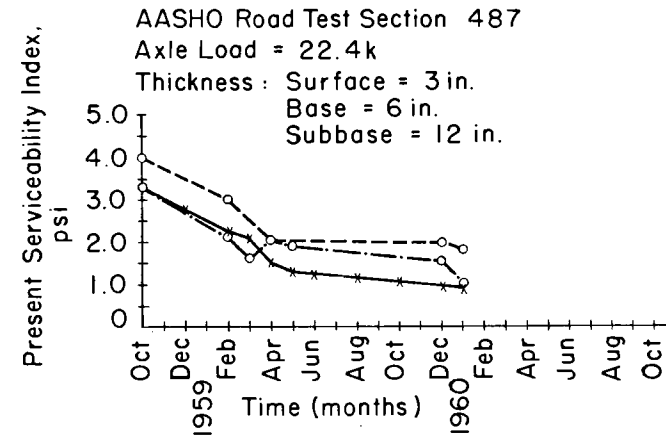


Fig A8.76. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—  
 Observed (outer wheel) —○—  
 Calculated —\*—

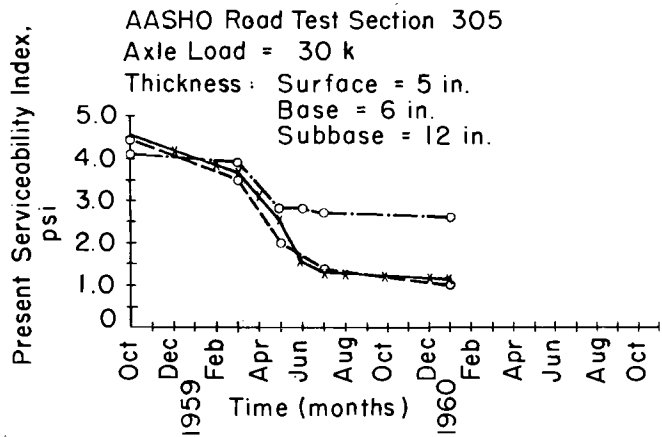


Fig A8.77. Observed versus calculated present serviceability index.

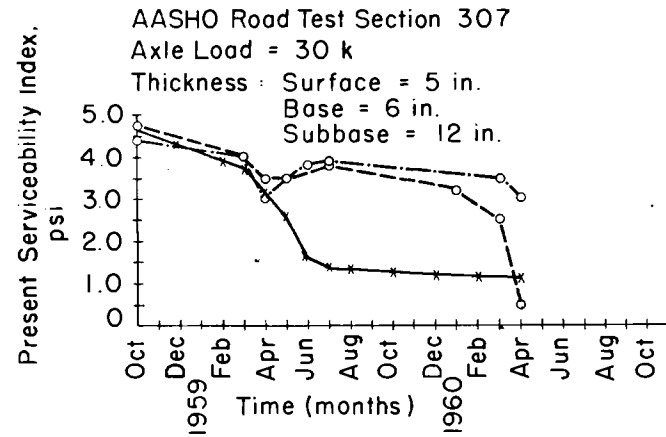


Fig A8.78. Observed versus calculated present serviceability index.

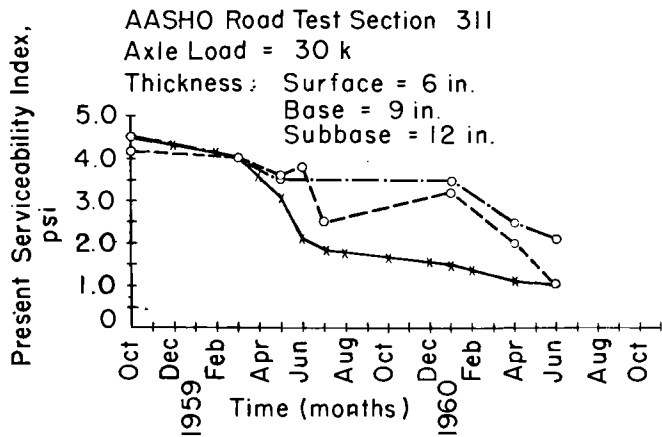


Fig A8.79. Observed versus calculated present serviceability index.

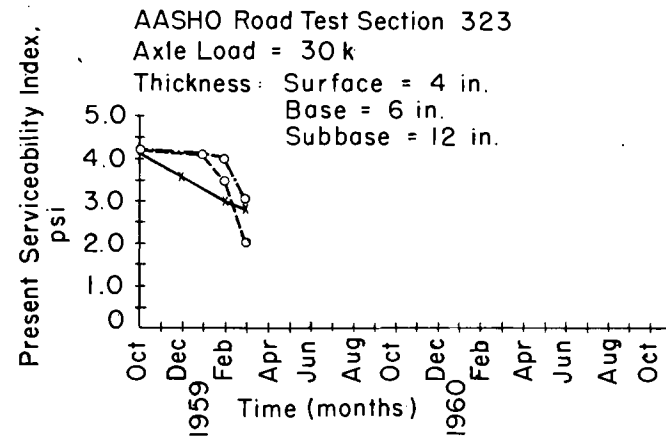


Fig A8.80. Observed versus calculated present serviceability index.

Observed (inner wheel) —○—  
 Observed (outer wheel) —○—  
 Calculated —\*—

APPENDIX 9

COMPUTER INPUT AND OUTPUT FOR EXAMPLE PROBLEMS



TABLE A9.1. EXAMPLE PROBLEM

THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE								
LAYER CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.	TRIAxIAL CLASS	
1	A ASPHALT CONCRETE	10.00	.44	3.00	3.00	50.00	.50	
2	B BASE MATERIAL	5.00	.14	6.00	6.00	50.00	1.00	
3	C SUBBASE MATERIAL	2.00	.11	8.00	8.00	50.00	3.70	
	SUBGRADE	0.00	0.00	0.00	0.00	0.00	5.60	
NUMBER OF OUTPUT PAGES DESIRED (8 DESIGNS/PAGE)								3
TOTAL NUMBER OF INPUT MATERIALS, EXCLUDING SUBGRADE								3
LENGTH OF THE ANALYSIS PERIOD (YEARS)								2.2
WIDTH OF EACH LANE (FEET)								12.0
REGIONAL FACTOR								1.0
SERVICEABILITY INDEX OF THE INITIAL STRUCTURE								4.2
SERVICEABILITY INDEX P1 AFTER AN OVERLAY								4.2
MINIMUM SERVICEABILITY INDEX P2								1.5
SWELLING CLAY PARAMETERS -- P2 PRIME								4.20
B1								0.0000
ONE-DIRECTION AUT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)								1370
ONE-DIRECTION AUT AT END OF ANALYSIS PERIOD (VEHICLES/DAY)								1371
ONE-DIRECTION 2-YR ACCUMULATED NO. OF EQUIVALENT 18-KIP AXLES								1102700
PROPORTION OF AUT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)								6.0
THE ROAD IS IN A RURAL AREA.								
MINIMUM TIME TO FIRST OVERLAY (YEARS)								.5
MINIMUM TIME BETWEEN OVERLAYS (YEARS)								.5
TIME TO FIRST SEAL COAT AFTER INITIAL OR OVERLAY CONST. (YEARS)								2.2
TIME BETWEEN SEAL COATS (YEARS)								2.2
MAX FUNDS AVAILABLE PER SQ. YD. FOR INITIAL DESIGN (DOLLARS)								15.00
MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)								60.0
MINIMUM OVERLAY THICKNESS (INCHES)								0.0
ACCUMULATED MAXIMUM DEPTH OF ALL OVERLAYS (INCHES)								8.0
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)								75.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)								1.80
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE O.O.D. (MILES)								0.00
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE N.O.O.D. (MILES)								0.00
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)								0.00
OVERLAY CONSTRUCTION TIME (HOURS/DAY)								0.0
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN O.O.D.								1
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN N.O.O.D.								2
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN O.O.D. (PERCENT)								0.00
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN N.O.O.D. (PERCENT)								0.00
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN O.O.D. (HOURS)								0.000
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN N.O.O.D. (HOURS)								0.000
AVERAGE APPROACH SPEED TO THE OVERLAY ZONE (MPH)								35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN O.O.D. (MPH)								35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN N.O.O.D. (MPH)								35.0
TRAFFIC MODEL USED IN THE ANALYSIS								3
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE MILE)								50.00
INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE MILE)								20.00
COST OF A SEAL COAT (DOLLARS/LANE MILE)								900.00
INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)								5.0

TABLE A9.2. EXAMPLE PROBLEM

FOR THE 3 LAYER DESIGN WITH THE FOLLOWING MATERIALS--

LAYER CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.	TRIAXIAL CLASS
1	A ASPHALT CONCRETE	10.00	.44	3.00	3.00	50.00	.50
2	B BASE MATERIAL	5.00	.14	6.00	6.00	50.00	1.00
3	C SUBBASE MATERIAL	2.00	.11	8.00	8.00	50.00	3.70
	SUBBGRADE	0.00	0.00	0.00	0.00	0.00	5.60

- 3 THE OPTIMAL DESIGN FOR THE MATERIALS UNDER CONSIDERATION--  
 FOR INITIAL CONSTRUCTION THE DEPTHS SHOULD BE  
 ASPHALT CONCRETE 3.00 INCHFS  
 BASE MATERIAL 6.00 INCHFS  
 SUBBASE MATERIAL 8.00 INCHFS  
 THE LIFE OF THE INITIAL STRUCTURE = .59 YEARS  
 THE OVERLAY SCHEDULE IS  
 2.50 INCH(ES) (INCLUDING 1 INCH LEVEL-UP) AFTER .59 YEARS.  
 TOTAL LIFE = 3.06YEARS/  
 THERE SHOULD NOT BE ANY SEAL COATS.

THE TOTAL COSTS PER SQ. YD. FOR THESE CONSIDERATIONS ARE

INITIAL CONSTRUCTION COST	2.111
TOTAL ROUTINE MAINTENANCE COST	.014
TOTAL OVERLAY CONSTRUCTION COST	.661
TOTAL USER COST DURING OVERLAY CONSTRUCTION	0.000
TOTAL SEAL COAT COST	0.000
SALVAGE VALUE	-1.135
TOTAL OVERALL COST	1.651

NUMBER OF FEASIBLE DESIGNS EXAMINED FOR THIS SET -- 1

AT THE OPTIMAL SOLUTION, THE FOLLOWING  
 BOUNDARY RESTRICTIONS ARE ACTIVE--

1. THE MINIMUM DEPTH OF LAYER 1
2. THE MAXIMUM DEPTH OF LAYER 1
3. THE MINIMUM DEPTH OF LAYER 2
4. THE MAXIMUM DEPTH OF LAYER 2
5. THE MINIMUM DEPTH OF LAYER 3
6. THE MAXIMUM DEPTH OF LAYER 3

A SUMMARY OF THE BEST DESIGN FOR EACH COMBINATION  
 OF MATERIALS, IN ORDER OF INCREASING TOTAL COST

DESIGN NUMBER	TOTAL COST
3	1.651

THE MATERIALS ASSOCIATED WITH EACH OF THE FOLLOWING DESIGN  
 NUMBERS DO NOT HAVE AT LEAST ONE FEASIBLE DESIGN.

1  
 2

TABLE A9.3. EXAMPLE PROBLEM

THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE							
LAYER CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.	TRIAxIAL CLASS
1	A ASPHALT CONCRETE	4.40	.44	2.00	6.00	50.00	.50
2	B BASE MATERIAL	1.40	.14	3.00	9.00	50.00	1.00
3	C SUB BASE MATERIAL	1.10	.11	4.00	16.00	50.00	3.70
	SUB GRADE	0.00	0.00	0.00	0.00	0.00	5.60
NUMBER OF OUTPUT PAGES DESIRED (A DESIGNS/PAGE)							3
TOTAL NUMBER OF INPUT MATERIALS, EXCLUDING SUBGRADE							3
LENGTH OF THE ANALYSIS PERIOD (YEARS)							2.2
WIDTH OF EACH LANE (FEET)							12.0
REGIONAL FACTOR							1.0
SERVICEABILITY INDEX OF THE INITIAL STRUCTURE							4.2
SERVICEABILITY INDEX P1 AFTER AN OVERLAY							4.2
MINIMUM SERVICEABILITY INDEX P2							1.5
SWELLING CLAY PARAMETERS -- P2 PRIME							4.20
R1							0.0000
ONE-DIRECTION ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)							1370
ONE-DIRECTION ADT AT END OF ANALYSIS PERIOD (VEHICLES/DAY)							1371
ONE-DIRECTION 2-YR ACCUMULATED NO. OF EQUIVALENT 18-KIP AXLES							1102700
PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)							6.0
THE ROAD IS IN A RURAL AREA.							
MINIMUM TIME TO FIRST OVERLAY (YEARS)							2.2
MINIMUM TIME BETWEEN OVERLAYS (YEARS)							2.2
TIME TO FIRST SEAL COAT AFTER INITIAL OR OVERLAY CONST. (YEARS)							2.2
TIME BETWEEN SEAL COATS (YEARS)							2.2
MAX FUNDS AVAILABLE PER SQ. YD. FOR INITIAL DESIGN (DOLLARS)							15.00
MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)							60.0
MINIMUM OVERLAY THICKNESS (INCHES)							0.0
ACCUMULATED MAXIMUM DEPTH OF ALL OVERLAYS (INCHES)							0.0
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)							75.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)							1.80
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE O.D. (MILES)							0.00
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE N.O.D. (MILES)							0.00
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)							0.00
OVERLAY CONSTRUCTION TIME (HOURS/DAY)							0.0
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN O.D.							1
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN N.O.D.							2
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN O.D. (PERCENT)							0.00
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN N.O.D. (PERCENT)							0.00
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN O.D. (HOURS)							0.000
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN N.O.D. (HOURS)							0.000
AVERAGE APPROACH SPEED TO THE OVERLAY ZONE (MPH)							35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN O.D. (MPH)							35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN N.O.D. (MPH)							35.0
TRAFFIC MODEL USED IN THE ANALYSIS							3
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE MILE)							50.00
INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE MILE)							20.00
COST OF A SEAL COAT (DOLLARS/LANE MILE)							900.00
INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)							5.0

TABLE A9.4. EXAMPLE PROBLEM

	1	2	3	4	5	6	7	8
MATERIAL ARRANGEMENT	ABC	ABC	ABC	ABC	ABC	AB	ABC	ABC
INIT. CONST. COST	1.022	1.022	1.022	1.022	1.022	1.022	1.022	1.022
OVERLAY CONST. COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
USER COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
SEAL COAT COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
ROUTINE MAINT. COST	.017	.017	.017	.017	.017	.017	.017	.017
SALVAGE VALUE	-.459	-.459	-.459	-.459	-.459	-.459	-.459	-.459
TOTAL COST	.580	.580	.580	.580	.580	.580	.580	.580
NUMBER OF LAYERS	3	3	3	3	3	2	3	3
LAYER DEPTH (INCHES)								
D(1)	4.50	4.25	4.00	3.00	2.75	5.50	3.75	3.50
D(2)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
D(3)	4.00	5.00	6.00	10.00	11.00		7.00	8.00
NO. OF PERF. PERIODS	1	1	1	1	1	1	1	1
PERF. TIME (YEARS)								
T(1)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
OVERLAY POLICY (INCH)								
(INCLUDING LEVEL-UP)								
NUMBER OF SEAL COATS	0	0	0	0	0	0	0	0
SEAL COAT SCHEDULE								
(YEARS)								

(Continued)

TABLE A9.4. (Continued)

	9	10	11	12	13	14	15	16
*****								
MATERIAL ARRANGEMENT	ABC	AB	ABC	ABC	ABC	ABC	ABC	ABC
INIT. CONST. COST	1.022	1.024	1.024	1.024	1.024	1.024	1.024	1.024
OVERLAY CONST. COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
USER COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
SEAL COAT COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
ROUTINE MAINT. COST	.017	.017	.017	.017	.017	.017	.017	.017
SALVAGE VALUE	-.459	-.460	-.460	-.460	-.460	-.460	-.460	-.460
*****								
TOTAL COST	.580	.580	.580	.580	.580	.580	.580	.580
*****								
NUMBER OF LAYERS	3	2	3	3	3	3	3	3
*****								
LAYER DEPTH (INCHES)								
D(1)	3.25	5.75	4.75	4.50	4.25	3.75	3.50	3.25
D(2)	9.00	8.25	8.25	8.25	8.25	8.25	8.25	8.25
D(3)	9.00		4.00	5.00	6.00	8.00	9.00	10.00
*****								
NO. OF PERF. PERIODS	1	1	1	1	1	1	1	1
*****								
PERF. TIME (YEARS)								
T(1)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
*****								
OVERLAY POLICY (INCH)								
(INCLUDING LEVEL-UP)								
*****								
NUMBER OF SEAL COATS	0	0	0	0	0	0	0	0
*****								
SEAL COAT SCHEDULE								
(YEARS)								
*****								

(Continued)

TABLE A9.4. (Continued)

SUMMARY OF THE BEST DESIGN STRATEGIES  
IN ORDER OF INCREASING TOTAL COST

	17	18	19	20	21	22	23	24
MATERIAL ARRANGEMENT	ABC	ABC	ABC	AB	ABC	ABC	ABC	ABC
INIT. CONST. COST	1.024	1.024	1.024	1.025	1.025	1.025	1.025	1.025
OVERLAY CONST. COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
USER COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
SEAL COAT COST	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
ROUTINE MAINT. COST	.017	.017	.017	.017	.017	.017	.017	.017
SALVAGE VALUE	-.460	-.460	-.460	-.460	-.460	-.460	-.460	-.460
TOTAL COST	.580	.580	.580	.581	.581	.581	.581	.581
NUMBER OF LAYERS	3	3	3	2	3	3	3	3
LAYER DEPTH (INCHES)								
D(1)	3.00	2.75	4.00	6.00	5.00	4.75	4.50	4.25
D(2)	8.25	8.25	8.25	7.50	7.50	7.50	7.50	7.50
D(3)	11.00	12.00	7.00		4.00	5.00	6.00	7.00
NO. OF PERF. PERIODS	1	1	1	1	1	1	1	1
PERF. TIME (YEARS)								
T(1)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
OVERLAY POLICY (INCH) (INCLUDING LEVEL-UP)								
NUMBER OF SEAL COATS	0	0	0	0	0	0	0	0
SEAL COAT SCHEDULE (YEARS)								

THE TOTAL NUMBER OF FEASIBLE DESIGNS CONSIDERED WAS 993

TABLE A9.5. EXAMPLE PROBLEM

THE CONSTRUCTION MATERIALS UNDER CONSIDERATION ARE							
LAYER CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.	
1	A ASPHALT CONCRETE	10.00	.90	3.00	3.00	50.00	
2	B BASE MATERIAL	5.00	.45	6.00	6.00	50.00	
3	C SUBBASE MATERIAL	2.00	.30	8.00	8.00	50.00	
	SUBGRADE	0.00	.17	0.00	0.00	0.00	
NUMBER OF OUTPUT PAGES DESIRED (A DESIGNS/PAGE)							3
TOTAL NUMBER OF INPUT MATERIALS EXCLUDING SUBGRADE							3
LENGTH OF THE ANALYSIS PERIOD (YEARS)							2.2
WIDTH OF EACH LANE (FEET)							12.0
DISTRICT TEMPERATURE CONSTANT							30.0
SERVICEABILITY INDEX OF THE INITIAL STRUCTURE							4.2
SERVICEABILITY INDEX P1 AFTER AN OVERLAY							4.2
MINIMUM SERVICEABILITY INDEX P2							1.5
SWELLING CLAY PARAMETERS -- P2 PRIME							4.20
B1							0.0000
ONE-DIRECTION ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)							1370
ONE-DIRECTION ADT AT END OF ANALYSIS PERIOD (VEHICLES/DAY)							1371
ONE-DIRECTION 2-YR ACCUMULATED NO. OF EQUIVALENT 18-KIP AXLES							1102700
PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)							6.0
THE ROAD IS IN A RURAL AREA.							
MINIMUM TIME TO FIRST OVERLAY (YEARS)							.5
MINIMUM TIME BETWEEN OVERLAYS (YEARS)							.5
TIME TO FIRST SEAL COAT AFTER INITIAL OR OVERLAY CONST. (YEARS)							2.2
TIME BETWEEN SEAL COATS (YEARS)							2.2
MAX FUNDS AVAILABLE PER SQ. YD. FOR INITIAL DESIGN (DOLLARS)							15.00
MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)							60.0
MINIMUM OVERLAY THICKNESS (INCHES)							0.0
ACCUMULATED MAXIMUM DEPTH OF ALL OVERLAYS (INCHES)							8.0
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)							75.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)							1.80
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE O.D. (MILES)							0.00
C.L. DISTANCE OVER WHICH TRAFFIC IS SLOWED IN THE N.O.D. (MILES)							0.00
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)							0.00
OVERLAY CONSTRUCTION TIME (HOURS/DAY)							0.0
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN O.D.							1
NUMBER OF OPEN LANES IN RESTRICTED ZONE IN N.O.D.							2
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN O.D. (PERCENT)							0.00
PROPORTION OF VEHICLES STOPPED BY ROAD EQUIPMENT IN N.O.D. (PERCENT)							0.00
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN O.D. (HOURS)							0.000
AVERAGE TIME STOPPED BY ROAD EQUIPMENT IN N.O.D. (HOURS)							0.000
AVERAGE APPROACH SPEED TO THE OVERLAY ZONE (MPH)							35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN O.D. (MPH)							35.0
AVERAGE SPEED THROUGH OVERLAY ZONE IN N.O.D. (MPH)							35.0
TRAFFIC MODEL USED IN THE ANALYSIS							3
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE MILE)							50.00
INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE MILE)							20.00
COST OF A SEAL COAT (DOLLARS/LANE MILE)							900.00
INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)							5.0

(Continued)

TABLE A9.5. (Continued)

FOR THE 3 LAYER DESIGN WITH THE FOLLOWING MATERIALS--

LAYER CODE	MATERIALS NAME	COST PER CY	STR. COEFF.	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.
1	A ASPHALT CONCRETE	10.00	.90	3.00	3.00	50.00
2	B BASE MATERIAL	5.00	.45	6.00	6.00	50.00
3	C SUBBASE MATERIAL	2.00	.30	8.00	8.00	50.00
	SUBGRADE	0.00	.17	0.00	0.00	0.00

3 THE OPTIMAL DESIGN FOR THE MATERIALS UNDER CONSIDERATION--  
 FOR INITIAL CONSTRUCTION THE DEPTHS SHOULD BE  
 ASPHALT CONCRETE 3.00 INCHES  
 BASE MATERIAL 6.00 INCHES  
 SUBBASE MATERIAL 8.00 INCHES

THE SCI OF THE INITIAL STRUCTURE = 1.469  
 THE LIFE OF THE INITIAL STRUCTURE = .53 YEARS  
 THE OVERLAY SCHEDULE IS  
 3.00 INCH(ES) (INCLUDING 1 INCH LEVEL-UP) AFTER .53 YEARS.  
 TOTAL LIFE = 2.31 YEARS/  
 THERE SHOULD NOT BE ANY SEAL COATS.

THE TOTAL COSTS PER SQ. YD. FOR THESE CONSIDERATIONS ARE

INITIAL CONSTRUCTION COST	2.111
TOTAL ROUTINE MAINTENANCE COST	.014
TOTAL OVERLAY CONSTRUCTION COST	.794
TOTAL USER COST DURING OVERLAY CONSTRUCTION	0.000
TOTAL SEAL COAT COST	0.000
SALVAGE VALUE	-1.198
TOTAL OVERALL COST	1.721

NUMBER OF FEASIBLE DESIGNS EXAMINED FOR THIS SET -- 1

AT THE OPTIMAL SOLUTION, THE FOLLOWING BOUNDARY RESTRICTIONS ARE ACTIVE--

1. THE MINIMUM DEPTH OF LAYER 1
2. THE MAXIMUM DEPTH OF LAYER 1
3. THE MINIMUM DEPTH OF LAYER 2
4. THE MAXIMUM DEPTH OF LAYER 2
5. THE MINIMUM DEPTH OF LAYER 3
6. THE MAXIMUM DEPTH OF LAYER 3

A SUMMARY OF THE BEST DESIGN FOR EACH COMBINATION OF MATERIALS, IN ORDER OF INCREASING TOTAL COST

DESIGN NUMBER	TOTAL COST
3	1.721

THE MATERIALS ASSOCIATED WITH EACH OF THE FOLLOWING DESIGN NUMBERS DO NOT HAVE AT LEAST ONE FEASIBLE DESIGN.



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