TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CFHR 1-8-69-123-18		

4. Title and Subtitle	5. Report Date
"Probabilistic Design Concepts Applied to Flexible Pavement System Design"	May 1973
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Michael I. Darter and W. Ronald Hudson	Research Report 123-18
9. Performing Organization Name and Address	10. Work Unit No.
Center for Highway Research The University of Texas at Austin Austin, Texas 78712	11. Contract or Grant No. Research Study 1-8-69-123
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Texas Highway Department 11th & Brazos Austin, Texas 78701	Interim May 1972 - May 1973
	14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with DOT, FHWA.
Research Study Title: "A System Analysis of Pavement Design and Research Implementation"

16. Abstract

A major problem in pavement design has been the inherent uncertainty and variation of the design parameters and models. Empirical safety factors and judgement factors have been applied to "adjust" for the uncertainties involved, but the result has been much overdesign and underdesign. A method was needed which would consider the variations and uncertainties of pavement design quantitatively and make it possible to design for a specific level of reliability. As a basic start in the solution of the problem, a theory and procedures were developed, based on classical reliability theory, to apply probabilistic design concepts to flexible pavement system design. The probabilistic theory and procedures have been both practical and useful as they have been implemented into the pavement design operations of the Texas Highway Department. Original implementation was with the deterministic FPS-7 program, which was modified to include some probabilistic design capability and renamed FPS-11. That version, which has been used by ten districts of the Texas Highway Department since late 1971, has been further developed to include variations occurring in individual pavement layers and subgrade, and the consideration of traffic forecasting error. The overlay mode was improved by making it possible to "adjust" the performance model to a specific pavement by considering its past performance history, and the new program was named FPS-13 (CFHR).

17. Key Words	18. Distribution Statement
flexible pavements, pavement design, probability, stochastic, reliability, pavement systems, variability, overlay	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	202 (Printed)	

Form DOT F 1700.7 (8-69)

PROBABILISTIC DESIGN CONCEPTS APPLIED TO FLEXIBLE PAVEMENT SYSTEM DESIGN

by

Michael I. Darter
W. Ronald Hudson

Research Report Number 123-18

A System Analysis of Pavement Design
and Research Implementation

Research Project 1-8-69-123

conducted for

The Texas Highway Department

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

by the

Highway Design Division
Texas Highway Department

Texas Transportation Institute
Texas A&M University

Center for Highway Research
The University of Texas at Austin

May 1973

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report documents the work performed from 1970 to 1973 in developing and implementing a probabilistic design approach in the Texas flexible pavement design system. This work has resulted in the development of program FPS-11, which has been in use by the Texas Highway Department since 1971. The report also documents another version, FPS-13 (CFHR), of the probabilistic design method, which has several capabilities beyond those of the FPS-11 program.

This is the eighteenth in a series of reports that describe the work accomplished in the project entitled "A System Analysis of Pavement Design and Research Implementation." The project is a long-range comprehensive research program to develop a system analysis of pavement design and management. The project is conducted in cooperation with the Federal Highway Administration, Department of Transportation.

Special thanks and appreciation are extended to Mr. James L. Brown for his assistance and helpful suggestions throughout the entire study. Appreciation is expressed also to Dr. B. Frank McCullough for a critical review of the report; to Mr. Frank Scrivner, Dr. Ramesh K. Kher, and Mr. Frank Carmichael for their assistance on the project; to Mrs. Marie Fisher for typing and other help with the report; and to Mr. Arthur Frakes for editing the manuscript.

Michael I. Darter
W. Ronald Hudson

May 1973

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. March 1970.

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. March 1970.

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. August 1970.

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. February 1971.

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. November 1970.

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. March 1971.

Report No. 123-6A, "Calculation of the Elastic Moduli of a Two Layer Pavement System from Measured Surface Deflections, Part II," by Frank H. Scrivner, Chester H. Michalak, and William M. Moore, is a supplement to Report No. 123-6 and describes the effect of a change in the specified location of one of the deflection points. December 1971.

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. April 1971.

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. August 1971.

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. April 1972.

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 stuctural 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. January 1972.

Report No. 123-11, "Flexible Pavement System Computer Program Documentation," by Dale L. Schafer, provides documentation and an easily updated documentation system for the computer program FPS-9. April 1972.

Report No. 123-12, "A Pavement Feedback Data System," by Oren G. Strom, W. Ronald Hudson, and James L. Brown, defines a data system to acquire, store, and analyze performance feedback data from in-service flexible pavements. May 1972.

Report No. 123-13, "Benefit Analysis for Pavement Design System," by Frank McFarland, presents a method for relating motorist's costs to the pavement serviceability index and a discussion of several different methods of economic analysis. April 1972.

Report No. 123-14, "Prediction of Low-Temperature and Thermal-Fatigue Cracking in Flexible Pavements," by Mohamed Y. Shahin and B. Frank McCullough, describes a design system for predicting temperature cracking in asphalt concrete surfaces. August 1972.

Report No. 123-15, "FPS-11 Flexible Pavement System Computer Program Documentation," by Hugo E. Orellana, gives the documentation of the computer program FPS-11. October 1972.

Report No. 123-16, "Fatigue and Stress Analysis Concepts for Modifying the Rigid Pavement Design System," by Piti Yimprasert and B. Frank McCullough, describes the fatigue of concrete and stress analyses of rigid pavement. October 1972.

Report No. 123-17, "The Optimization of a Flexible Pavement System Using Linear Elasticity," by Danny Y. Lu, Chia Shun Shih and Frank H. Scrivner, describes the integration of the current Flexible Pavement System computer program and Shell Oil Company's program BISTRO, for elastic layered systems, with special emphasis on economy of computation and evaluation of structural feasibility of materials. March 1973.

Report No. 123-18, "Probabilistic Design Concepts Applied to Flexible Pavement System Design," by Michael I. Darter and W. Ronald Hudson, describes the development and implementation of the probabilistic design approach and its incorporation into the Texas flexible pavement design system for new construction and asphalt concrete overlay. May 1973.

ABSTRACT

A major problem which currently exists in pavement design is the consideration in design of the inherent uncertainty and variation of the design parameters and models. Empirical safety factors and judgement factors have been applied in the past to "adjust" for the many uncertainties involved. These safety factors usually do not depend upon the associated magnitude of variations and, therefore, have resulted in much overdesign and underdesign. A need was found to develop a method which would consider the associated variations and uncertainties of pavement design on a quantitative basis whereby designs can be made to specified levels of adequacy or reliability.

The theory and procedures were developed, based upon classical reliability theory, to apply probabilistic design concepts to flexible pavement system design as a basic start in the solution of the problem. The probabilistic approach makes it possible to design for a desired level of reliability through consideration of the variabilities and uncertainties associated with pavement design. The theory was applied to the Texas flexible pavement system (FPS), which has heretofore functioned as a deterministic method.

The probabilistic approach considers the following variations:

(1) variations within a design project length,

(2) variations between design values and actual as-constructed values, and

(3) variations due to lack-of-fit of the design models.

Approximate estimates of these variations were made for the specific design parameters and models of the Texas FPS, which included pavement layer and subgrade stiffness, pavement layer thickness, pavement initial serviceability, temperature parameter, performance model, deflection model, and traffic forecasting.

The probabilistic theory and procedures have been shown to be both practical and useful as they have been implemented into the daily pavement design operations of the Texas Highway Department. Original implementation was with the deterministic FPS-7 program. This program was modified to

include some probabilistic design capability and was renamed FPS-11. This version has been used by ten districts of the Texas Highway Department since late 1971. The FPS-11 program has been further developed to include variations occurring in individual pavement layers and subgrade, and the consideration of traffic forecasting error. The overlay mode of the program was also improved by making it possible to "adjust" the performance model to a specific pavement by considering its past performance history. This new program is named FPS-13 (CFHR) and includes these inputs, which add new capabilities to the system. Design examples are given for actual projects and the results illustrate the potential of the probabilistic approach.

Recommended design reliability levels were established based upon specific criteria of the pavements being designed. This will assist in producing uniform reliability in pavement design and minimizing costs.

Basically, the probabilistic pavement design approach developed in this study provides a first-order approach to a useful and implementable method to quantify adequacy of designs by considering uncertainties and variations and designing for specified levels of reliability.

KEY WORDS: flexible pavements, pavement design, probability, stochastic, reliability, pavement systems, variability, overlay.

SUMMARY

A theory and procedures are developed to apply probabilistic design concepts to the Texas flexible pavement design system. This allows the design engineer to design for a specific level of reliability, considering traffic load associated distress. These concepts were implemented into the deterministic FPS-7 program, which was modified to include the probabilistic concepts. The new program is named FPS-11 and since 1971 has been used for flexible pavement design by the Texas Highway Department.

Another program, FPS-13 (CFHR), is also documented in this report. It has more capabilities and considers more variations than the FPS-11 program. Design examples are given along with recommended levels of reliability for highways with various functions and characteristics.

The method is practical yet soundly based upon theory and has been shown to be implementable for actual design usage by the Texas Highway Department.

IMPLEMENTATION STATEMENT

At this time, many of the results of this study have been implemented into the flexible pavement design system of the Texas Highway Department and to this extent the study results represent unusual implementation success.

The need for consideration of the many variations and uncertainties in design such as material strengths, load estimation, and environment, was established in late 1970 after six months of trial implementation of the FPS-7 design program. The basic theory was then developed and variations were quantified and incorporated into the design system in a new program named FPS-11. This system has undergone implementation in the Texas Highway Department since late 1971 and is currently in use by the Department.

Another program, FPS-13 (CFHR), is also documented in this report and represents an expansion of capabilities of the FPS-11 program such as consideration of variation of individual pavement layer stiffness and thickness and subgrade stiffness, and traffic load variations. Probabilistic concepts have also been applied to the overlay mode of FPS.

Therefore, the basic results developed in this study are now in use in flexible pavement design by the Texas Highway Department. Other results should be carefully considered by the Department for possible incorporation into the FPS-11 program.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

One of the most pressing problems facing pavement engineers, as stated by the FHWA-HRB Advisory Committee of the "Workshop on the Structural Design of Asphalt Concrete Pavement Systems" in 1970, is the need for applying probabilistic or stochastic concepts to pavement design. This need exists because of the inherent uncertainty and variability of the design parameters and of the design models. The report from the workshop concisely describes this problem as follows:

> So that designers can better evaluate the reliability of a particular design, it is necessary to develop a procedure that will predict variations in the pavement system response due to statistical variations in the input variables, such as load, environment, pavement geometry, and material properties including the effects of construction and testing variables. As part of this research it will be necessary to include a significance study to determine the relative effect on the system response of variations in the different input variables. (Ref 2).

Purpose

The general purpose of this research effort is to formulate the necessary concepts and to develop a procedure such as described above and also to apply the method to a current (existing) flexible pavement design system. The probabilistic design concepts have been specifically applied to the Texas flexible pavement system (FPS), but the concepts are presented in a general format to be applicable to other design procedures. The underlying reasons for formulating a probability-based design procedure is to make the design process responsive to the actual existing variabilities and uncertainties associated with the design, construction, and performance of flexible pavements. Such a procedure provides a rational means of designing for varying levels of reliability. The levels can be set depending upon the function of the pavement, that is, Interstate, primary, or secondary highway and city street. Additional economy should be obtained by varying the design reliability since not all pavement types require the same level. This design approach in conjunction with the "systems approach" makes the design process closer to reality than the present deterministic method, therefore upgrading the current procedure. Specifically, it will allow the designer to

(1) design for a given level of reliability or probability of success,
(2) quantify design risk,
(3) optimize design results considering variability, and
(4) evaluate economic feasibility of improved construction control techniques.

General Background

The nature of practically all of the factors involved in the pavement design system is stochastic (probabilistic). Due to lack of knowledge and information and uncertain future social-economic conditions, many design factors cannot be exactly predicted; and also inherent along the roadway variations in pavement strength due to nonhomogeneous materials and variable construction practices exist. This uncertainty in prediction and natural variations of important parameters result in variations in pavement system performance and in some supposedly identical pavement sections "failing" before others. This variable nature of failure or distress may be observed along every in-service pavement. Essentially, this uncertainty results in some amount of early failure before the average life has ended. The analysis of these types of variabilities and uncertainties can be handled through a so-called probabilistic or stochastic approach.

In structural and foundation design, the various uncertainties have been provided for by empirical safety factors. This generally has resulted in few failures, but has probably resulted many times in an overdesign or sometimes underdesign, depending on the magnitude of variations and the level of applied safety factors. The use of arbitrarily large safety factors in pavement design is further questioned because human lives are not endangered in pavement wear-out-failure the way they are if a building or bridge fails. The minimization of costs while satisfying the performance requirements is the objective of pavement design. Using the probabilistic approach, it is possible to quantify the design risk and to design for a specified level of reliability.

The concepts and procedures are developed for general application to pavement design. The method has been applied specifically to the Texas flexible pavement (design) system denoted by FPS. This is a computerized working system which now contains many of the stochastic concepts developed in this research work and is currently being implemented by the Texas Highway Department. The system was initially developed by Scrivner, Moore, McFarland, and Carey (Ref 86) in 1968 and has undergone trial implementation since 1969 under a three-agency joint research project between the Texas Highway Department (Highway Design Division), Texas Transportation Institute, and the Center for Highway Research.

General Approach

A brief description of the approach developed in this study for the application of probabilistic design concepts to pavement design is given to provide an overview of the theory.

The following conceptual equation includes some of the major factors which cause loss of serviceability of a pavement:

$$\begin{aligned} \text{serviceability loss} \quad = \quad & f(\text{traffic loadings}) \\ & + f(\text{subgrade shrink/swell}) \\ & + f(\text{thermal cracking}) + \ldots . \end{aligned}$$

The probabilistic theory developed in this study is limited to the consideration of serviceability loss due to traffic loadings only. The other factors are also important and the theory should be expanded to consider them in future work.

Stochastic Nature of Design Variables. All pavement design methods which consider loss of serviceability due to repeated traffic loadings (fatigue) ultimately require the determination of two parameters. These are (1) the prediction of traffic loads to be applied, n , and (2) the prediction of the allowable load applications the pavement/subgrade system can withstand to minimum acceptable serviceability N . The allowable applications N depends upon many design factors such as pavement thickness (T), material properties (M), and environment (E). These factors are illustrative of the multitude of factors which affect the multivariate N .

The actual applied load applications, n , depends upon many factors such as average daily traffic (A), percent trucks (t), axle load distribution (L), and equivalency factors (F), estimated for a certain analysis period. These factors are illustrative of the multitude of factors which are involved with the determination of n . To illustrate the process we can show with appropriate models that the N and n are functionally related to the several design variables as follows:

$$N = f(T, M, E \ldots)$$
$$n = f(A, t, L, F \ldots)$$

In existing design methods N and n are assumed to be determined precisely by the input variables. In reality, there is considerable variability associated with each design factor. The three basic types of variations associated with flexible pavement design parameters can be considered as (1) variation within a design project length, (2) variation between design and actual values, and (3) variation due to lack-of-fit of the design models. The purpose of this study is to develop a method of accounting for this variability in the design process. As a first step, estimates of these variations of the design parameters were made for in-service highway pavements in Texas.

Since all the factors are variable, it therefore follows that $f(T, M, E \ldots)$ and $f(A, t, L, F \ldots)$ are themselves stochastic variables determined by the combined statistical characteristics of the design factors. As outlined herein N and n have been found to be distributed approximately log normal.

Variance Models of N and n. Since N and n are multivariates and stochastic in nature, the variance of each must be determined before the reliability theory can be applied effectively. This is accomplished herein by using the partial derivative method. The estimates of variance for $\log N$ and $\log n$ thus determined can now be used in the next phase, where the reliability function is derived.

Reliability Function. Reliability (R), for pavements is defined herein as the probability that N will exceed n . This is synonymous with the statement that reliability is the probability that the serviceability level of the pavement will not fall below the minimum acceptable level before the design performance period is over:

$$R = P\,(N > n)$$

By assuming N and n to be log normally distributed and applying statistical theory, the following relationship may be derived.

$$\overline{\log N_R} = \overline{\log n} + Z_R \sqrt{s_{\log N}^{\;2} + s_{\log n}^{\;2}}$$

where

$\overline{\log N_R}$ = average number of equivalent 18-kip single-axle load applications to be used for design at level of reliability (R),

$\overline{\log n}$ = average traffic forecast of 18-kip single-axle load applications,

Z_R = standardized normal deviate from normal distribution tables with mean zero and variance of one for given level of reliability (R).

This reliability function may be used either to design a pavement for a specific reliability level or to analyze the reliability of a given pavement.

This basic approach for applying probabilistic theory to pavement design is developed and presented in this study. Using the resulting computer program, the pavement designer may now design a pavement for a specified reliability level considering traffic associated loadings only. The inputs to the probabilistic design approach are the means and standard deviations of the design parameters. The following design factors are considered in the program as stochastic: pavement layer and subgrade stiffness coefficients, pavement layer thickness, initial serviceability, temperature parameter, lack-of-fit of performance and deflection model, design average daily traffic, percent trucks, axle factor, and combined axle-load distribution and load equivalency factors. These factors represent all design parameters related to the traffic load associated structural design models of the program.

The results of this study will be presented in the following sequence:

Chapter 2 - Discussion of the overall concept of pavement systems, the FPS working system, and pavement systems reliability.

Chapter 3 - Development of the necessary theory and concepts to apply probabilistic theory to pavement systems design.

Chapter 4 - Description and quantification of the stochastic nature of the various design factors and design models.

Chapter 5 - Consideration of the uncertainties associated with traffic load forecasting.

Chapter 6 - Details of the application of probabilistic concepts to FPS, and to the overlay design mode, including a sensitivity study.

Chapter 7 - Detailed examples of pavement design problems using the new computerized system, and illustrations of the quantitative effect of variations of design parameters on pavement performance.

Chapter 8 - Deals specifically with the problem of selecting design reliability and gives recommendations.

Chapter 9 - Summarizes results and gives recommendations for implementation and future work.

CHAPTER 2. PAVEMENT SYSTEMS RELIABILITY

This chapter describes the pavement systems concept, the FPS working system, and pavement systems reliability.

Pavement Systems Concept

A systems approach to pavement design does not provide new technical knowledge for the design process, but rather assists in the organization, coordination, and optimization of pavement design. The systems approach has been used extensively in areas such as electronics, communications, and aerospace. The development of a systems approach came about in pavement design as

(1) engineers and researchers sought for improved methods of pavement design and research implementation;

(2) the computer became more available;

(3) the need to consider the operational problems (user-delay) caused by pavement maintenance operations increased; and

(4) a greater need to optimize design occurred due to scarcity of highway funds and the ever expanding needs of the highway system.

Through many years of highway construction, operation, and research experience, it has become evident that it is extremely difficult to construct a smooth pavement and to keep it that way throughout a rather long lifetime and to accomplish these goals at a minimum overall cost. The nature of a pavement structure is extremely complex, and the environment within which it must perform is also complex. Many diverse loadings are applied to a pavement. The use of systems engineering concepts provides a coordinated framework to synthesize the overall problem and to optimize the design process. The pavement system attempts to consider the design, construction, operation, maintenance, salvage, and performance of a pavement throughout the analysis period in arriving at an optimum design strategy. There is an increasing demand upon the engineer to optimize his design and at the same time to keep the reliability level high. To accomplish this difficult goal, he must consider as many factors and interactions between factors as possible that may affect the system performance. A systems approach would generate many possible alternative design strategies and evaluate them using sound economic analysis and engineering decision criteria.

The systems approach can also be helpful in the development and continual improvement of a working pavement design system. Through implementation, sensitivity analysis, and feedback from the field, the most significant weak points of the system will become evident. This process is illustrated by the conceptual diagram developed by Hudson, Kher, and McCullough (Ref 44) shown in Fig 2.1. Therefore, research priorities can be determined and projects can be funded which have goals that are designed to fulfill specific needs for improving the system.

This concept was first applied to pavements in the latter part of the 1960's by several investigators. Hutchinson, in 1966, suggested a conceptual framework for pavement design decisions (Ref 48). NCHRP Project 1-10 gave significant emphasis to the systems approach; its results were published in 1968 by Hudson et al (Refs 42 and 43). Also in 1968, Hutchinson and Haas (Ref 47) and Lemer and Moavenzadeh (Ref 69) published concepts relating to systems analysis of the highway pavement design process.

The first computerized working pavement design system was developed by Scrivner et al (Ref 86) in 1968 for the Texas Highway Department. This initial system was named Flexible Pavement System 1 or FPS-1.

Since these initial efforts, several investigators have published concepts and developed working systems, such as the development of SAMP through NCHRP Project 1-10/1 by Hudson and McCullough (Ref 41) and work by Lemer and Moavenzadeh (Ref 60) and Moavenzadeh (Ref 70); Kher, Hudson and McCullough (Refs 55 and 56); Phang (Ref 77) and Peterson et al (Ref 76). The basic Texas FPS-1 system has been further developed since 1968 and has progressed into actual implementation in the Texas Highway Department by a joint research project (Project 123) between the Texas Highway Department, the Texas Transportation Institute, and the Center for Highway Research. Several publications have resulted from this project (such as Refs 8, 37, 40, 49, 54, 84, and 96), and the work reported here also initiates from this project.

The basic concept of the systems approach applied to pavement design can be represented by the conceptual flow chart shown in Fig 2.2. Important components of this conceptual pavement design system for purposes of pavement

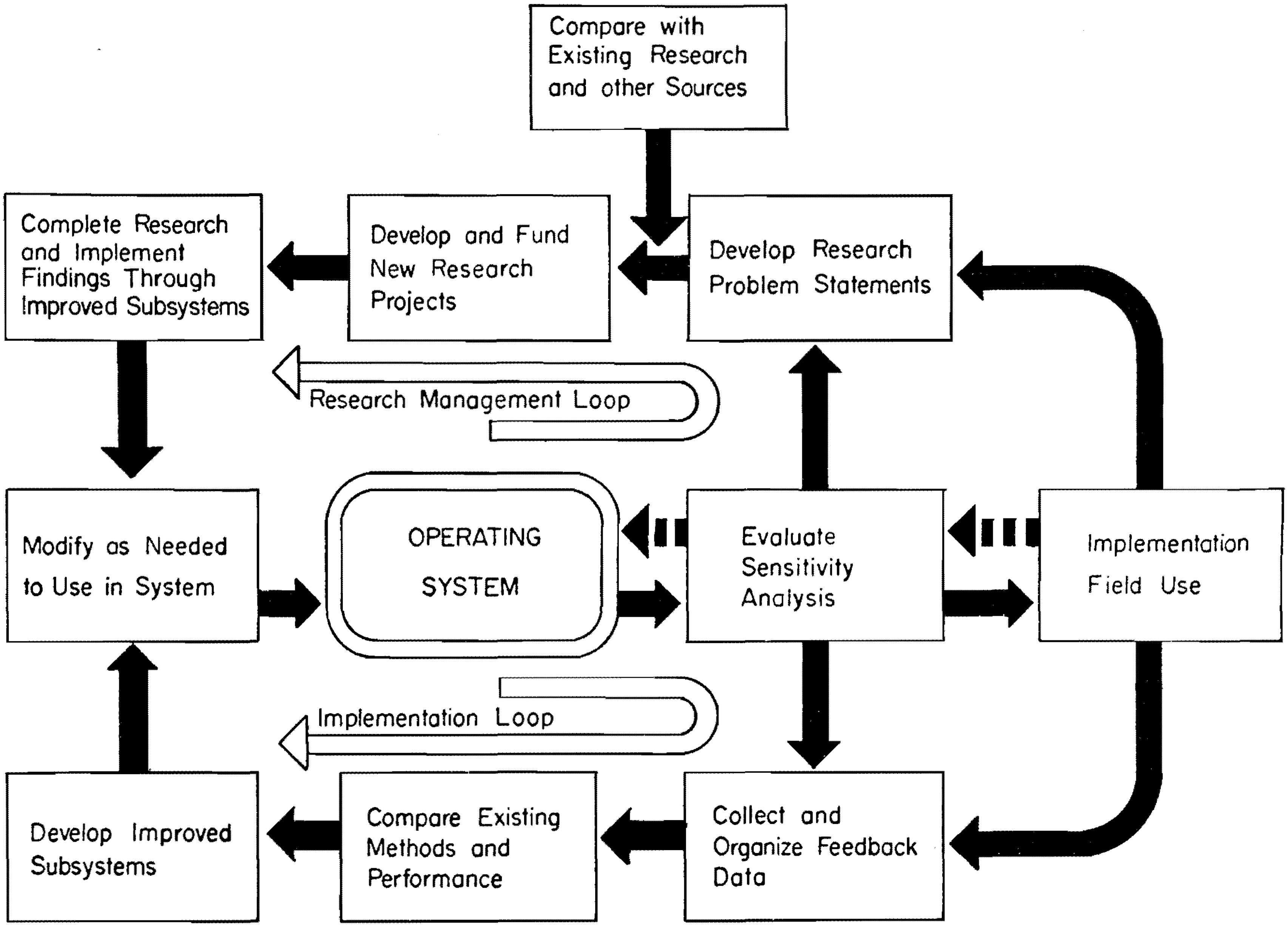


Fig 2.1. Iterative improvement pattern of pavement design and management system (Ref 44).

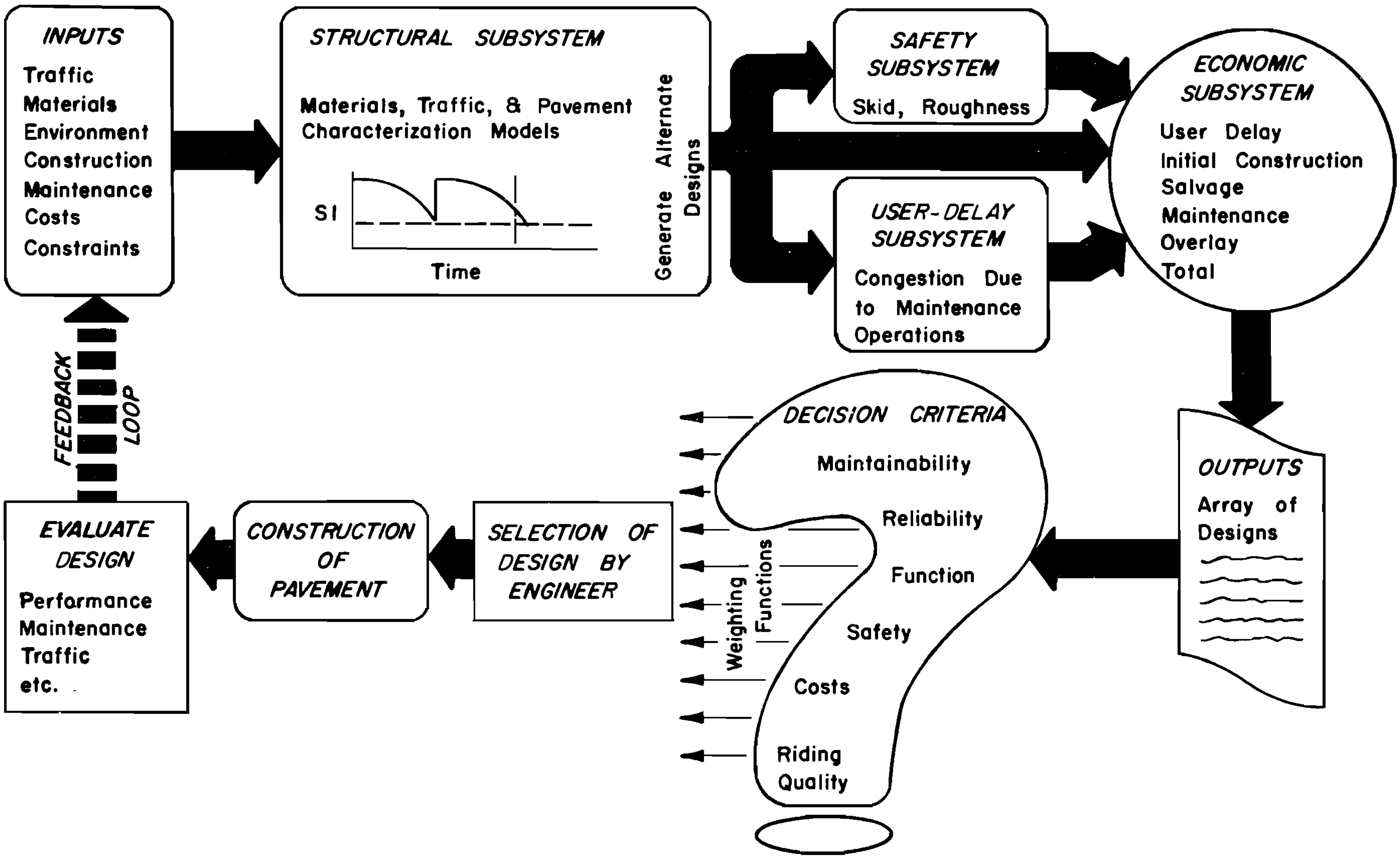


Fig 2.2. Simplified conceptual pavement design system showing basic subsystems and other considerations.

reliability consideration are

(1) inputs,
(2) various design and economic subsystems,
(3) outputs,
(4) decision criteria,
(5) selection of design,
(6) construction, and
(7) feedback loop.

Inputs. The inputs consist of all factors necessary for the various subsystems to function as shown. The inputs also include the necessary constraints to insure practical outputs, such as minimum construction thickness of each layer.

Structural Subsystem. This subsystem generates all possible initial and overlay design strategies that satisfy the performance constraints. It would consist of various mathematical models associated with traffic and pavement behavior and performance which structurally analyze all possible design strategies (material combinations and thicknesses) and predict their performance.

Safety Subsystem. The safety aspects that could be considered in pavement design are skid resistance, roughness, and reflective quality of aggregate.

User-Delay Subsystem. The delay to highway users due to maintenance operations could be determined. This is a very important aspect for high-volume urban highways and also low-volume highways.

Economic Subsystem. The various costs for each pavement design strategy, such as initial construction, maintenance, salvage, user-delay due to overlays, and seal-coats, can be determined. These can be discounted to a present worth using an appropriate interest rate so that the various design strategies can be compared on an equal basis.

Outputs. The outputs include information on possible design strategies which meet the constraints of the system and design conditions. These designs can be arrayed in one of several possible orders depending upon the decision criteria.

Decision Criteria. This function is essential to the systems approach to evaluate the relative goodness of the various alternate designs. Using decision criteria along with appropriate weighting factors, the engineer can compare all alternative designs on the basis of such factors as costs, reliability, safety, function, and maintainability.

Selection of Design and Construction. The engineer can now select the optimum design among the many alternatives and proceed to construction.

Feedback Loop. Measure performance, operation, and maintenance and improve system models as illustrated in Fig 2.1, through sensitivity analysis, research, and implementation.

FPS Working System

The Flexible Pavement System (FPS) is a working flexible highway pavement design system developed (Ref 86) for the Texas Highway Department. FPS is the result of seven years of concerted effort at "extending the AASHO Road Test results in Texas." That study, which terminated in 1968, resulted in the computerized flexible pavement design system designated FPS-1, which has more than 50 inputs, and the output consists of an array of recommended pavement design strategies based on the net present worth of the lowest total cost. The FPS design system has been further developed and subjected to trial implementation through a three-agency research project between the Texas Highway Department, Texas Transportation Institute, and the Center for Highway Research. The improved versions of FPS have been designated as FPS-2, FPS-3, etc.

The basic objective of the FPS is to provide, from available materials, the most economical pavement structure that will provide an adequate serviceability level through the analysis period at a minimum overall cost. The present version of FPS is certainly far from the ideal system. The version used for initial trial implementation was FPS-7. A diagram of the FPS-7 working system is shown in Fig 2.3. There are several empirical models in FPS-7 subsystems which were derived from limited data and therefore have limited application. The basic components of an ideal pavement system are included but some are on a very elementary basis. Details of the FPS-7 system are contained in Refs 8, 40, 54, 81, 82, and 86.

Trial implementation in five Texas Highway Department Districts in 1970 showed several significant deficiencies in the FPS-7 system. One of the most important, by consensus, was that the resulting pavement designs were not adequate considering both initial thickness and predicted life to overlay. This conclusion, however, varied in magnitude with location and highway type. The

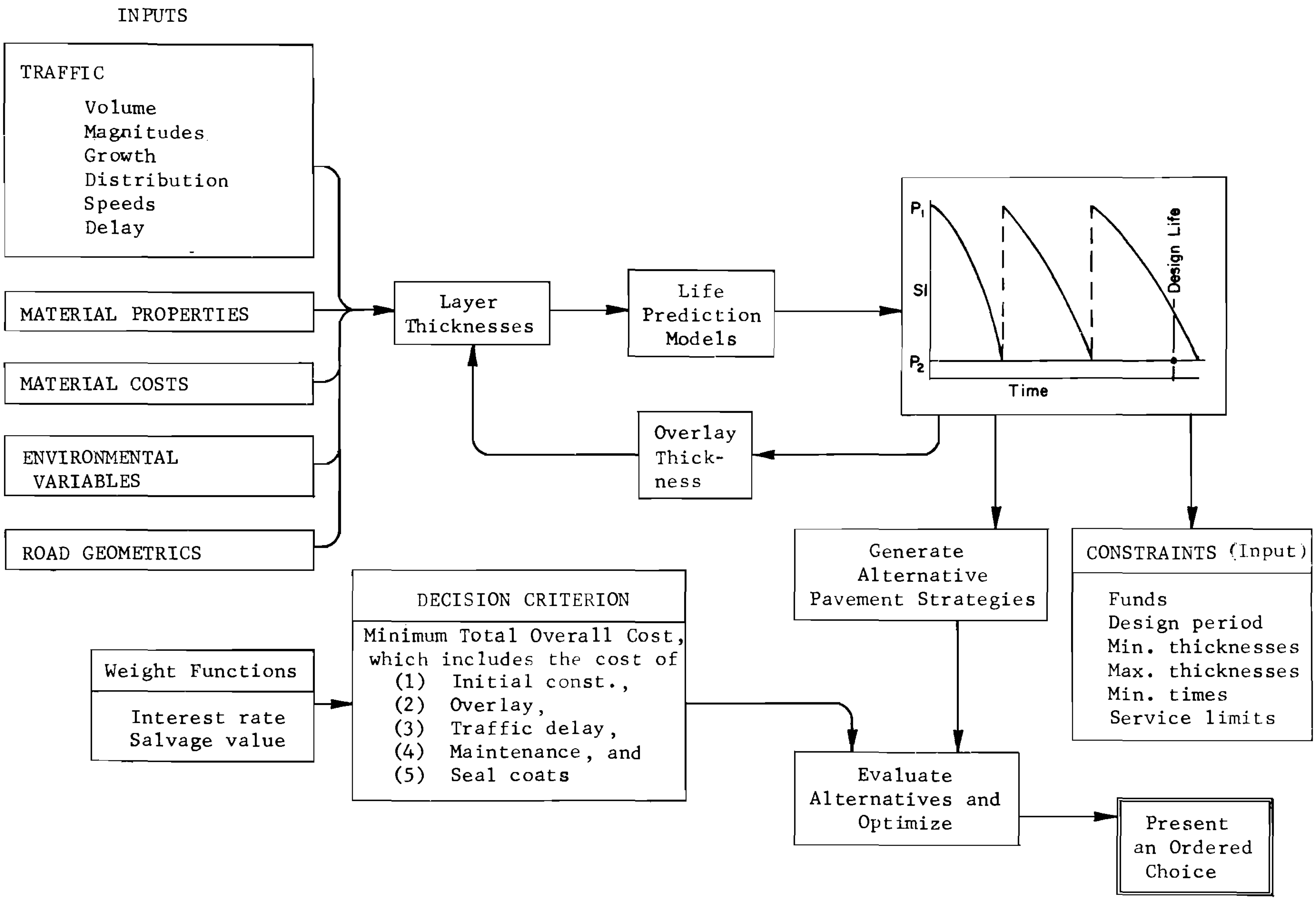


Fig 2.3. Structure of working pavement design system (Ref 44).

FPS-7 system uses essentially "best-fit" regression models derived from empirical data with essentially no factor of safety. It was fully realized that some factor of safety should be applied but the magnitude and amount were not easily determinable. It was realized that the various strengths, serviceability, traffic loadings, and other design factors had considerable variations associated with them, and another realization was that the actual design models used to predict pavement behavior and performance had significant lack-of-fit associated with them. Therefore, an attempt was made to revise FPS-7 using probabilistic design concepts so that the uncertainties and variations of the design system might be quantified. This would allow a designer to design at a specified level of reliability.

The FPS-7 system was modified using probabilistic design concepts (which are detailed herein), and also other improved design models such as the swelling foundation model, and a new version numbered FPS-11 is currently undergoing implementation in the Texas Highway Department and is documented herein.

This section is intended to give only a brief overview of the FPS. The FPS consists of various subsystems as follows:

Structural. The structural subsystem consists of a traffic, a deflection (or material-pavement characterization), and a performance model. The total 18-kip equivalent load applications are input to the system along with other parameters. The traffic equation calculates the accumulated 18-kip equivalent load applications at any time during the design life (Ref 37). The deflection model utilizes layer strength coefficients and thicknesses to calculate the deflection at the surface of a given design section (Ref 35). The performance model calculates the loss in serviceability of a given pavement structure using the deflection of the pavement structure, accumulated equivalent 18-kip axle load applications, an environmental parameter, and swelling clay parameter (Ref 36).

The deflection and performance equations are empirical and were derived from actual field data from the AASHO Road Test and from test sections located at Texas A&M University. They both have significant lack-of-fit associated with them. The traffic equation represents a smooth curve of accumulated loads versus time. Actual traffic buildup is usually more erratic; therefore, it also has considerable lack-of-fit.

Safety. This subsystem for pavement design purposes is restricted to the skid resistance of the pavement surface. Ideally, the subsystem would predict the need of a seal-coat at the time the coefficient of friction of the pavement dropped to a minimum critical value. Steitle and McCullough (Ref 95) have developed this type of subsystem for FPS, but it has not yet been implemented.

User-Delay. For pavement design purposes, the user-delay of a highway is considered only in terms of overlay operations. If a pavement design strategy calls for an overlay at some point in time, the congestion caused by the overlay operations could be severe and the corresponding user's cost would be excessive.

This subsystem calculates the cost to users due to delay (time) and vehicle operation on the basis of dollars per square yard. The economic subsystem adds this cost to the various other costs to form the total cost in dollars per square yard. In practice, for rural highways, the user-delay cost is usually very small compared to the other costs. However, when a highway is operating near capacity at time of overlay, and one or more lanes are closed due to overlay operations and an appropriate detour is not available, severe congestion may set in causing an untenable situation. It serves, therefore as a basic warning to the designer that a particular pavement design strategy may give unsatisfactory results during the operation of the highway.

Economic. The economic subsystem calculates the total cost of the project throughout its design life. The costs are converted to present value at a given interest rate input by the engineer. There are six types of economic submodels used in FPS:

(1) initial construction,
(2) overlay construction,
(3) routine maintenance,
(4) users' cost (delay),
(5) salvage value, and
(6) total overall cost.

Decision Criterion. The basic decision criterion used in FPS is the total overall cost of a particular design strategy. All pavement designs are ranked in order of lowest total cost. Any number of designs may be output for the practical constraints that are input by the engineer, which eliminates many non-practical designs. The engineer may then apply his own decision criteria in examining the array of possible alternate designs.

Overlay Design Mode. The rehabilitation of an existing pavement may be accomplished using the overlay design mode. The asphalt concrete overlay subsystem was developed by Brown and Orellana (Ref 9) and is currently a subsystem of FPS-11. The overlay mode uses many of the subsystems of FPS as previously discussed. The overlay subsystem has also been through trial implementation in several Districts and is discussed further in Chapter 6.

Pavement Systems Reliability

The conceptual pavement design system and the FPS working system have been briefly described. The pavement design system was found to consist of several subsystems and other components which attempt to model the real world process of construction, operation, maintenance, and performance of the pavement. It is obvious, however, that there are many uncertainties and variabilities associated with these predictive models. The failure of any of the subsystems to predict their function will result in some sort of failure of the overall system.

The importance of reliability of a highway pavement has increased steadily for the past several years as indicated by the increase in demand for smooth, maintenance free, low cost, and safe pavement surfaces. These demands are closely related to the increases in vehicle capability and improved geometric design standards of today's modern highways.

The ever-increasing need for the highway dollar for new facilities has also contributed to the increased reliability requirements. The demand for better reliability has also occurred in almost every segment of industry and particularly in national defense related engineering facilities (Refs 20 and 62). It also appears that the demand for greater reliability has been brought about by increased problems associated with the consequences of failure or unreliability. W. N. Carey, Jr. in introductory remarks to the FHWA-HRB Workshop on the Structural Design of Asphalt Concrete Pavements stated the following:

> Everyone who is honest knows that we have been designing pavements by black magic for 40 years. This was acceptable when traffic was light and when anything the highway departments did was a step forward and was welcomed with shouts of

> glee from the public. As you know, traffic is no longer light, and the public is at least confused about its highway programs ... In fact, there is a good bit of evidence that the public is negative about highways these days. So, when the Government Accounting Office starts criticizing our pavement designs (which they have and with embarrassing justification), we had better hurry to get some rational answers. When our interstate highways show serious distress in five years (which they have), we need a better defense than, "The contractor did not build it right," or "The soil at that spot was not what it was supposed to be," or "There are more trucks than we guessed there would be." (Ref 13).

The consequences of pavement failure can be vividly seen when a major urban freeway must be closed down prematurely for maintenance reconstruction operations, as recently occurred on Chicago's Day Ryan Expressway (Ref 100).

Definition of Reliability. A definition of pavement reliability has been proposed by Lemer and Moavenzadeh (Refs 59 and 60). They state that "reliability is the probability that serviceability will be maintained at adequate levels, from a user's point of view, throughout the design life of the facility." They define serviceability as a measure of the degree to which the pavement provides satisfactory service to the user. Reliability might be defined many ways, such as in terms of the probability that the maximum tensile strength is not exceeded by the applied stress in the surface course. A more complete definition would be as follows:

> Reliability is the probability that the pavement system will perform its intended function over its design life (or time) and under the conditions (or environment) encountered during operation.

The four basic elements involved in this concept of pavement system reliability seem to be probability, performance, time, and environment.

Probability: Reliability is the probability of success that a system has in performing its function. There are significant variations and uncertainties in prediction associated with all the models in any pavement design system, and, therefore, the chance of success will always be less than 100 percent. This is discussed in detail in Chapter 3.

Peformance: The degree to which a pavement performs its intended function is its reliability. Performance (in this broad context only) can be defined in several ways in the pavement system with regard to serviceability, skid resistance, user-delay due to maintenance operations, and costs. As used in this study, however, performance refers to the serviceability history of a pavement.

Time: This element is essential in the definition of reliability because the reliability of a pavement must consider its intended life.

Environment: The environmental conditions include the operating circumstances under which the pavement is used. The environment that a pavement "sees" will greatly affect its life span, its performance, and consequently its reliability. Thus, if a pavement's environment changes significantly from that for which it was originally designed, it may not perform with the same reliability.

Reasons for Unreliability. There are many specific reasons for unreliability. Lloyd and Lipow (Ref 62) suggest that the basic cause is due to "the dynamic complexity of system development concurrent with a background of urgency and budget restrictions." This may be applied to pavement design in that designers are working at the limits of technological knowledge. There simply is not enough time or money to examine, synthesize, and analyze each consideration and the almost limitless variability of materials, environments, and traffic makes it impossible to work out all the problems with even one type of material. The complexity of pavement systems has also increased as loads increase in weight and speed and more layers are added to the pavement structure. The more complex the pavement system, the greater the chance for failure.

Subsystem Reliability. There are many facets to the concept of pavement system reliability because there are many functions which a pavement must perform satisfactorily. The various functions or subsystems on which pavement systems reliability depends are shown in Fig 2.2. Each of the functions has several possible failure modes which could affect the overall pavement reliability. These various modes also interact with each other, tending to further complicate the reliability analysis.

The structural distress modes have been defined by McCullough (Ref 64) as fracture, distortion, and disintegration, as shown in Fig 2.4. Manifestations of these types of distress are also shown along with possible causes for each distress manifestation. It may be noted that there are several distress manifestations which can seriously reduce the serviceability of the pavement, thus leading to the necessity of repair maintenance (overlay) to

Distress Mode	Distress Manifestation	Examples of Distress Mechanism (1)
Fracture	Cracking	Excessive loading Repeated loading (i.e., fatigue) Thermal changes Moisture changes Slippage (horizontal forces) Shrinkage
	Spalling	Excessive loading Repeated loading (i.e., fatigue) Thermal changes Moisture changes
Distortion	Permanent deformation	Excessive loading Time-dependent deformation (e.g., creep) Densification (i.e., compaction) Consolidation Swelling
	Faulting	Excessive loading Densification (i.e., compaction) Consolidation Swelling
Disintegration	Stripping	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic
	Raveling and scaling	Adhesion (i.e., loss of bond) Chemical reactivity Abrasion by traffic Degradation of aggregate Durability of binder

(1) Not intended to be a complete listing of all possible distress mechanisms.

Fig 2.4. Categories of pavement distress (Ref 64).

restore serviceability. Serviceability is defined as it was originally by Carey and Irick (Ref 12) as "the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic." A pavement can drop to minimum acceptable serviceability one or more times through its design analysis period. Through maintenance, such as an overlay, serviceability can be restored. When the effort to restore serviceability becomes too costly, the pavement is considered as "failed" structurally and not successfully performing its function.

The safety subsystem has perhaps two modes of distress. The two modes directly concerned with the pavement are skid resistance and excessive roughness. It should be noted that vertical and horizontal geometric designs are not considered in this analysis. Excessive roughness can be caused by dangerous potholes and extensive sudden distortion of the pavement surface caused by expansive clay subgrades. It is a well-documented fact that most pavement surfaces lose skid resistance with traffic applications due to polish and rounding of the aggregates on the pavement surface (Ref 95). The rate of polish depends upon many factors, but especially on traffic volume and aggregate type. The rate of polish for a specific surfacing and traffic can be predicted and overlay or seal-coats programmed to avoid critical minimum skid resistance. Loss of the skid resistance can lead to repair maintenance in terms of a seal-coat or overlay.

The costs of the pavement system are estimated by the economic subsystem based upon predictions of the other subsystems. For example, a structural distress related to thermal cracking which caused a loss in serviceability would affect maintenance costs. Due to loss in structural integrity, the maintenance costs could become so high in attempting to maintain an adequate serviceability level that the pavement was essentially failed and would have to be reconstructed.

Reliability, Performance, and Costs. The reliability requirements of a pavement system are essentially determined by its users, the traveling public. There are known to be several serious consequences from a pavement's showing early or premature distress and also high user costs due to delay and damage to vehicles due to rough pavements. The engineer therefore tries to avoid this situation and applies safety factors so that the pavement will have more than a 50 percent chance of success to survive the required performance period. As concisely stated by Finn with regard to flexible pavement design:

> It is the role of research to improve, quantify, and control the reliability factor in order to provide the most economic balance between performance requirements and costs. (Ref 21).

In pavement design, the cost of attaining an incremental increase in reliability must be balanced against the costs associated with not attaining it. Since failure in terms of pavement performance (with exception of skid resistance loss) does not carry with it the same great problem of loss of life that failure of a bridge does, the optimum reliability may be lower than in typical bridge or building design.

A general relationship between reliability R , performance P , and costs C can be developed conceptually at this point, and a quantitative relationship is developed in Chapter 8. Considering the past definitions of reliability, performance (relating only to serviceability), time, and environment, the general relationship between reliability and costs would be that shown in Fig 2.5a.

There are two basic costs involved as shown in Fig 2.5a. The motorist costs are due to such factors as delay, accidents, and vehicle maintenance and operation caused by rough pavements. The facility costs include such considerations as initial construction, maintenance, and salvage. The motorist costs have not been adequately quantified yet and therefore are not considered in this study. They are important and eventually should be considered. The motorist costs would be high at low levels of reliability. Due to public demand, pavements are normally designed and maintained at relatively higher levels of reliability where the motorist costs due to rough pavements may be relatively small in comparison to the facility costs. Therefore, the motorist costs are neglected in this study because they have not been adequately determined yet and also they may be somewhat less than facility costs in the higher levels of reliability.

As R increases, the C (facility costs) also increases, at an increasing rate as 100 percent R is approached. This increase in R could be induced, for example, by the following factors, among others:

(1) use of better quality materials,

(2) less material variation,

(3) greater maintenance input, and

(4) increase in pavement layer thickness (in general).

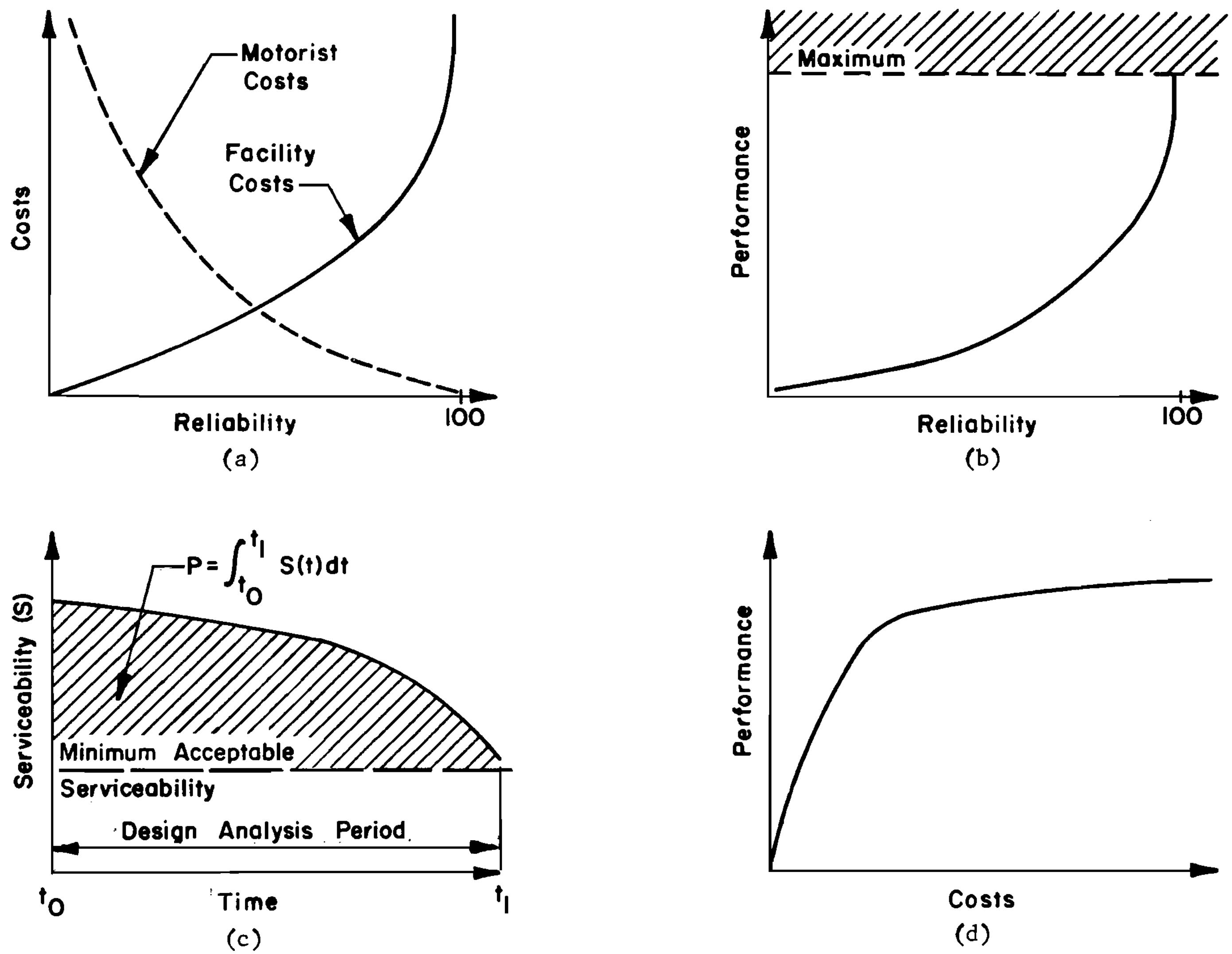


Fig 2.5. Illustration of relationships between reliability (R), performance (P), and costs (C).

Each of these factors causes an increase in the total facility costs and in general supports the conceptual relationship proposed in Fig 2.5a.

The relationship between performance and reliability may be that shown in Fig 2.5b. Performance will increase as reliability increases where P is the integral of the serviceability-time plot shown in Fig 2.5c. Therefore, using the R versus C (facility costs) and R versus P relationships, the relationship between P and C may be established as is shown in Fig 2.5d. The basic reason that facility costs are believed to increase at a faster rate than performance increases is that serviceability has an upper bound and cannot increase indefinitely. This conceptual relationship is quantified for the FPS system in Chapter 8.

This study attempts to develop concepts and theory to achieve a first order solution to "improve, quantify, and control the reliability factor in order to provide the most economic balance between performance requirements and costs" using as a basis the Texas Highway Department flexible pavement design system (FPS).

CHAPTER 3. PROBABILISTIC DESIGN APPROACH

The theory and concepts necessary to apply probabilistic (or stochastic) design methods to pavement design are presented in this chapter. A review of past research is given first, followed by the development of probabilistic theory for pavements, and finally an analysis of reliability for more than one performance period is given.

Review of Previous Work

There have been relatively few efforts in the past toward applying probabilistic design concepts to pavement design. Most of the efforts have been within the last five years. There has been, however, a considerable amount of research work concerning structural reliability analysis in the area of structural design. The basic concept of structural reliability analysis was first outlined by Freudenthal in 1947 (Ref 30). Since then there have been many efforts to apply reliability analyses or probabilistic concepts to structural design (Refs 7, 23 to 29, 31). Most of these efforts deal heavily with the safety aspects of the design, which must be considered differently in pavement design since failure of a pavement is more likely to be a wearout phenomena than a castrophic failure such as a bridge might experience. The advantages and limitations of the introduction of probabilistic concepts in design codes were discussed in a series of four articles published by the American Concrete Institute (Refs 6, 14, 87, 88). An excellent review of literature on structural safety has recently been published by an ASCE Task Committee on Structural Safety (Ref 97). A recent textbook by Haugen, entitled Probabilistic Approaches to Design (Ref 38), outlines the basic approach to structural reliability.

A brief review of work in the pavement area is now given. A conceptual framework for pavement design decisions which involved statistical decision theory was proposed by Hutchinson (Ref 48). He stated the need for such a method as follows:

> The characteristic common to virtually all pavement design decisions is the uncertainty under which they are made. Uncertainty enters into pavement design decisions whenever the outcome of a particular pavement design cannot be exactly predicted. In order to simplify pavement design decisions, it has been expedient, both in actual design decisions and in analytical and empirical models of pavement behavior, to act as if the consequences of various actions could be predicted with certainty.
>
> To overcome this uncertainty, existing pavement design procedures have relied heavily on engineering experience and judgement, much of which is of a subjective nature. This subjectivity has resulted in pavement design procedures whose underlying structure is not always clear, and which cannot easily be modified in the light of new data. Thus, an undesirable situation exists with all standard design procedures where solutions to specific design problems are based more on the forms that have been found as solutions to old problems, rather than on the particular nature of the design problem at hand.

Hutchinson further states that the purpose of the pavement design system is to select a pavement design strategy from among alternate designs whose expected present worth is a minimum with respect to the alternate designs and whose expected life is equal to the design life (Ref 48). He cites references to show that the age at failure of highway pavements cannot be exactly predicted. Since the time to failure is a random variable, the present worth is also a random variable and hence Hutchinson uses statistical decision theory to select the optimum design. The decision process may be expressed in terms of four basic parameters (Ref 48):

(1) A number of alternate courses of action are open to the decision maker,

(2) A number of states of nature that may obtain after a particular course of action has been selected,

(3) The probability measures defined over the states of nature, and

(4) The desirabilities of each of the outcomes that result from combinations of specific courses of action and particular states of nature.

The probability analyses which Hutchinson used to achieve these results are complex and would need considerable development before they could be applied to actual pavement design.

A procedure for evaluating pavements with nonuniform paving materials was developed by Levey and Barenberg (Ref 61). The procedure consists of defining the layered system by a physical model consisting of mass points tied together by springs and bars. The material variability is simulated by assigning different characteristics of the material properties to springs connecting the mass points. The material properties values are selected on a random basis with variations and means corresponding to those found in various layers of typical pavements. Reference 61 says:

> Results from the study show that the response of the layered system is influenced by the statistical characteristics of the materials. Preliminary results strongly indicate a need for the type of analysis presented in the paper as a guide for establishing realistic quality control criteria for paving materials. With results from a procedure it is possible to establish a cost benefit from higher quality control criteria.

An analysis of highway pavement systems and reliability of highway pavement was made by Lemer and Moavenzadeh (Refs 59 and 60). They concluded that the highway pavement system could be evaluated in terms of serviceability, reliability, and matainability. Serviceability was defined as "a measure of the degree to which the pavement provides satisfactory service to the user ... Reliability is a measure of the probability that serviceability will be at an adequate level throughout the design service life ... Maintainability is a measure of the degree to which effort may be required during service life to keep serviceability at a satisfactory level" (Ref 59).

Lemer and Moavenzadeh further state the need for consideration of reliability in design as follows:

> Reliability is important in the pavement system because of the uncertainty involved in all aspects of the pavement process: planning, design, construction, operation and maintenance. Uncertainty arises from lack of information and inability to predict the future. It is embodied in the assumptions that must be made to derive analytical models, the limited amount of data available from tests, and the variable quality of the real-world environment. (Ref 59).

Lemer and Moavenzadeh (Ref 59) state that reliability will generally be a time-dependent parameter (such as failure of material through fatigue). Stochastic models of a facility's behavior, which are generally time-dependent probabilistic representations of physical processes, may be used.

The formulation of a pavement systems concept by Hudson and McCullough in NCHRP Project 1-10 and 1-10/1 (Ref 41) included the development of a general format for illustrating the stochastic nature of distress in a pavement structure. Illustrations of the determination of probability of failure when applied stress and strength are probabilistic are given.

A significant emphasis concerning the application of stochastic concepts to pavement design occurred during the FHWA-HRB workshop on the structural design of asphalt concrete pavement systems held in 1970. The application of stochastic concepts to pavement design was listed as one of the ten most pressing problems facing pavement engineers, as was summarized in Chapter 1. The following briefly summarizes the various committee conclusions about this subject.

(1) The Committee on Solutions to Boundary Value Problems recommended use of stochastic techniques to show potential for future developments of new design methods (Ref 80):

> A stochastic technique would include such probabilistic entities in the appropriate boundary value problem. The solution that is developed will provide stress, strain, and displacement as statistical entities. That is, each numerical value of pavement response would carry a probability of occurrence. Thus, a theory of pavement behavior would assign a level of confidence (or probability) to a particular predicted occurrence.

(2) The Committee on Load and Environmental Variables recommended that an important area of needed research was the establishment of the error involved in the present system of estimating equivalent wheel loads (Ref 93).

(3) The Traffic-Induced Fracture Committee recommended that the variability of every input to the subsystem, along with the uncertainty inherent at the design stage, be considered in design to allow the designer to consider their effects on his decisions (Ref 99).

(4) The Committee on Relating Distress to Pavement Performance suggested that to develop a more rational relationship between distress variables and serviceability performance, it may be necessary to consider stochastic concepts (Ref 45).

(5) Westmann, Chairman of the HRB Committee on Mechanics of Earth Masses and Layered Systems, concluded that stochastic analyses are one pavement design problem needing immediate attention: "Until the present time, surprisingly little has been done to determine the statistical response of a layered system due to a statistical distribution of loadings or for a nondeterministic set of material properties. This aspect of the analysis must be considered if the prediction algorithm is to fit into the overall design system" (Ref 103).

Several papers presented at the workshop discussed the stochastic or variable nature of the pavement design. Pister in the keynote address stated the need to examine the modeling problems of design and management of pavement systems in the light of uncertainty involved (Ref 78).

McCullough stresses the need for the use of stochastic concepts in analyses of distress mechanisms (Ref 64) and gives examples of their use for stress and fatigue considerations. In discussing solutions and solution techniques for boundary value problems, Nair concludes that due to the variability of materials and of loads, complete analyses of these problems will require the inclusion of stochastic and probabilistic considerations (Ref 72). He poses the interesting and important question: "Does it matter if materials are characterized as linear or nonlinear in the context of the variability in materials due to construction techniques?" Damage and distress in highway pavements are considered by Moavenzadeh (Ref 70) to be a stochastic phenomenon. Monte Carlo simulation techniques are suggested for use along with appropriate linear elastic and viscoelastic layered analysis techniques.

In situ materials variability was investigated by Sherman (Ref 91) with many significant implications. Variabilities associated with many phases of design including soil profile (estimation of subgrade strengths), deflection of subgrade base and surface layers under load, compaction of pavement/subgrade layers, thickness, and materials properties (gradation, percent aspahlt, penetration, voids, etc.) were presented.

A brief review of the major investigations of stochastic or probabilistic design concepts applied to pavement design has been made. It is obvious that much has been said for the <u>need</u> of such consideration, but <u>very few practical results</u> have been achieved to date in solving the problem.

Development of Theory

The application of several conceptual methods such as statistical decision theory and Markov processes to pavement design was reviewed. These methods appear to have certain advantages, but they are inadequate (in their current state of development at least) for application to the overall problem of providing a practical and implementable method of considering stochastic variations in the pavement design system. Therefore, a method has been developed, based upon classical structural reliability concepts but modified appropriately for the pavement system criteria and environment.

The use of probabilistic (or stochastic) design concepts provides for the usage of reliability as a major design criterion. The general definition of reliability as related to pavement systems was given as:

> Reliability is the probability that the pavement system will perform its intended function over its design life and under the conditions (or environment) encountered during operation.

The phrase "perform its intended function" could relate to serviceability, skid resistance, user-delay, and costs, but in this study it is used to refer only to serviceability and costs. The skid resistance consideration cannot be made at this time as it is not contained within the current FPS. User-delay is currently considered on a deterministic basis and this is believed adequate until better models are developed. It is also desirable, in the overall derivation of pavement reliability theory, to define consecutive 0.2-mile lengths along a project as pavement sections. A pavement section is considered as a homogeneous unit of design or length of pavement within a regular design project. This 0.2-mile length was chosen because it represents a reasonable length for maintenance and serviceability determination and is also appropriate for use in the probabilistic design approach.

For a single performance period, "perform its function" is defined as an expected percentage (or greater) of pavement sections within a design project showing an adequate serviceability within a limited maintenance cost. There may be one or several performance periods within a design analysis period, as shown in Fig 3.1. The expected percentage of pavement sections that will retain adequate serviceability within maintenance cost limits is the reliability of design.

As a pavement is subjected to traffic and various other loadings, it loses serviceability, as is shown in Fig 3.2, and maintenance costs increase. If the pavement cannot retain adequate serviceability within limited maintenance funds, as shown for one of the pavements in Fig 3.2, it is considered to have not "performed its function." Routine maintenance efforts may help to reduce the rate of loss or restore some serviceability, as illustrated in Fig 3.2, but eventually the pavement section will reach minimum acceptable serviceability and repair maintenance (overlay or reconstruction) will be needed to completely restore serviceability.

There are many factors which cause a loss in serviceability of a pavement such as traffic loadings, swelling subgrade soil, and environmental effects. The relative effect of each of these factors will vary with geographic location. The FPS program considers only the effects of traffic loadings and swelling foundation soil. The swelling subgrade phenomenon is currently

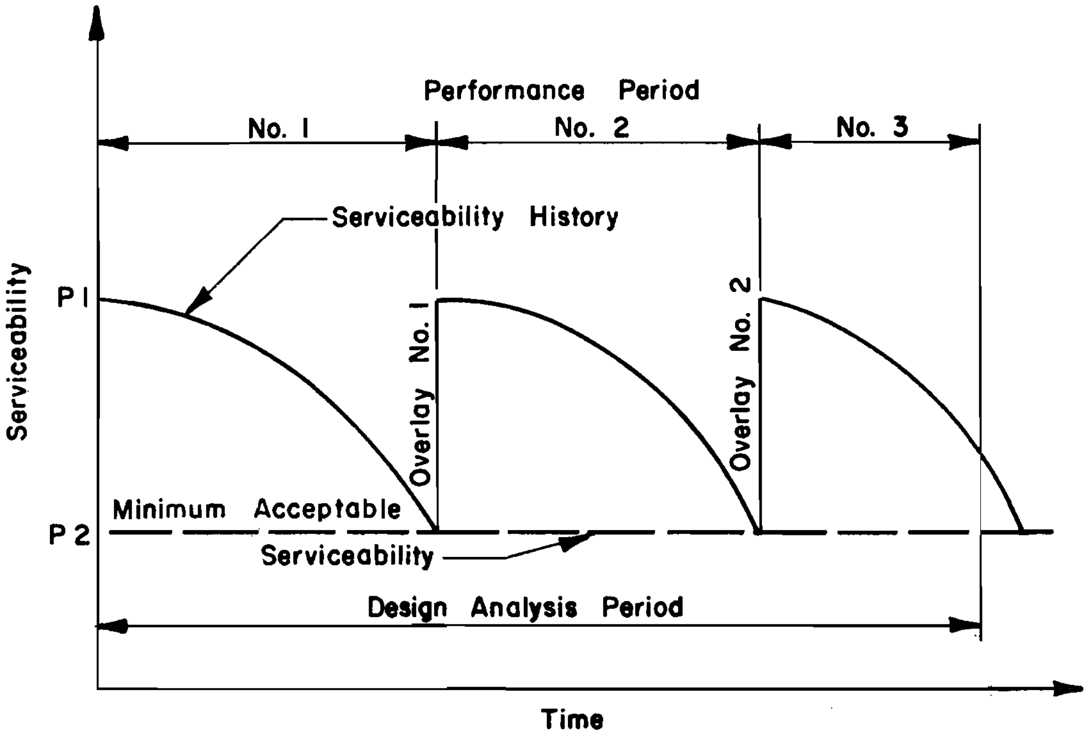


Fig 3.1. Illustration of three performance periods within the design analysis period for a pavement.

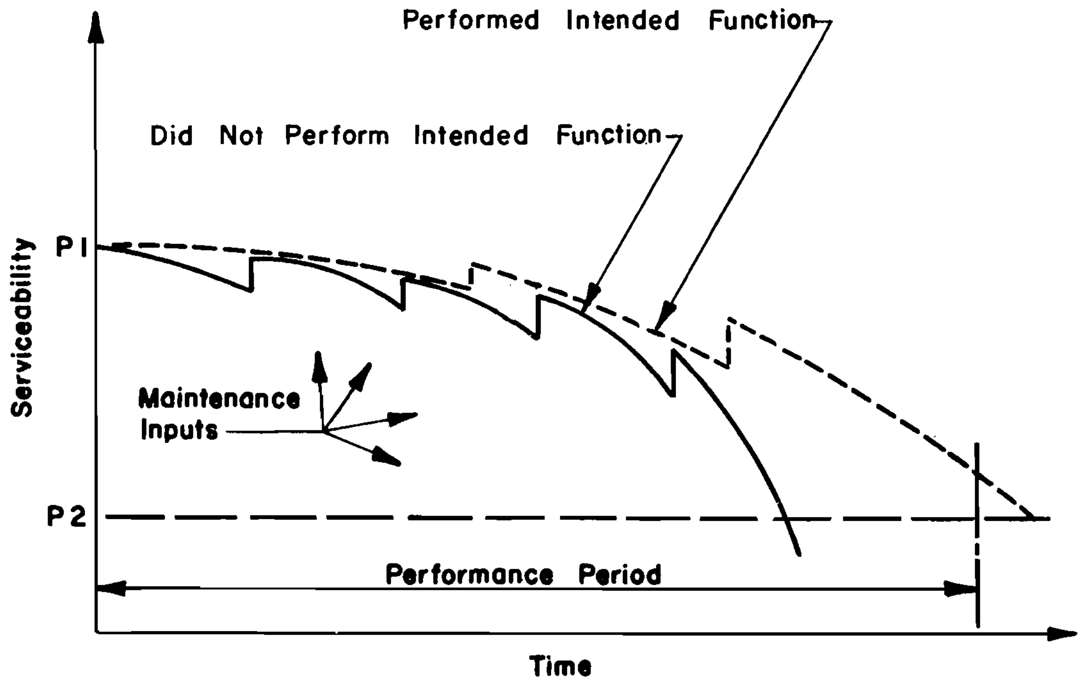


Fig 3.2(a). Illustration of serviceability histories for pavement which did and for pavement which did not perform its intended function.

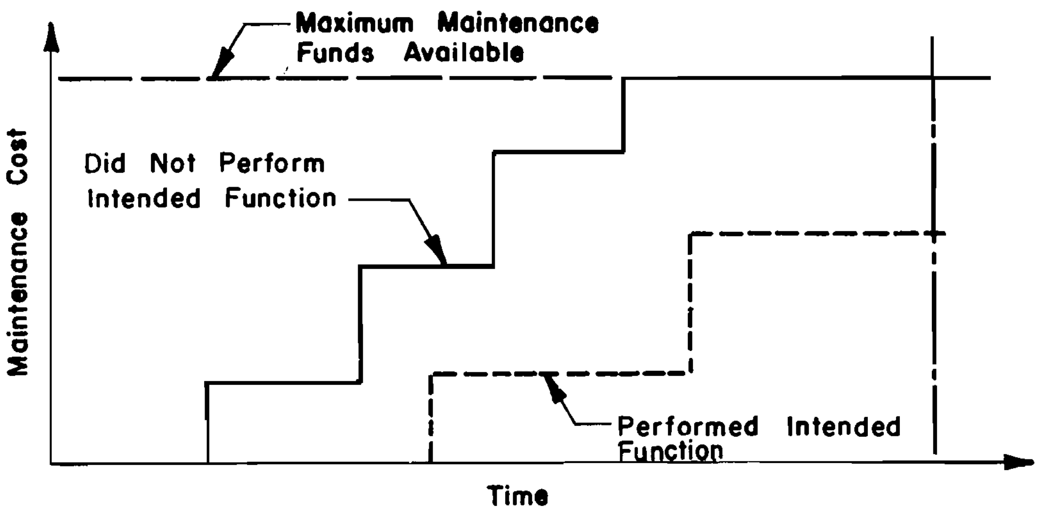


Fig 3.2(b). Illustration of corresponding maintenance costs for the two pavements shown above.

considered in FPS as a function of swelling rate constant, probable length of swelling along a project, and the potential vertical rise of the pavement (Ref 22). This loss of serviceability is considered in FPS to be independent of the loss due to traffic loadings.

Finn (Ref 21) concluded that the most prevalent type of pavement cracking in the United States was traffic-associated cracking. This conclusion was based upon several pavement surveys in the country and is believed to be true for Texas also. The reliability analysis in this study is based upon traffic load associated distress.

The two basic parameters associated with predicting the life of a pavement section (considering only traffic-induced failure) are the number of load applications the pavement can take (or allowable applications) and the number of loadings that may be applied to the pavement. Both of these parameters are stochastic variables as the factors on which they depend are variable or stochastic. The reliability of a pavement section is determined from the basic concept that a no-failure probability exists when the number of load repetitions to minimum acceptable serviceability (N) is not exceeded by the number of load applications applied (n):

N = number of load applications that a section of pavement can withstand before minimum allowable serviceability is reached within a limited maintenance input,

n = number of load applications which are applied to a pavement section.

The number of load applications refers to 18,000-pound equivalent single-axle applications.

Reliabilty (R) is defined mathematically as the probability that N will exceed n , as shown by the following expression:

$$R = P[N > n] \tag{3.1}$$

where

P[] = probability that the event shown in the brackets will occur.

This statement is analogous to the statement that reliability is the probability that strength is greater than stress. Both N and n are stochastic variables and have a probability distribution associated with them, as illustrated in Fig 3.3.

The probability of n having a value of n_1 is equal to the area of the element of width dn , or to A_1 , as shown in Fig 3.3. The f(n) and f(N) are defined as the density functions of n and N , respectively:

$$P\left(n_1 - \frac{dn}{2} \leq n \leq n_1 + \frac{dn}{2}\right) = f(n_1)dn = A_1$$

Since f(n) and f(N) are density functions, the probability that $N > n_1$ equals the shaded area under the f(N) density curve A_2 :

$$P(N > n_1) = \int_{n_1}^{\infty} f(N)dN = A_2$$

The reliability R (i.e., the probability of no failure at n_1) is the product of these two probabilities.

$$P\left(n_1 - \frac{dn}{2} \leq n \leq n_1 + \frac{dn}{2}\right) \cdot P(N > n_1)$$

and

$$dR = f(n_1)dn \cdot \int_{n_1}^{\infty} f(N)dN$$

Reliability of the pavement structure is the probability that N will be greater than the possible values (over the range) of n . Thus, the basic equation

$$R = \int dR = \int_{-\infty}^{\infty} f(n) \left[\int_{n}^{\infty} f(N)dN\right]dn \qquad (3.2)$$

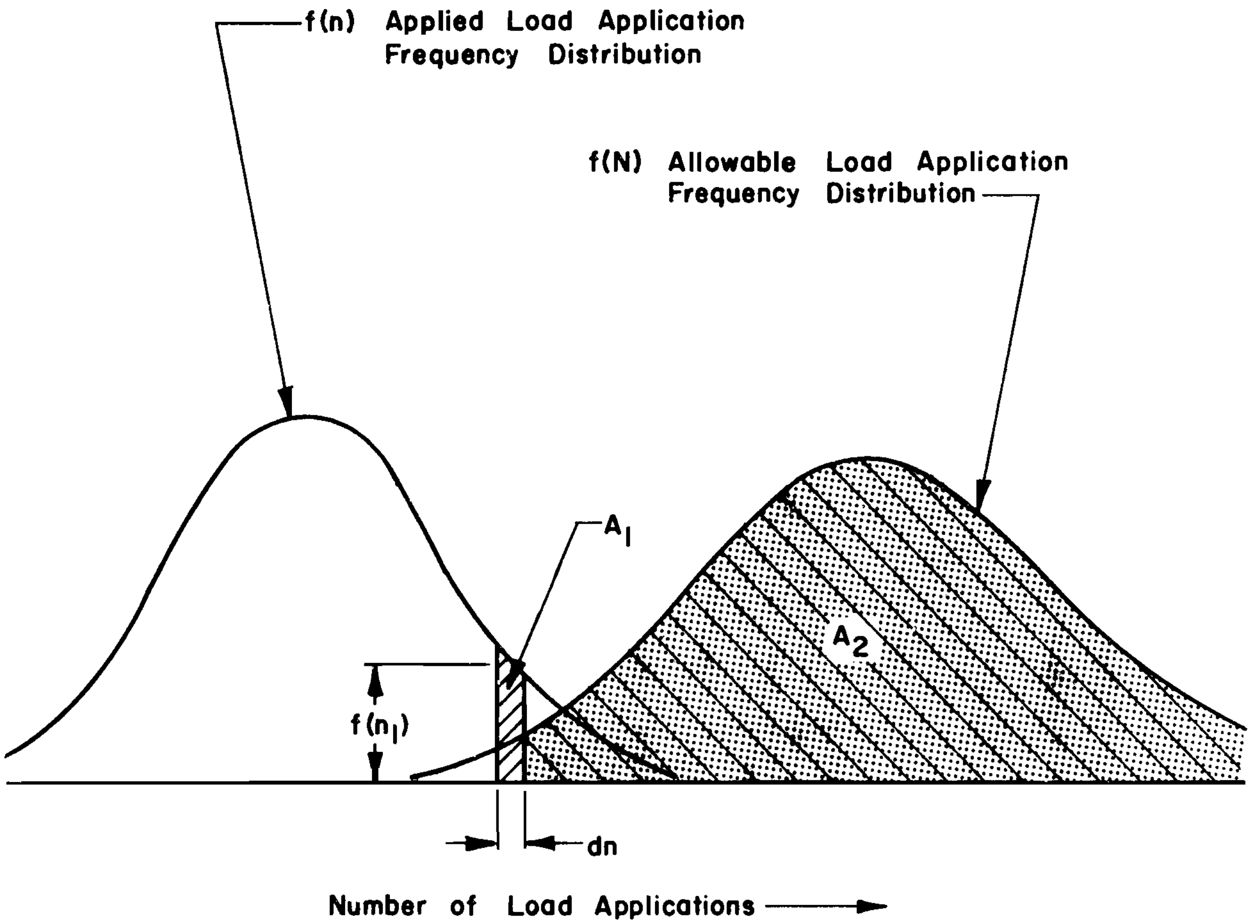


Fig 3.3. Illustration of allowable load application (N) distribution and applied load application (n) distribution.

where

$$\int_{-\infty}^{\infty} f(N)dN = 1 ,$$

$$\int_{-\infty}^{\infty} f(n)dn = 1$$

Alternatively, an expression for reliability R may be obtained by considering that a no-failure probability exists when applied load n remains less than some given value of N .

Equation 3.2 may be solved for exact answers if the distributions of N and n are normal or can be transformed to be normally distributed. The distributions of n and N are believed to be approximately log normal, based upon the following results:

(1) The N to terminal serviceability is similar to the number of applications to failure in a fatigue test. The fatigue life of asphalt concrete specimens under various loading conditions has been found to be approximately log normally distributed by Pell and Taylor (Ref 75) and by Moore and Kennedy (Ref 71). The error in predicting the N to failure of the AASHO Road Test pavement sections was found to be approximately log normally distributed, as shown in Chapter 4.

(2) The n depends upon design ADT, percent trucks, axle factor, and axle load distributions, as is shown in Chapter 4. There are not adequate data available to verify that any of these factors is normally distributed. Since each of these depend upon several other factors, the error in prediction of each of these parameters would tend to approach normal by the central limit theorem (Ref 19).

(3) Simulation of log N and log n using Monte Carlo techniques was used to give further data concerning the distribution of N and n. Values of the design parameters shown in Eq 4.4 given in Chapter 4 were selected from normal distributions using typical means and standard deviations and log N was calculated each time by Eq 4.4 for 1000 trials. The same technique was used to obtain 1000 values for log n by Eq 5.3 given in Chapter 5. The χ^2 goodness of fit test, skewness, and kurtosis were used to test the hypothesis of normality. The results are summarized in Table 3.1. The assumption of normality could not be rejected at a level of significance of 0.05 for log N and log n . Using the same methods the design parameters were sampled from uniform distributions and the log N and log n were calculated. Results from 1000 simulated samples

are shown in Table 3.1 also. The hypothesis of normailty was not rejected for log N , but was for log n at the 0.05 level of significance. Plots for log N simulated from normal and from uniform distributions are shown in Fig 3.4 and can be visually compared. There does not appear to be much difference between them.

Reliability can now be evaluated considering log N and log n to be normally distributed (Note: All logarithms are to base ten.):

$$R = P[(\log N - \log n) > 0] = P[D > 0] \tag{3.3}$$

where

$$D = \log N - \log n$$

Therefore $f(D)$ is the <u>difference density function</u> of log N and log n. Since log N and log n are both normally distributed, D will also be normally distributed. Function D is shown in Fig 3.5. Using bars above the expressions to represent their mean values, the following equation can be written:

$$\overline{D} = \overline{\log N} - \overline{\log n} \tag{3.4}$$

The standard deviation of D may be computed as s_D by the following equation:

$$s_D = \sqrt{s_{\log N}^{\;2} + s_{\log n}^{\;2}} \tag{3.5}$$

where

$s_{\log N}$ = standard deviation of log N ,

$s_{\log n}$ = standard deviation of log n

As shown in Fig 3.5, reliability is given by the area to the right of 0.

$$R = P[D > 0] = \int_0^\infty f(D)\,dD \tag{3.6}$$

TABLE 3.1. SUMMARY OF STATISTICS RELATED TO SIMULATED VALUES OF LOG N AND LOG n

Distribution Sampled	Parameter	Sample Size	Mean	Standard Deviation	Skewness	Kurtosis	χ^2	Intervals for χ^2
Normal	log N	1000	6.342	0.461	-0.24*	+3.28*	30.92	38
Normal	log n	1000	5.179	0.167	-0.02	+2.93	3.68	14
Uniform	log N	1000	6.546	0.426	+0.002	+2.97	53.03	42
Uniform	log n	1000	5.170	0.165	-0.030	2.52*	29.89*	20

*Test statistic significant at level of significance of 0.05.

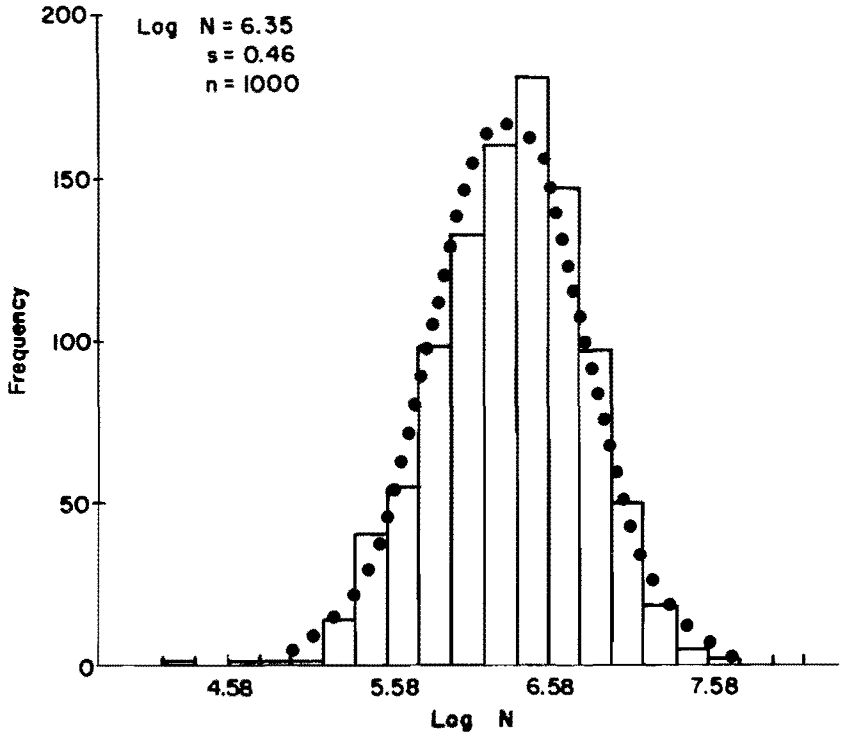


Fig 3.4(a). Histogram of log N from simulation of design factors from <u>normal</u> distributions (Eq 4.4).

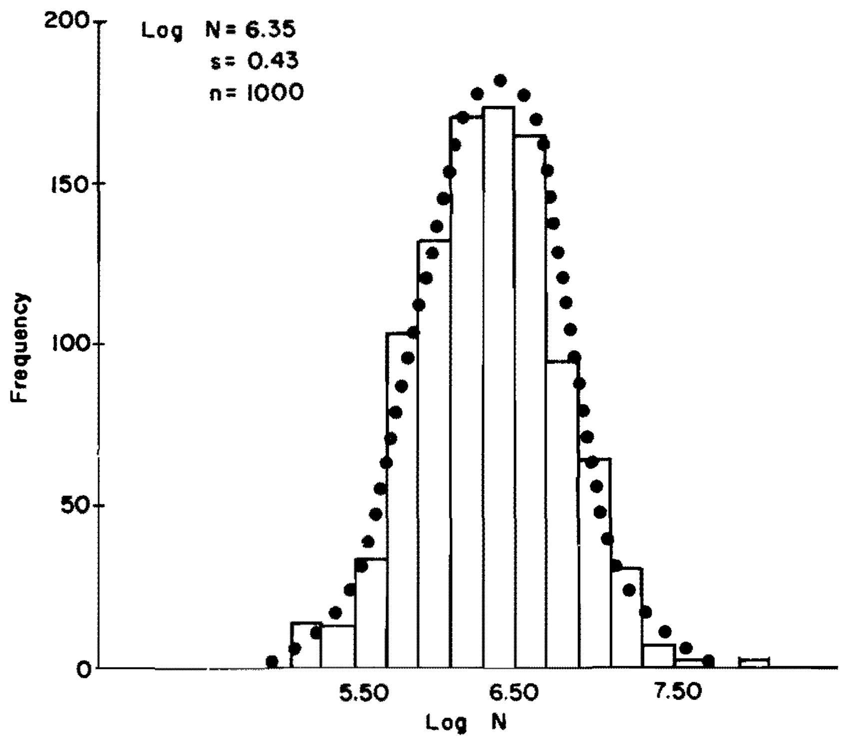


Fig 3.4(b). Histogram of log N from simulation of design factors from <u>uniform</u> distributions (Eq 4.4).

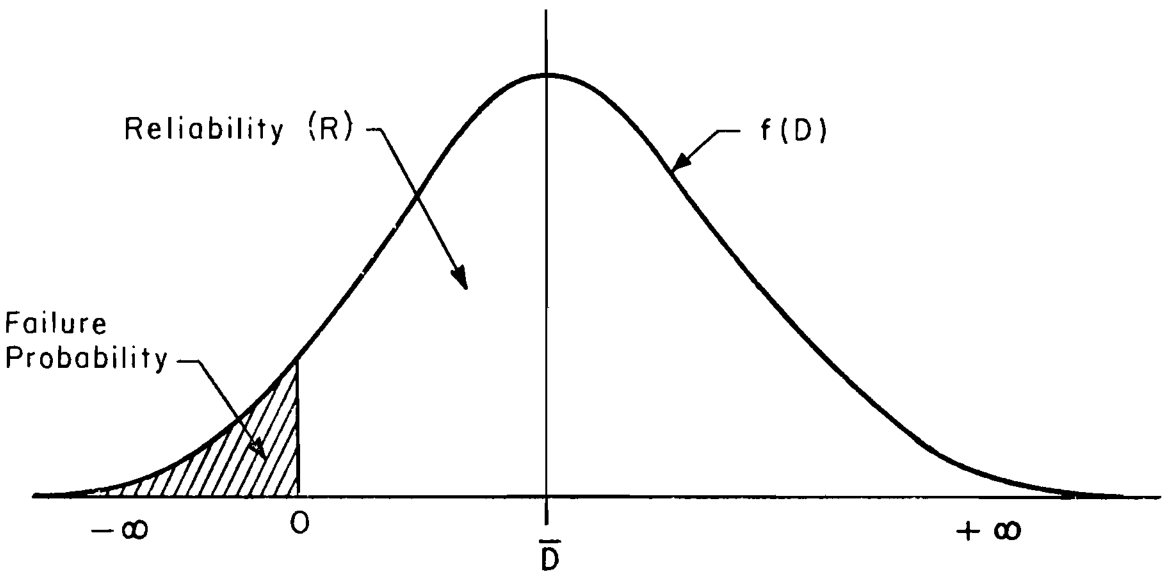


Fig 3.5. Difference density function (D = log N - log n).

or

$$R = P[0 < (\log N - \log n) < \infty] = P[0 < D < \infty] \tag{3.7}$$

The transformation which relates D and the standardized normal variable z is

$$z = \frac{D - \overline{D}}{s_D} \tag{3.8}$$

for

$$D = 0 , \quad z = z_0 = -\frac{\overline{D}}{s_D} = -\frac{\overline{\log N} - \overline{\log n}}{\sqrt{s_{\log N}^2 + s_{\log n}^2}} \tag{3.9}$$

for

$$D = \infty , \quad z = z_\infty = \infty \tag{3.10}$$

The expression for reliability may be rewritten as

$$R = P[z_0 < z < z_\infty] \tag{3.11}$$

The reliability may now be determined very easily by means of the normal distribution table. The area under the normal distribution curve between the limits of $z = z_0$ and $z = \infty$ gives the reliability of a design as given from the following expression:

$$R = \frac{1}{\sqrt{2\pi}} \int_{-\frac{\overline{D}}{s_D}}^{\infty} e^{-\left(\frac{z^2}{2}\right)} dz \tag{3.12}$$

As an example for the calculation of reliability, assume $(\overline{\log N}, s_{\log N}) =$ $(7.100, 0.400)$, and $(\overline{\log n}, s_{\log n}) = (6.500, 0.200)$.

$$z_0 = -\frac{7.100 - 6.500}{\sqrt{(0.4)^2 + (0.2)^2}} = -1.342$$

From normal distribution tables the area from - 1.342 to ∞ is 0.91. Therefore, the reliability R is 91 percent.

Reliability Considerations - Several Performance Periods

The reliability of a pavement design project for one performance period is the expected percentage of 0.2-mile sections along the project maintaining an adequate serviceability without the total maintenance cost exceeding a prescribed limit. When a pavement is designed for more than one performance period (stage construction), some complications arise as to handling the design situation.

A properly constructed overlay restores serviceability of the pavement to near the level that it had just after construction. Observations in Texas show that various pavements exhibit widely differing performance characteristics during the second and succeeding performance periods after they have been overlaid. The time to failure of an overlaid pavement may be longer or shorter than the initial pavement life. To further complicate this problem, the actual decision criterion used for actually placing an overlay is not fully known. The reasons appear to vary widely from location to location throughout the state. The decision to place overlays is probably a function of such factors as available funds, traffic volume, serviceability level, areas of extreme localized failures, distress manifestations such as cracking or spalling, and even anticipation of the future distress.

The basic decision rule used in this study for design is that the overlay will be placed when the expected 1 - R percent sections have reached minimum acceptable serviceability level. The overlay that is placed will be designed to last the next performance period with a probability of R .

The following cases have been formulated to illustrate the boundaries of the problem which have been observed in Texas.

Case I. The pavement/subgrade system may be such that after a pavement section serviceability once falls to the minimum acceptable level, an overlay will only restore the serviceability for a brief time period. A pavement structure containing a cement-treated base that cracks badly is an example of this type. An overlay placed on this pavement will only maintain adequate serviceability for a short period of time. Assume that a pavement design strategy calls for three performance periods. The pavement is designed for $R = 0.90$ chance of success during each period and the overlay is placed when $1 - R$ sections reach minimum acceptable serviceability level. An analysis of the reliability involved is given in Table 3.2 for Case I. An expected 10 percent of the pavement sections reach minimum serviceability during the first period, 19 percent by the end of the second period, and 27.1 percent by the end of the third period. These probabilities are determined according to the assumptions that a section that has reached minimum serviceability cannot be restored to full capacity again, and that those sections that do not reach minimum serviceability have an R chance of success during the next period. The overall percentage of sections expected to succeed at the end of the analysis period is 72.9 percent.

Case II. The pavement/subgrade system is such that when a pavement section falls below the minimum allowable serviceability level and an overlay is placed, the section will have R chance of surviving the next performance period. Those sections that did not reach minimum serviceability during the period but were also overlaid will last throughout the entire remaining time of the design analysis period. A pavement that may have localized areas of failure caused by swelling subgrade or poor construction would be an example of this case. This pavement would normally be completely overlaid, and since many sections would be in good condition before the overlay, they would last throughout the rest of the design analysis period with the overlay. The reliability involved for this situation is shown in Table 3.2, where $R = 0.90$ and there are three performance periods. An expected 10 percent of the sections would reach minimum serviceability during the first period, 1 percent the next, and 0.1 percent the final period. The overall expected percentage of sections to succeed at the end of the design analysis period would be 99.9 percent.

The criteria given for Case I and Case II are believed to be the extreme ends of the spectrum of actual pavement performance. Therefore the following procedure was developed; it provides results that are between those described and is also an implementable procedure in the FPS program. This procedure is also felt to be closer to that actually occurring in the field than Case I or Case II.

TABLE 3.2. SUMMARY OF RELIABILITY CALCULATIONS FOR PAVEMENTS SHOWING CASE I, CASE II, AND THE SELECTED METHOD PERFORMANCE CHARACTERISTICS

Situation	Performance Period	Expected Success (R)	Expected Failure
Case I	First	0.90	1.00 - 0.90 = 0.10
	Second	0.9 x 0.9 = 0.81	1.00 - 0.81 = 0.19
	Third	0.81 x 0.9 = 0.729	1.00 - 0.729 = 0.271
Case II	First	0.90	1.00 - 0.90 = 0.10
	Second	0.90 + 0.10 x 0.9 = 0.99	1.00 - 0.99 = 0.01
	Third	0.99 + 0.01 x 0.9 = 0.999	1.00 - 0.999 = 0.001
Selected Method	First	0.90	1.00 - 0.90 = 0.10
	Second	0.90	1.00 - 0.90 = 0.10
	Third	0.90	1.00 - 0.90 = 0.10

<u>Selected Method</u>. The pavement/subgrade system is such that after an overlay has been placed, pavement sections that reached minimum acceptable serviceability and those that did not, previous to the placing of the overlay, both have R chance of surviving the next performance period. Therefore the pavement would show R percent of the sections succeeding at each performance period. This pavement may have combinations of Case I and Case II characteristics, but overall the average section that is overlaid at the time of 1 - R failures, will show R chance of lasting through the next performance period. An analysis of the corresponding reliabilities is shown in Table 3.2. The analysis is made as before, considering $R = 0.90$ and three performance periods. An expected 10 percent of all sections reach minimum serviceability during the first period, 10 percent by the end of the second period, and 10 percent by the end of the third period. Therefore the overall expected percentage of sections to survive the design analysis period would be 90 percent. A pavement design strategy with none or several overlays would have the same reliability at the end of the design analysis period.

In summary, a pavement is designed by the selected method so that each performance period has an expected R percent of the sections maintaining adequate serviceability and the overlay is placed when the expected 1 - R sections have reached minimum acceptable serviceability. In reality, the number of sections to reach this level will vary from project to project as the expected percentage refers to all projects designed by the method. There is also some chance that the overlay itself will fail through improper construction and/or materials usage and therefore cause additional pavement distress not considered in this analysis.

CHAPTER 4. STOCHASTIC NATURE OF DESIGN PARAMETERS

The basic theory for the application of probabilistic design methods to pavements has been presented. The random or stochastic nature of many of the design parameters was pointed out in the discussion. A random or stochastic variable is a variable which assumes each of its possible values with a definite probability (Ref 10). Studies concerning the variability of highway materials and properties have shown that they do follow common statistical distributions, such as the normal distribution (Ref 91). Recent studies in statistical quality control and in situ measurements of pavement properties have pointed out the large variability which exists in the "as-built" properties of pavement materials (Refs 34, 35, 50, 57, 66, 67, 68, 89, 90, 91, and 102). There are also many uncertainties associated with traffic load forecasting (Ref 11). This chapter attempts to illustrate the types, distributions, and magnitudes of variation of the design parameters and models of the Texas flexible pavement design system. These results are used later in the application of probabilistic design concepts to FPS in Chapters 6 and 7. It should be noted that the variations estimated in this chapter are approximate and based upon limited data. The further quantification of these variations is one of the major recommendations of this study.

There are essentially three basic types of variations associated with flexible pavement design:

(1) variation within a design project length,
(2) variation between design and actual values, and
(3) variation due to lack-of-fit of the design models.

The division of the total variation into these types helps to conceptualize the application of probabilistic concepts to pavement design. These types of variations are now examined in detail and examples are given to help estimate the types, distributions, and magnitudes.

Variation Within a Design Project Length

A design project can be defined as a specific length along a highway that is designed for a uniform pavement thickness and materials type. The design project must also have the same traffic loading throughout its length. A highway construction project usually covers several miles in length and it is not uncommon to find several distinct soil types along the project (Ref 73). In some instances, the pavement design is varied along a project to take into account these subgrade soil variations as it may be economically advantageous to do so. The usual practice in most states is to construct the same pavement section along the entire project, thus increasing the within-project variability.

Within a design project, there also exist many variations associated with the strength of the pavement and subgrade, thickness of the pavement layers, and the smoothness of the finished surface. To study these actual variations that occur within a project design section length, in situ measurements were made on several in-service highways in Texas by means of a Dynaflect* and the Surface Dynamics Profilometer (Ref 101). The Dynaflect was used to estimate the structural variations and the profilometer to measure the roughness variations.

Pavement and Subgrade Stiffness. The Dynaflect applies a dynamic cyclic load to the pavement and measures the response of the pavement. The resulting surface deflection basin can be used to calculate the in situ stiffness coefficients, which vary from about 0.15 for a weak wet clay to 1.00 for asphalt concrete (Ref 84), and are calculated by means of a computer program developed by Scrivner et al (Ref 22), called Stiffness Coefficient. The program is used by the Texas Highway Department as part of FPS. The theory and development of the stiffness coefficient are similar to that of the elastic modulus but with simplifying assumptions, as described by Scrivner and Moore in Ref 81. The pavement/subgrade stiffness is characterized by the "Surface Curvature Index" or SCI, which represents the numerical difference between sensors No. 1 and No. 2 of the Dynaflect. The general layout of the Dynaflect load wheels and sensors is shown in Figs 4.1 and 4.2.

---

* Registered trademark, Dresser-Atlas Company, Dallas, Texas (Ref 85).

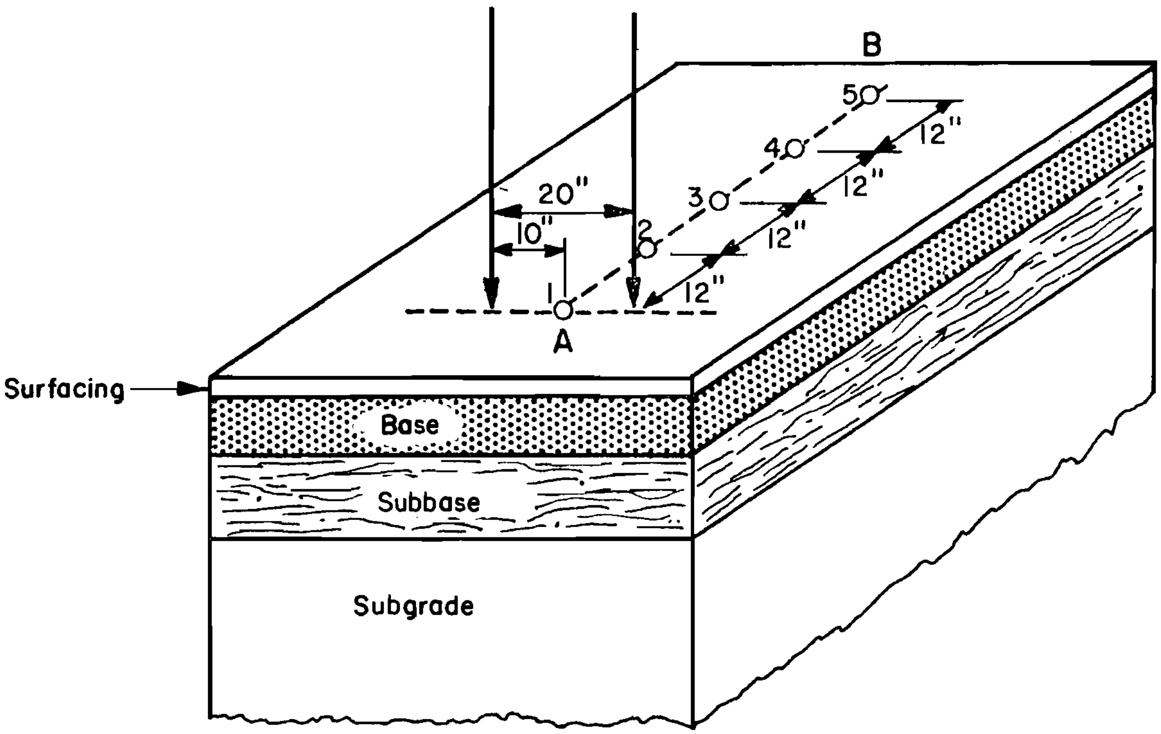


Fig 4.1. Position of Dynaflect sensors during test. Vertical arrows represent load wheels. Points numbered 1 through 5 indicate location of sensors. (Ref 81).

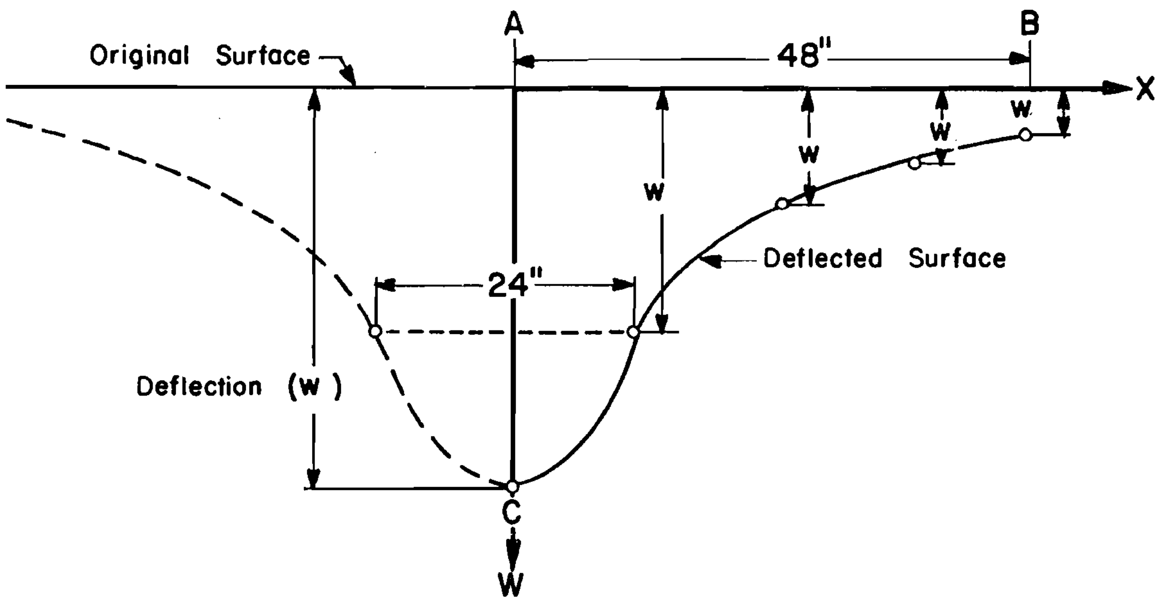


Fig 4.2. Typical deflection basin reconstructed from Dynaflect readings. Only half of basin is measured. (Ref 81).

Many pavements in Texas consist of a relatively thin surfacing (less than 2 inches) and a base course. This makes it possible to measure the in situ stiffness coefficients of pavement materials and subgrades. Dynaflect deflection measurements were made along several pavements containing untreated aggregates and treated (with asphalt, cement, and lime) gravels and soils. These deflections (SCI) were used in the stiffness coefficient program to calculate stiffness of the pavement material and of the subgrade. The resulting data have been analyzed and summarized in various tables and figures. Histograms showing typical distributions of SCI, stiffness of pavement, and subgrade within a project are shown in Figs 4.3 through 4.8. These data were collected from various locations throughout the state of Texas. In each plot, a normal distribution curve has been fitted.

It is important to determine the general type of probability distribution that the SCI and stiffness coefficients follow. A visual analysis can be aided by using three statistical tests to test the hypothesis that the samples come from normally distributed populations. The tests are the χ^2 goodness-of-fit, skewness, and kurtosis. The χ^2 test can be used to judge the assumption of normality. The skewness test can be used to test whether or not the data are significantly skew to one or the other side. Kurtosis shows whether or not the distribution is too peaked or too flat-topped (Ref 94). These tests were applied to some 57 distributions of SCI, pavement stiffness coefficients, and subgrade stiffness coefficients. The level of significance used in the tests was 0.05 in all cases. A brief summary of these data shows the following percentages of projects which showed no significant reason to reject the assumption of normality.

	SCI	Pavement Stiffness	Subgrade Stiffness
Normality not rejected	58%	90%	63%
Skewness not significant	26%	69%	42%
Kurtosis not significant	79%	9%	69%

Results show that SCI tended to have skewed distributions towards the lower side, but normality was not rejected in 58 percent of the projects. The various pavement materials stiffness coefficients tended to be symmetrical and approximate the normal distribution in 90 percent of the projects. The subgrade

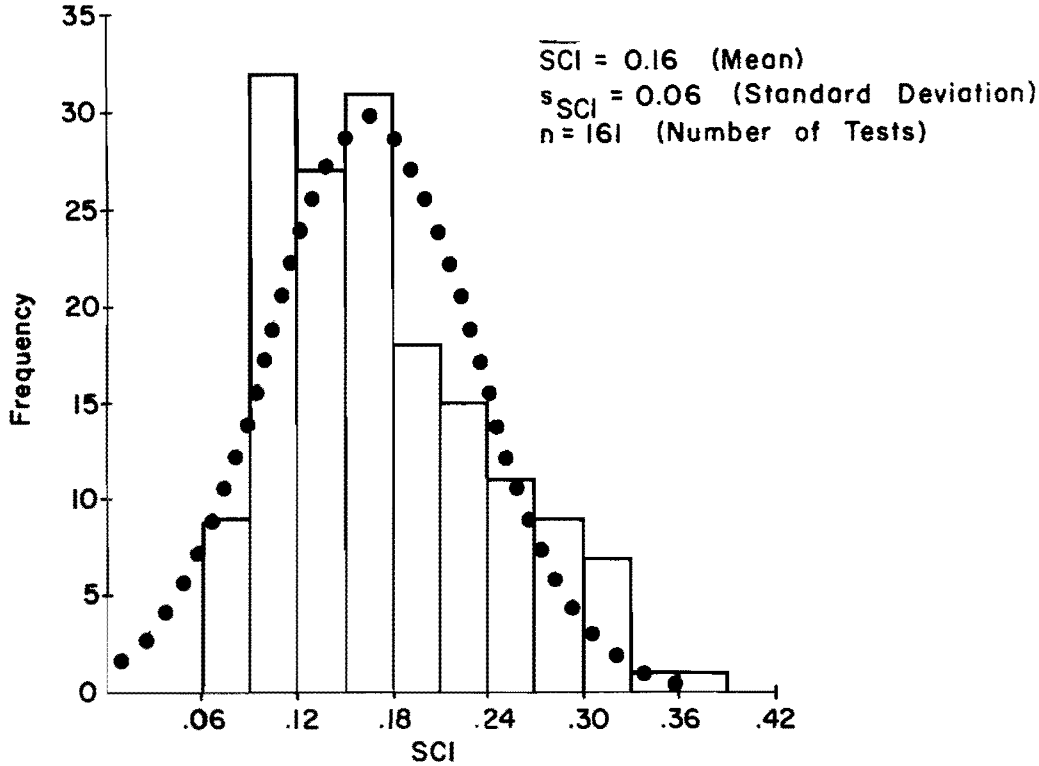


Fig 4.3. Histogram showing typical SCI distribution along an in-service highway in Texas measured by Dynaflect.

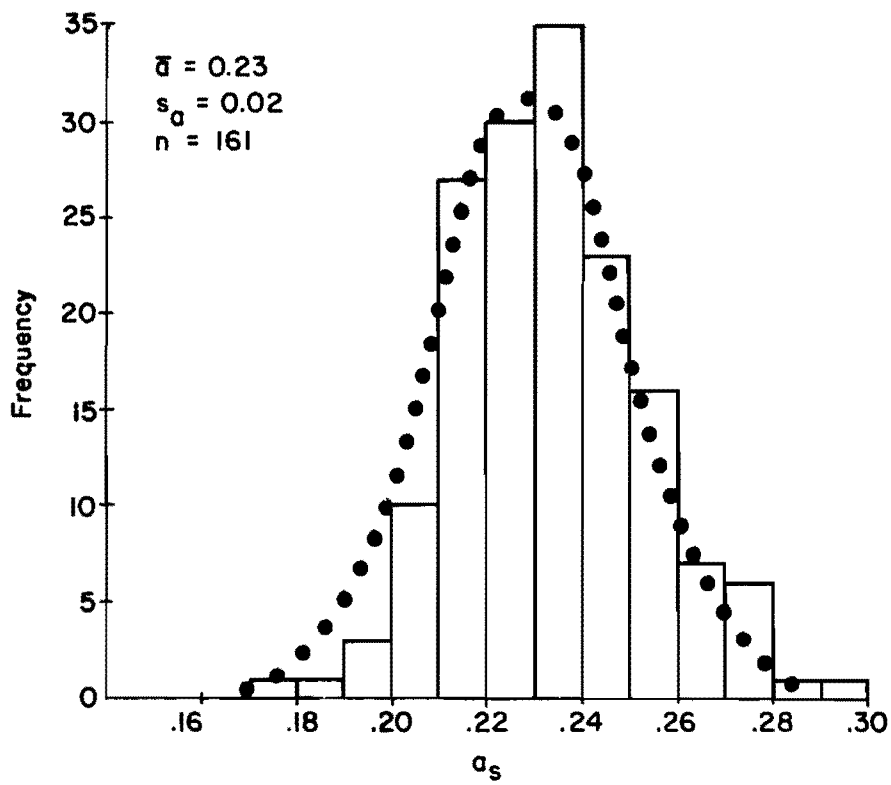


Fig 4.4. Histogram showing typical subgrade stiffness coefficient distribution along an in-service Texas highway.

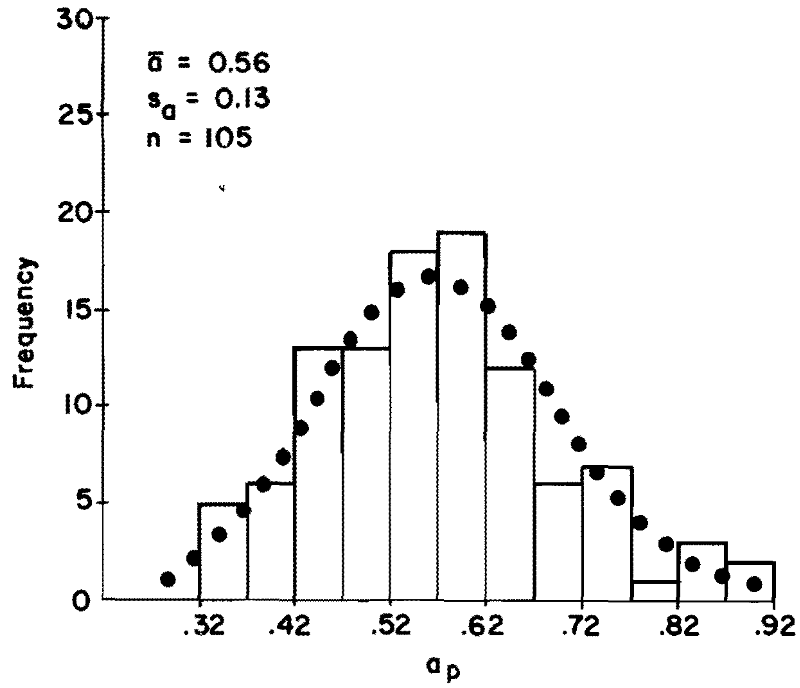


Fig 4.5. Histogram showing typical stiffness coefficients for crushed sandstone base along an in-service highway in Texas.

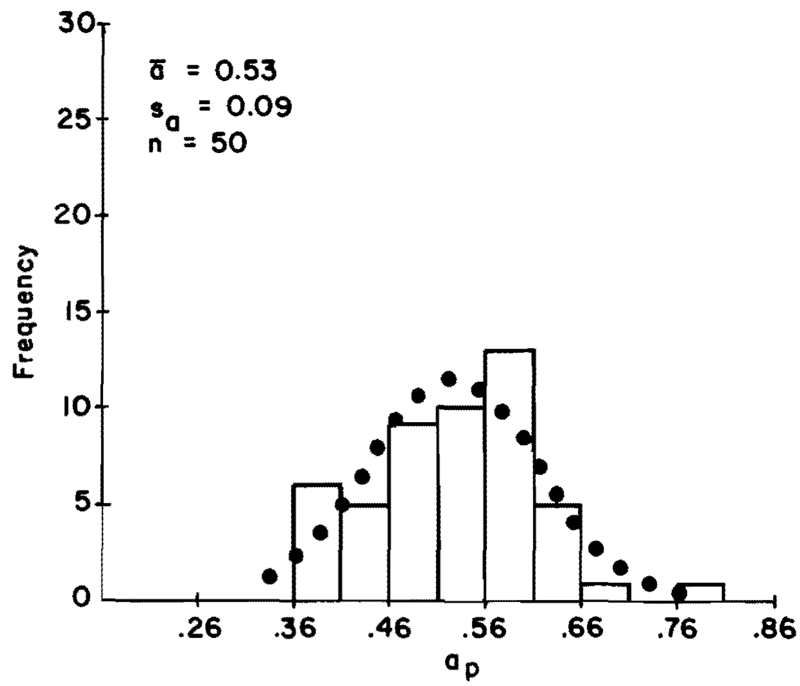


Fig 4.6. Histogram showing typical stiffness coefficients for lime-treated gravel base along an in-service highway in Texas.

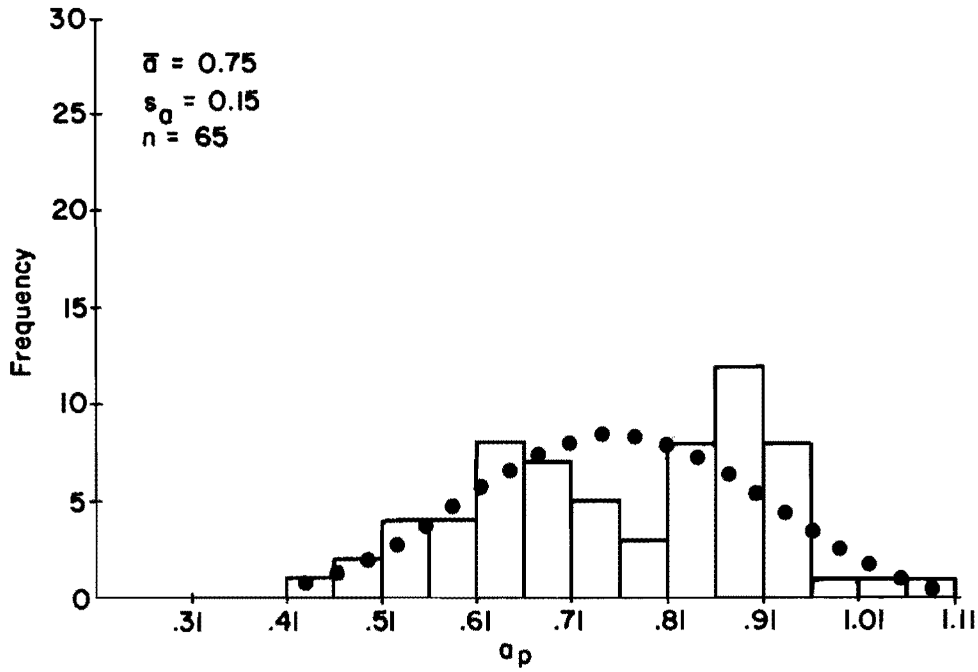


Fig 4.7. Histogram showing typical stiffness coefficient distribution for asphalt-treated base along an in-service highway in Texas.

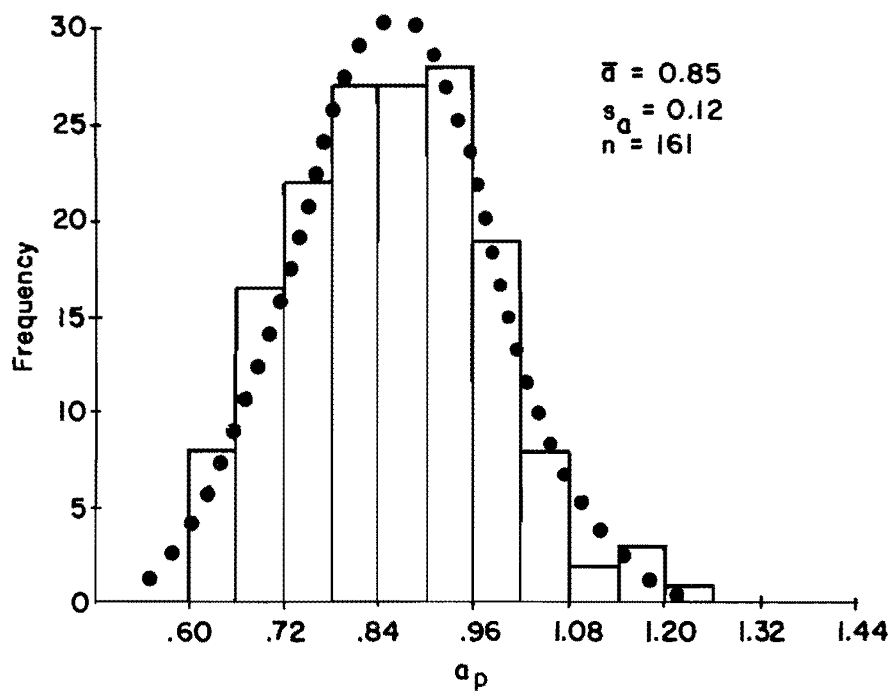
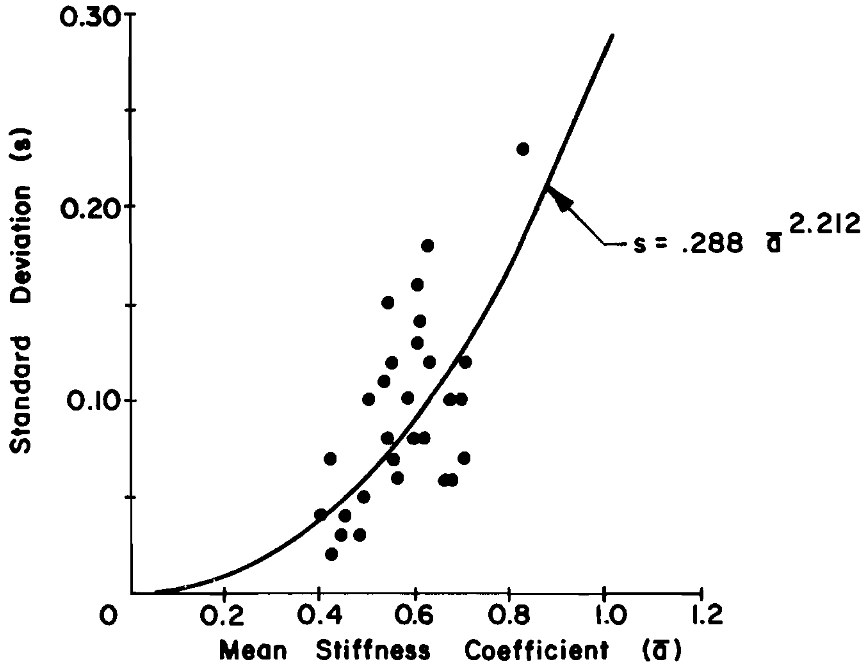


Fig 4.8. Histogram showing typical stiffness coefficient distribution for pavement consisting of asphalt concrete and cement-treated layers along an in-service Texas highway.

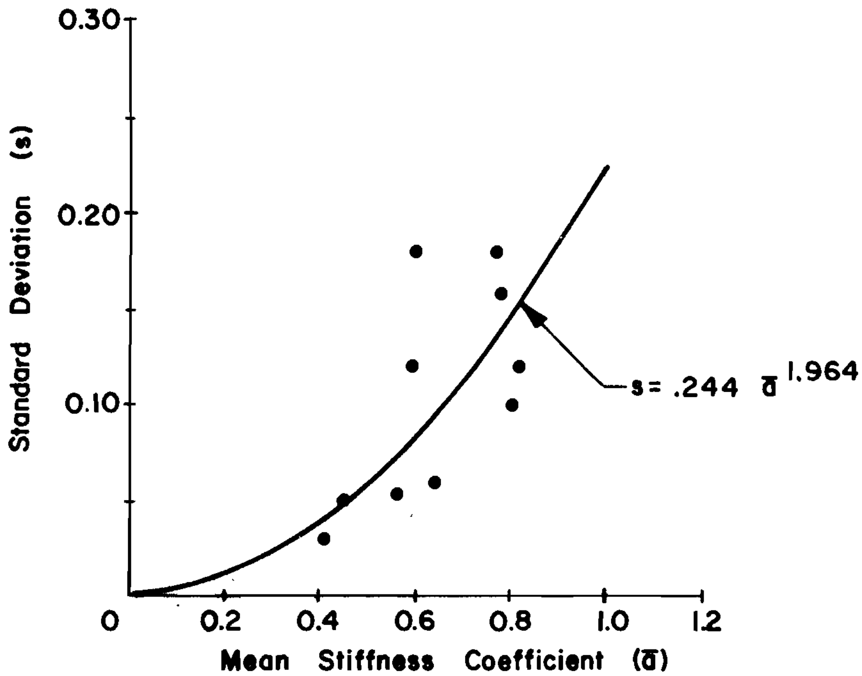
stiffness coefficients tended also to skew towards the lower values, but the assumption of normality could not be rejected in 63 percent of the projects. The basic conclusion which can be made, based upon available data, is that the SCI and the pavement and subgrade coefficients come from approximately normal distributions but have a tendency to skew toward the lower values. These results are for point estimates of SCI and stiffness coefficients measured at about 0.2-mile intervals. The distribution of average stiffness coefficients over some specific length would more closely follow a normal distribution due to the central limit theorem. Also, there are many contributing causes to stiffness coefficient and SCI variation, including such factors as compaction, gradation, moisture, and material characteristics. When the variation of a factor is the sum of variations from several sources, then, no matter what the probability distribution of the separate errors may be, their sum will have distribution that will tend more and more to the normal distribution as the number of sources increase, due to the central limit theorem (Refs 10 and 19).

Prediction of Variations of Stiffness Coefficients. Perhaps even more important than the distributions is the magnitude of variations associated with each material type. To obtain estimates of the magnitude, deflection data were collected by the Texas Highway Department along 181 projects at about 0.2-mile intervals of varying composition and subgrades and the in situ stiffness coefficients were calculated. There were usually at least 10 replications within each project and so a mean and a standard deviation for each project could be calculated. Plots were then made of the mean versus the standard deviation for each available material type. Typical plots for several of these materials are shown in Fig 4.9. There is a general curvilinear relationship between the mean and the standard deviation. The slope of the curve at any point represents the coefficient of variation. Regression equations to predict the standard deviation from the mean were derived using least-square techniques and are given for each material type in Fig 4.9.

The expected standard errors of the stiffness coefficient within a project may be estimated for these materials if the mean stiffness coefficient is known. To estimate standard errors of materials not given in Fig 4.9, the designer should use judgment based upon these results or other data that may be available.



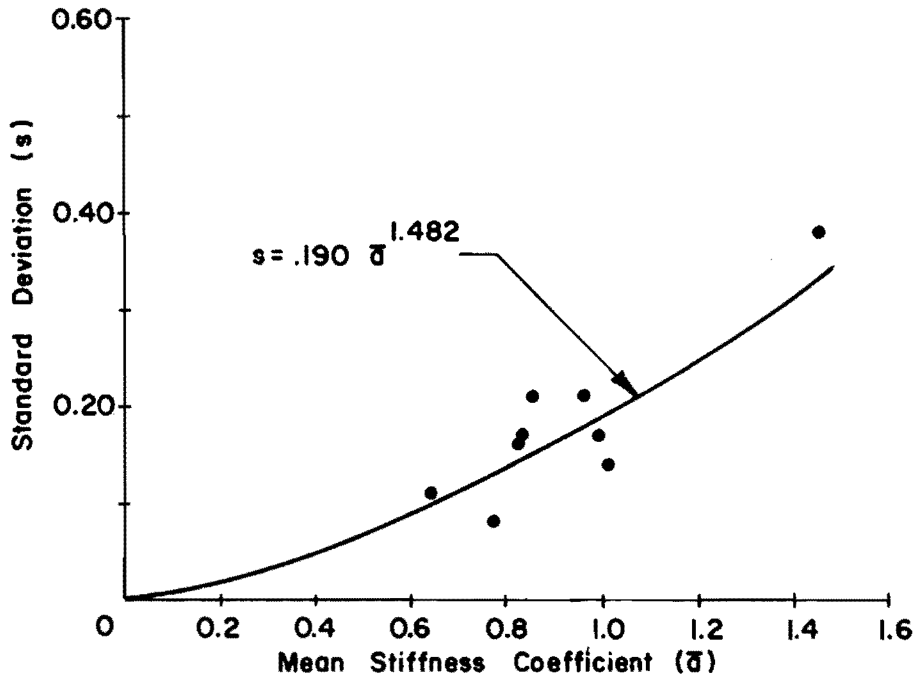
(a) Flexible base.



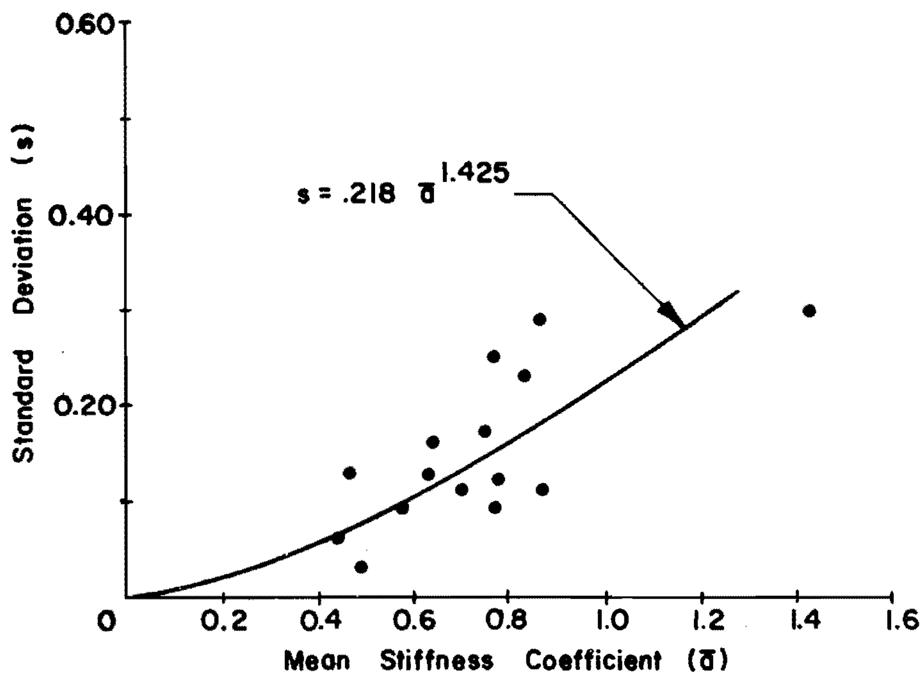
(b) Untreated limestone.

Fig 4.9. Mean versus standard deviation of stiffness coefficient of indicated materials for in-service highway pavements.

(Continued)



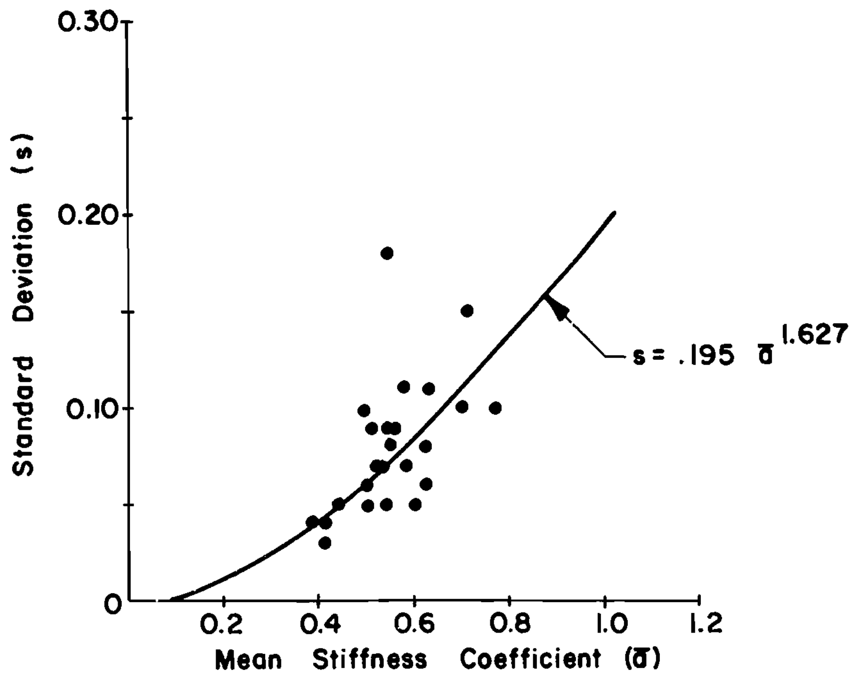
(c) Cement-treated.



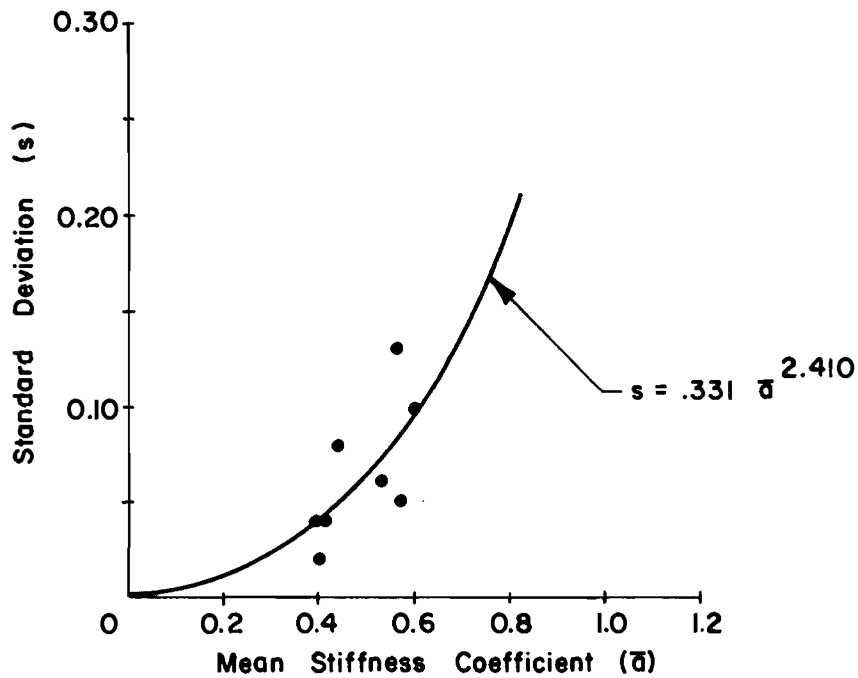
(d) Asphalt-treated.

Fig 4.9. Continued.

(Continued)



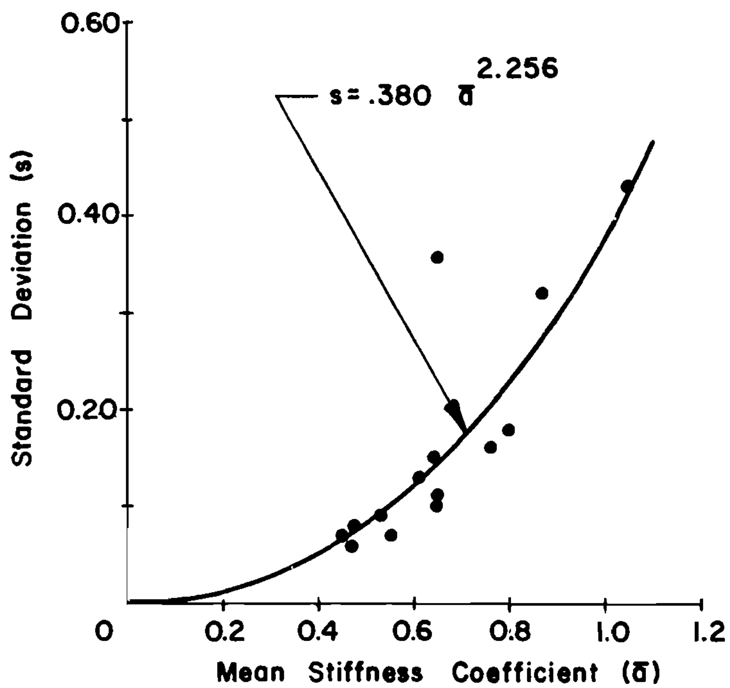
(e) Untreated gravel.



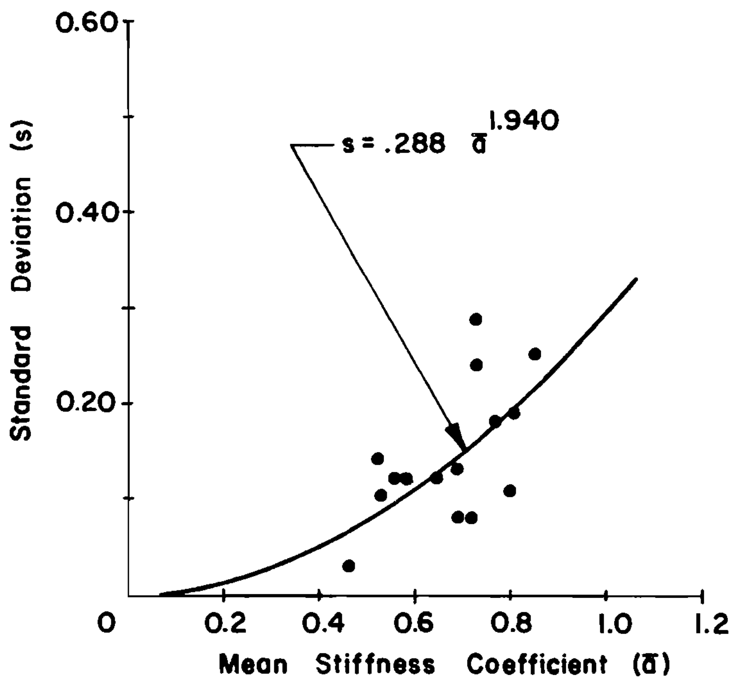
(f) Untreated sandstone.

Fig 4.9. Continued.

(Continued)



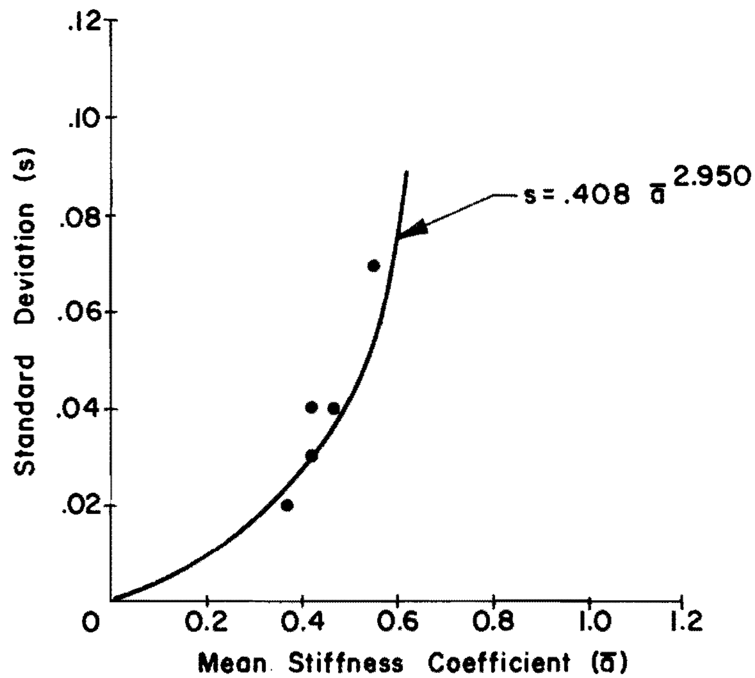
(g) Untreated iron ore-topsoil (IOTS).



(h) Lime-treated.

Fig 4.9. Continued.

(Continued)



(i) Untreated caliche.

Fig 4.9. Continued.

Pavement Layers Thickness. The thickness variations of the individual material layers that make up the pavement structure have been studied in numerous quality-control studies. The variation of layer thickness is important in that variations may have significant influence on performance.

Sherman (Ref 91) presented results from California showing thickness variations in various pavement layers from 1962 through 1969. Some of the conclusions that were reached are as follows:

(1) Paying for materials by the square yard in-place tends to encourage keeping the thickness to a minimum. Paying for materials by the ton in-place has a reverse effect.

(2) The ability of the contractor to accurately construct a layer is dependent upon the accuracy required in placing the layer below. The 1962-1969 data shown in Table 4.1 indicate an increasing degree of accuracy in the layer thicknesses from subbase through base to surface.

(3) The mean deviation from planned thickness was generally on the thicker side.

Approximate standard errors obtained from Table 4.1 by pooling the mean squares show the following estimates for the various pavement materials:

Material	Standard Error	Number of Tests
Asphalt concrete	0.41 inch	9,775
Cement-treated base	0.68 inch	9,749
Aggregate base	0.79 inch	8,053
Aggregate subbase	1.25 inches	10,578

Data obtained from 12 projects in four states show an asphalt concrete surface-course thickness standard error of 0.26 inch, as reported by Granley (Ref 34). Huculak (Ref 46) reports a standard error of 0.4 for an asphalt concrete surfacing. Keyser and Waell (Ref 52) report a standard error of 0.48 inch for 109 projects of asphalt concrete surfacing. They also report a standard error of 0.43 inch for 33 projects measured one year later.

The type of distribution associated with thickness variations would be expected to be approximately normally distributed due to the many contributing sources of error. Results of 1,100 core thicknesses on 109 projects, as reported by Keyser and Waell (Ref 52) are shown in Fig 4.10. This plot illustrates that the assumption of normality is reasonable.

TABLE 4.1. THICKNESS MEASUREMENT VARIATIONS (from California Division of Highways, Ref 91)

Year	Material	Mean Deviation from Planned Thickness (ft)	Standard Deviation	Number of Measurements
1962	Asphalt concrete	+0.02	0.03	823
	Cement-treated base	+0.02	0.06	934
	Aggregate base	+0.00	0.07	1,149
	Aggregate subbase	0.00	0.08	1,037
1963	Asphalt concrete	+0.01	0.03	1,327
	Cement-treated base	+0.02	0.06	1,173
	Aggregate base	0.00	0.06	1,310
	Aggregate subbase	0.00	0.09	1,183
1964-1965	Asphalt concrete	+0.02	0.03	1,760
	Cement-treated base	+0.02	0.05	2,187
	Aggregate base	0.00	0.06	1,285
	Aggregate subbase	+0.02	0.10	1,922
1966	Asphalt concrete	+0.02	0.04	1,569
	Cement-treated base	0.00	0.06	1,569
	Aggregate base	0.00	0.07	1,272
	Aggregate subbase	+0.03	0.12	1,833
1967	Asphalt concrete	+0.01	0.03	1,838
	Cement-treated base	0.00	0.06	1,412
	Aggregate base	+0.01	0.07	1,134
	Aggregate subbase	+0.03	0.11	1,887
1968	Asphalt concrete	+0.02	0.04	1,135
	Cement-treated base	+0.01	0.05	1,156
	Aggregate base	+0.01	0.06	828
	Aggregate subbase	+0.01	0.10	1,526
1969	Asphalt concrete	+0.02	0.04	1,323
	Cement-treated base	+0.01	0.06	1,318
	Aggregate base	+0.02	0.07	1,075
	Aggregate subbase	+0.02	0.11	1,370

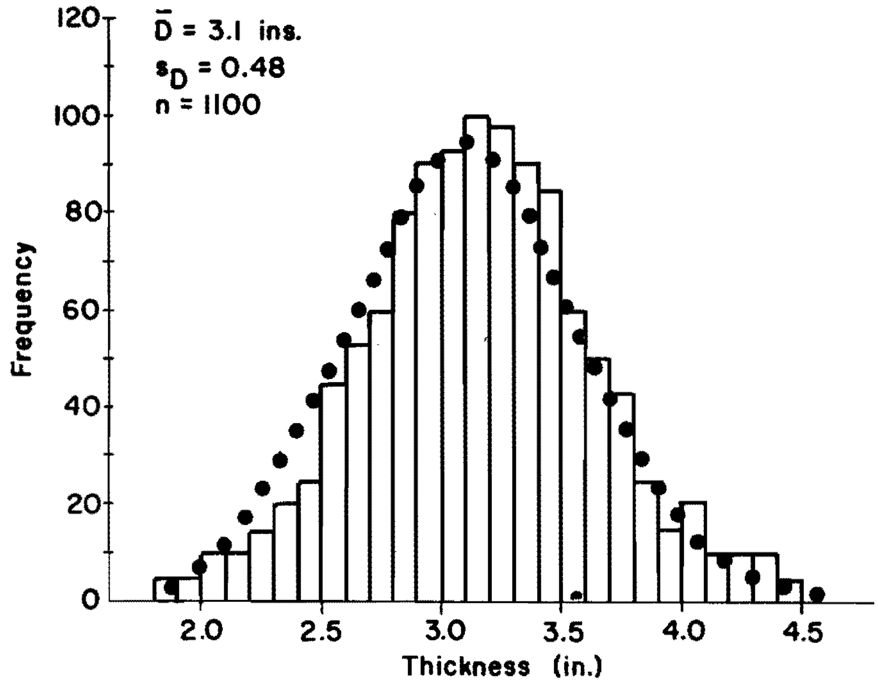


Fig 4.10. Thickness of individual cores for asphalt surfacing (Ref 52).

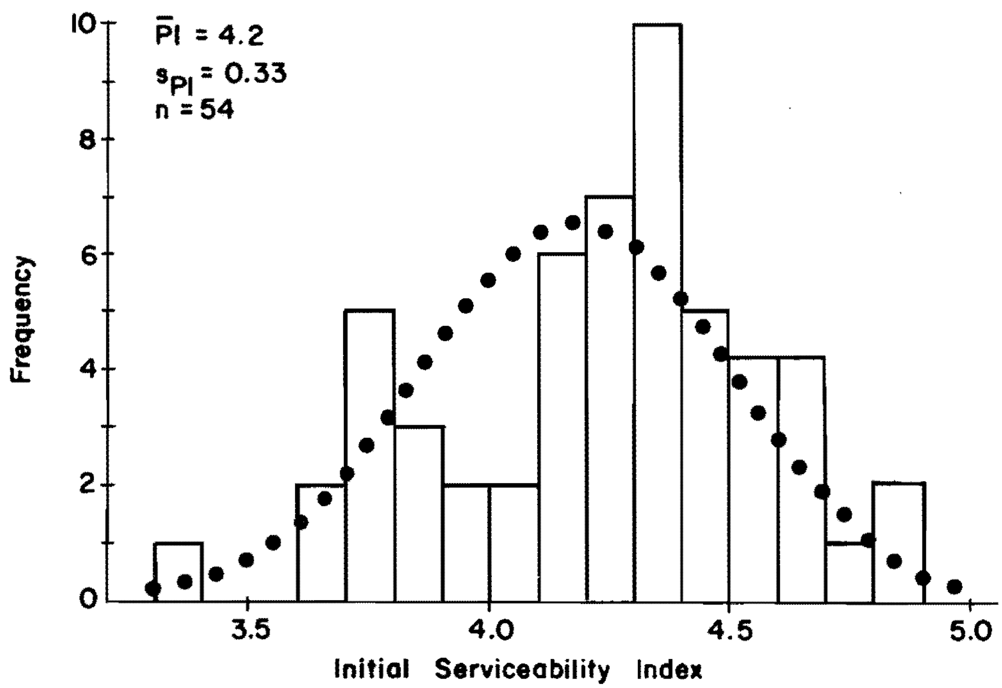


Fig 4.11. Histogram of initial serviceability index of newly constructed flexible pavement (measurements taken on 0.2-mile sections).

Pavement Smoothness. Newly constructed pavements usually show some roughness, which affects the initial serviceability index. Specification control for roughness usually includes a criterion for a maximum of, say, 1/8-inch in 10 feet. Many times a pavement surface may be within this specification criterion but will show significant roughness due to longer wavelengths which affect vehicles moving at high speeds. The Surface Dynamics Profilometer was used to obtain serviceability index values at 0.2-mile lengths along a newly constructed pavement. A histogram of initial serviceability index for an Interstate Highway is shown in Fig 4.11. The distribution of serviceability index values is fairly normal. The assumption of normality was not rejected using the χ^2 test at a level of significance of 0.05. Skewness of kurtosis was not significant either.

Variation Between Design and Actual Values

The pavement designer must estimate many important design parameters such as material properties, traffic loadings, environmental parameters, construction factors, and operation/maintenance inputs, based many times upon insufficient data. There are many reasons for the lack of complete information on which to base design values: a designer may not know exactly which aggregate source a contractor may use or may have insufficient subsurface soil data, traffic loading estimates could be in error, and so forth. Another problem that exists is that the usual design inputs are not directly controlled in the specification. The stiffness coeffients, for example, of the pavement and subgrade are only indirectly controlled through material compaction quality specifications at the present time. This section will attempt to quantify some of the variations which fall into this category, such as stiffness coefficients, thickness, smoothness, environment, and costs. Traffic load forecasting also falls into this category but is described in Chapter 5.

Stiffness Coefficients. Normally, the determination of pavement and subgrade material properties proposed for use in a pavement structure requires sampling from proposed aggregate pits or coring samples along the proposed alignment and testing them in the laboratory. There are always the problems of representative sampling, laboratory characterization of construction, environmental and load factors, and small sample sizes due to costs. For these reasons and others, a method was derived by Scrivner et al (Ref 86) for estimating material properties in situ as they exist for in-service pavements. The recommended Texas Highway Department procedure is to measure the deflections (by Dynaflect) and thickness (by coring) of similar pavement materials and similar subgrades in the area of the proposed project and then calculate the in situ stiffness coefficients. Many tests may be run in a short time period at relatively low cost. The values to be used for design may then be selected from these results. Data are not currently available to evaluate how well this procedure has worked or of the variations between the estimated design values and those actually constructed. This possible variation will be neglected for the present time in the probabilistic design procedure.

Seasonal variations of pavement deflections in Texas were investigated by Poehl and Scrivner (Ref 79). Environmental changes in pavement and subgrade stiffness are important and therefore must be considered in design if they are very large. Results of this study showed that "seasonal changes in strength do exist in Texas, that above-average deflections usually occur in the spring and summer months, and that although the magnitudes of the changes are generally smaller than those reported farther north, they are sufficiently great to warrant attention by the engineer using a deflection-based design system" (Ref 79). They also found that pavement deflections followed a sine curve with a period of one year. A most important conclusion from the standpoint of design was the following:

> In a mile or more of pavement of the same design, the odds are that the variation in deflections measured on the same day, at intervals of, say, 1000 feet will exceed the annual variation at any one of the measuring points. Thus, it appears that seasonal changes in the deflection of flexible pavements in Texas are usually less important than the random changes in the pavement subgrade system that occur in distance that, from the designer's viewpoint, are relatively short, say 1 mile (Ref 79).

These results point out that it is advisable to measure deflection of existing highways during the period when deflections in a locality are highest, but that deviation from this procedure will not greatly affect resulting stiffness coefficients for use in design.

The variability for similar materials as exist in situ for in-service highways in Texas may be estimated using data from the 181 projects that were tested with the Dynaflect. Before this variability can be estimated, the effect of existing pavement layer thickness on stiffness coefficients must be determined. A plot was made for each of the nine material types showing calculated stiffness coefficient versus pavement layer thickness. Results show that as the pavement layer thickness increases, the calculated stiffness coefficient decreases for every material type. This phenomenon may be due to the deflection model and/or the actual resilient response of the particular material. Laboratory studies have shown that for granular materials, the greater the confining pressure (or intergranular pressure), the greater the resilient modulus. The slope of the fitted regression line for each material was determined, which gives the change in stiffness coefficient per inch of material layer thickness, as shown in Table 4.2.

All untreated materials, with the exception of iron ore-topsoil (IOTS) have about the same rate of change of stiffness per inch of material, which averages 0.02 (not including IOTS). Assume that a stiffness coefficient of 0.98 has been measured for a 5-inch-thick asphalt-treated base similar to the type that is being specified for a new pavement. Also assume that the estimated design thickness for the new project is 10 inches. To adjust the measured coefficient to the thickness expected, proceed as shown:

$$0.98 - (10.0 - 5.0)0.04 = 0.78$$

The 0.78 would be the coefficient expected if the asphalt-treated layer were 10 inches thick.

The stiffness coefficient obtained from each of the 181 projects was adjusted to a 9-inch thickness so that their general magnitudes and variations could be compared for each of the nine material types. A summary of the statistics is shown in Table 4.3, along with results obtained for asphalt concrete for an average thickness of 3.5 inches. The general magnitude and range of the stiffness coefficients are illustrated in Fig 4.12. The stabilized materials show a relatively higher stiffness than the untreated materials but exhibit a higher coefficient of variation. These results clearly show the fairly wide range in stiffness that exists in situ in highway pavements for various material types. It must be realized that these variations were obtained throughout the state and that within a given highway district, they should be smaller.

Stiffness coefficients for the subgrades of all these projects were also calculated. The values ranged from 0.19 to 0.36, with a mean of 0.26. Most

TABLE 4.2. SUMMARY OF RATE OF CHANGE OF PAVEMENT LAYER STIFFNESS COEFFICIENT PER INCH OF MATERIAL LAYER THICKNESS

Material Type	Correction Per Inch of Thickness
Flexible base	0.025
Caliche	0.022
IOTS	0.061
Limestone	0.018
Sandstone	0.014
Gravel	0.016
Asphalt-treated	0.040
Cement-treated	0.080
Lime-treated	0.004
Asphalt concrete	0.055

TABLE 4.3. SUMMARY OF STATISTICS RELATING TO IN SITU STIFFNESS COEFFICIENTS FOR VARIOUS MATERIALS (ADJUSTED TO 9-INCH LAYER THICKNESS)

Material	Number Projects	Average Stiffness Coefficient	Standard Deviation	Coefficient Variation
Asphalt-treated	14	0.77	0.23	33
Cement-treated	10	1.11	0.25	22
Lime-treated	16	0.69	0.17	17
IOTS	17	0.68	0.11	16
Limestone	11	0.72	0.11	15
Sandstone	9	0.50	0.07	13
Gravel	26	0.58	0.07	12
Flexible base	33	0.58	0.08	14
Caliche	5	0.49	0.04	7
Asphalt concrete	12	1.01*	0.23	23

*For thickness of 3.5 inches

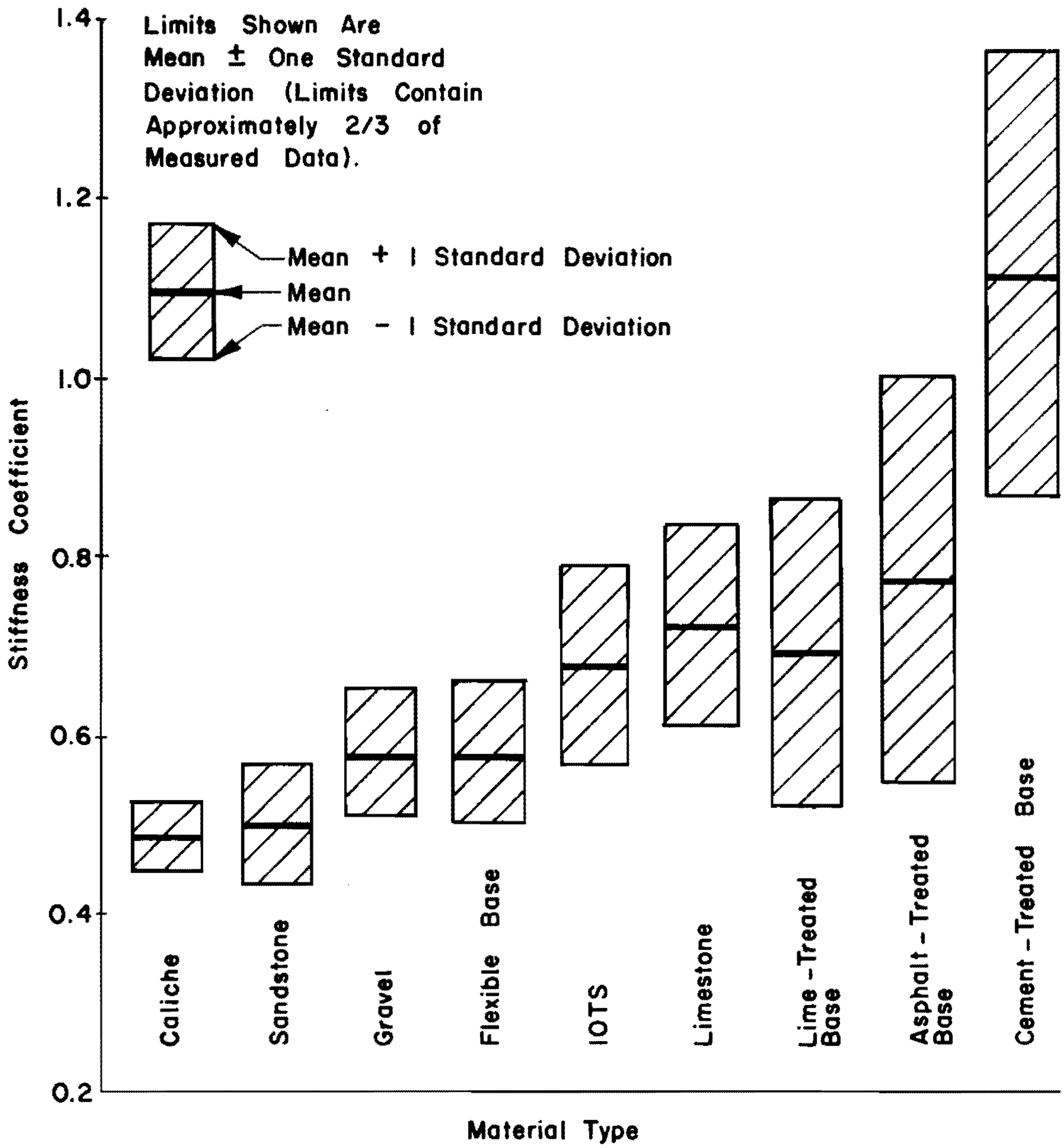


Fig 4.12. Illustration of in situ stiffness coefficients for various pavement materials (adjusted for 9-inch thickness).

values varied between 0.22 to 0.32, however, with an average coefficient of variation of 10 percent.

Environment. The environmental or climatic parameter used in the FPS is a temperature parameter α , which depends upon the maximum and minimum daily temperature in a given locality. An average temperature parameter has been estimated for each Highway Department district headquarters in Texas, as described in Ref 82. The range of α over the state is from 9 to 38. Since the temperature parameter was determined for each district headquarters over a ten-year period, it may be a fairly good estimate as far as long-time periods at a district headquarters are concerned, but it varies somewhat around a given district. Since a project located within a district will be designed with the same α , some variation will exist between design and actual temperature parameter. An estimate of the amount of variability may be found by taking the difference in temperature parameter α between each district headquarters and those surrounding the district and calculating the mean-square difference, as follows:

$$s_{\alpha}^{2} = \frac{\left[\sum_{i=1}^{q} \sum_{j=1}^{b} D_{ij}^{2}\right]}{h-2} \qquad (4.1)$$

where

D_{ij} = difference between the α for district i and an abutting district j ,

q = 25 (number of districts in Texas),

b = number of districts abutting a given district i ,

h = total number of abutting districts.

The resulting variance was determined to be $s_{\alpha}^{2} = 18.0$. This estimate will be used as representing the uncertainty associated with predicting the α for a specific project.

Pavement Smoothness. The variation in smoothness along a project has been discussed and quantified. There is also some uncertainty associated with the average smoothness as measured by the serviceability index of a newly constructed pavement. For example, a designer may estimate the average initial serviceability index of 4.5 for a new flexible pavement, but the contractor builds it with a 3.8, even though the specification criteria were enforced. Thus, the pavement would drop to minimum acceptable serviceability level sooner than predicted.

An estimate of the magnitude of this variation may be obtained from serviceability measurements made in Utah and Minnesota. Measurements in these states were made on several newly constructed pavements. A histogram of the Utah data for 76 such projects is shown in Fig 4.13. Standard errors of 0.2 for Utah (Ref 63) and 0.28 for Minnesota (Ref 51) were observed. The initial serviceability data shown in Fig 4.13 appear to be approximately normally distributed. A reasonable estimate of the project mean standard error would be 0.2. This value is considerably less than the within-project variation determined previously.

Costs. The designer must estimate unit costs in-place for each material proposed for use. It is a very difficult job to estimate what the contractor will bid for a certain material. The engineer does have available many records of material costs used in his locality, however, from which to gain information. Errors in estimating material costs may cause the selection of a non-optimum design for construction. This subject is of much importance in pavement design but is not considered further in this study.

Variation Due to Lack-of-Fit of Design Models

This type of variation or uncertainty is perhaps the most significant (and largest) of all types. Essentially, it represents the failure of the design models to predict exactly the results when actual average values of all design paramters are known.

Pavement design has made use of many empirical equations which were derived from experience and/or limited data. Even the design models which resulted from the $30 million AASHO Road Test did not contain many important material, traffic, and environmental parameters because these values were constant or not observed at the Road Test.

Due to the large lack-of-fit error associated with empirical methods, a great effort has been made in recent years to develop mechanistic pavement design methods that are based upon elastic, nonlinear elastic, or visco-elastic

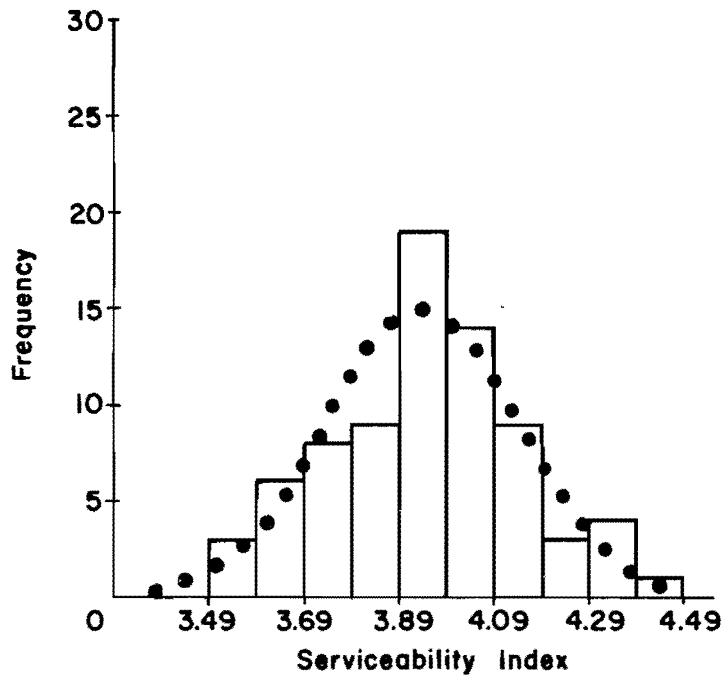


Fig 4.13. Histogram showing typical initial serviceability index distribution of highway pavements in Utah (Ref 63).

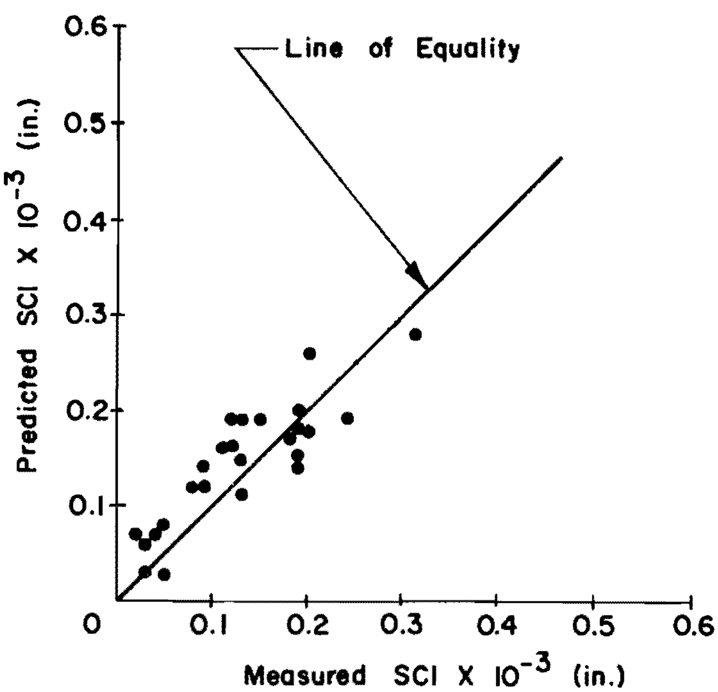


Fig 4.14. Measured SCI versus predicted SCI (data from TTI test sections, Ref 81).

theories. However, due to the tremendously complex problem of characterizing a pavement/subgrade system, it is believed that even when an approximate mechanistic method is developed, it will still contain some lack-of-fit. The magnitude may be less than current empirical procedures and hence may reduce pavement costs, as illustrated in Chapter 7.

The basic reasons for lack-of-fit are

(1) the model does not contain the proper design factors and/or

(2) the design factors used are not in proper combination within the model.

The basic design models used in the FPS structural subsystem, including the deflection and performance models, are evaluated for lack-of-fit.

Deflection Equation. The deflection equation characterizes the pavement structure response to loadings. It predicts the surface curvature index (SCI) measured by the Dynaflect if pavement layer thicknesses and strength coefficients and the subgrade strength coefficient are known.

> The equation was derived in two basic steps: (1) a mathematical model of the deflection phenomenon, containing certain undetermined coefficients was developed, and (2) the coefficients were evaluated by fitting the model to Dynaflect deflection data gathered on a set of special test sections (Ref 81).

The deflection equation is as follows:

$$SCI = W_1 - W_2 \tag{4.2}$$

where

$$W_j = \sum_{k=1}^{m+1} \Delta_{jk} ;$$

$$\Delta_{jk} = \frac{C_0}{\frac{C_1}{a_k}} \left[\frac{1}{r_j^2 + C_2 \left[\sum_{i=1}^{k-1} a_i D_i \right]^2} - \frac{1}{r_j^2 + C_2 \left[\sum_{i=0}^{k} a_i D_i \right]^2} \right] ;$$

SCI = surface curvature index inch x 10^3;

W_1 = deflection measured between the two force wheels of the Dynaflect;

W_2 = deflection measured 12 inches from point of W_1 ;

r_j = distance from point of application of either load to the j^{th} sensor, inches;

a_i = stiffness coefficient of i^{th} layer;

D_i = thickness of the i^{th} layer;

m = number of layers in pavement (not including subgrade);

C_0 , C_1 , C_2 = regression constants.

The deflection equation is explained in detail in Ref 81.

If the actual measured SCI is obtained from the special test sections and plotted versus the predicted SCI from the deflection equation, a scatter of data is found, indicating some error associated with the deflection equation, as shown in Fig 4.14.

The scatter about the line of equality is due to the

(1) lack-of-fit of the equation, and

(2) some replication or pure error due to inherent differences between strength coefficients between sections with the same materials.

Each data point is an average of five measurements within a test section. This averaging reduces the testing or within-section variation so that the remaining variation consists mainly of lack-of-fit and pure error.

Since there were no replicate sections, an estimate of pure error cannot be made. Therefore, all scatter about the line of equality will be assumed to be lack-of-fit error. The total residual is found by the following method:

$$\text{Sum of square (total residual)} = \sum_{i=1}^{i=26} (SCI_i - \widehat{SCI}_i)^2 \qquad (4.3)$$

where

SCI_i = measured SCI,

$\widehat{SCI}_i$ = predicted SCI from deflection equation,

sum of squares residual = 0.0360,

mean square residual $= \frac{0.0360}{24} = 0.00150.$

A coefficient of variation will be calculated by dividing the standard deviation by the mean SCI.

$$CV = \sqrt{0.0015}/0.127 = 0.30$$

This estimate of CV is probably not as large as it would be for actual in-service pavements built with materials different from those used at the TTI test facility.

There is also a lack-of-fit model error in predicting the SCI after an existing pavement has been overlaid. A gross estimate of the magnitude of this error was obtained from deflection measurements taken on 11 in-service pavements. The SCI was measured before and after overlay at the same location as reported in Ref 9. The mean-square difference between the actual measured SCI after the overlay and the predicted SCI after the overlay (by Eq 4.2) was determined using Eq 4.3. The coefficient of variation of this error was estimated by dividing the standard deviation by the mean SCI for all projects.

$$CV = \sqrt{0.00922}/0.253 = 0.38$$

This coefficient of variation contains variations in overlay material properties, variations in thickness of overlay, and the lack-of-fit of the deflection model. There is no way to break out these components of variation using the available data. It is believed that the majority of the error is due to lack-of-fit of the deflection model, however.

Performance Equation. The performance equation predicts behavior of the pavement based on the present serviceability concept developed at the

AASHO Road Test. The loss in serviceability of a pavement depends on deflection of the pavement structure, the number of load applications, temperature, and foundation movements due to swelling clays. The effect of swelling clay will not be considered in this analysis. The performance equation developed for FPS is as follows:

$$\text{Log } N = \log Q + \log \alpha - 2 \log SCI - \log B + 6.0 \tag{4.4}$$

where

- Q = $\sqrt{5 - P2} - \sqrt{5 - P1}$, function of serviceability loss;
- $P2$ = minimum acceptable serviceability index;
- $P1$ = initial serviceability index;
- B = regression coefficient = 53.6 (or 0.134 if the "partial deflection" produced by any given axle load is used for SCI and the number of applications of that load is used for N);
- SCI = SCI of pavement structure, as measured by the Dynaflect, in inches $\times 10^{-3}$;
- N = number of predicted equivalent 18-kip single-axle load applications; and
- α = temperature parameter described in Ref 82.

This equation was derived using data from the AASHO Road Test as explained in Ref 82. There is a certain error associated with the prediction equation. Part of the error is due to the fact that the Dynaflect deflections were obtained through correlations with axle-load deflections. The total error may be estimated by direct comparison of the actual number of load applications an AASHO Test pavement carried until it dropped out of service to the number of load applications predicted by Eq 4.4. There were ten performance periods used to develop data for Eq 4.4. These periods were each characterized by a constant α and a constant S . Therefore, the N predicted for a given test section was calculated and summed for each period that the section was in service. The predicted N by Eq 4.4 will be denoted by $\hat{N}$ and the actual by N in the following analysis.

For example, test section 315 lasted six performance periods and the predicted $\hat{N}$ was calculated as follows:

$$\hat{N}_{315} = \hat{N}_{\text{period 1}} + \hat{N}_{\text{period 2}} + \ldots \hat{N}_{\text{period 6}} ,$$

$$\hat{N}_{315} = 268{,}600 \text{ predicted load applications,}$$

$$N_{315} = 247{,}700 \text{ actual load applications.}$$

The N versus $\hat{N}$ results from 84 test sections are shown plotted on log paper in Fig 4.15. The data show about equal scatter bands from 3,500 to 550,000 load applications.

The mean square residual due to lack-of-fit of Eq 4.4 for predicting the life of AASHO Road Test sections was calculated as follows:

$$s_{l.o.f.}^{2} = \sum_{i=1}^{r} \left(\log N_i - \log \hat{N}_i\right)^2 \Big/ r-2 = 0.0812 \tag{4.5}$$

where

N_i = actual load applications for i^{th} section;

$\hat{N}_i$ = predicted load applications by Eq 4.4 for i^{th} section

r = number of sections = 84.

This estimate of lack-of-fit error is valid only for AASHO Road Test pavements and would probably be considerably larger when the design equation was used for pavements located in Texas, with widely varying environment, traffic, and materials. This value is the best estimate available at the present time but should be considered a "minimum" value.

The mean absolute residual ([log N - log $\hat{N}$]) for the 84 sections was 0.21. This value is less than the mean absolute residual obtained from the equations derived in the AASHO Road Test report (Ref 98) of 0.23.

The error to be included in the lack-of-fit of Eq 4.5 is $s^2_{l.o.f.}$ = 0.0812. This variance due to lack-of-fit occurs mainly because the performance model does not

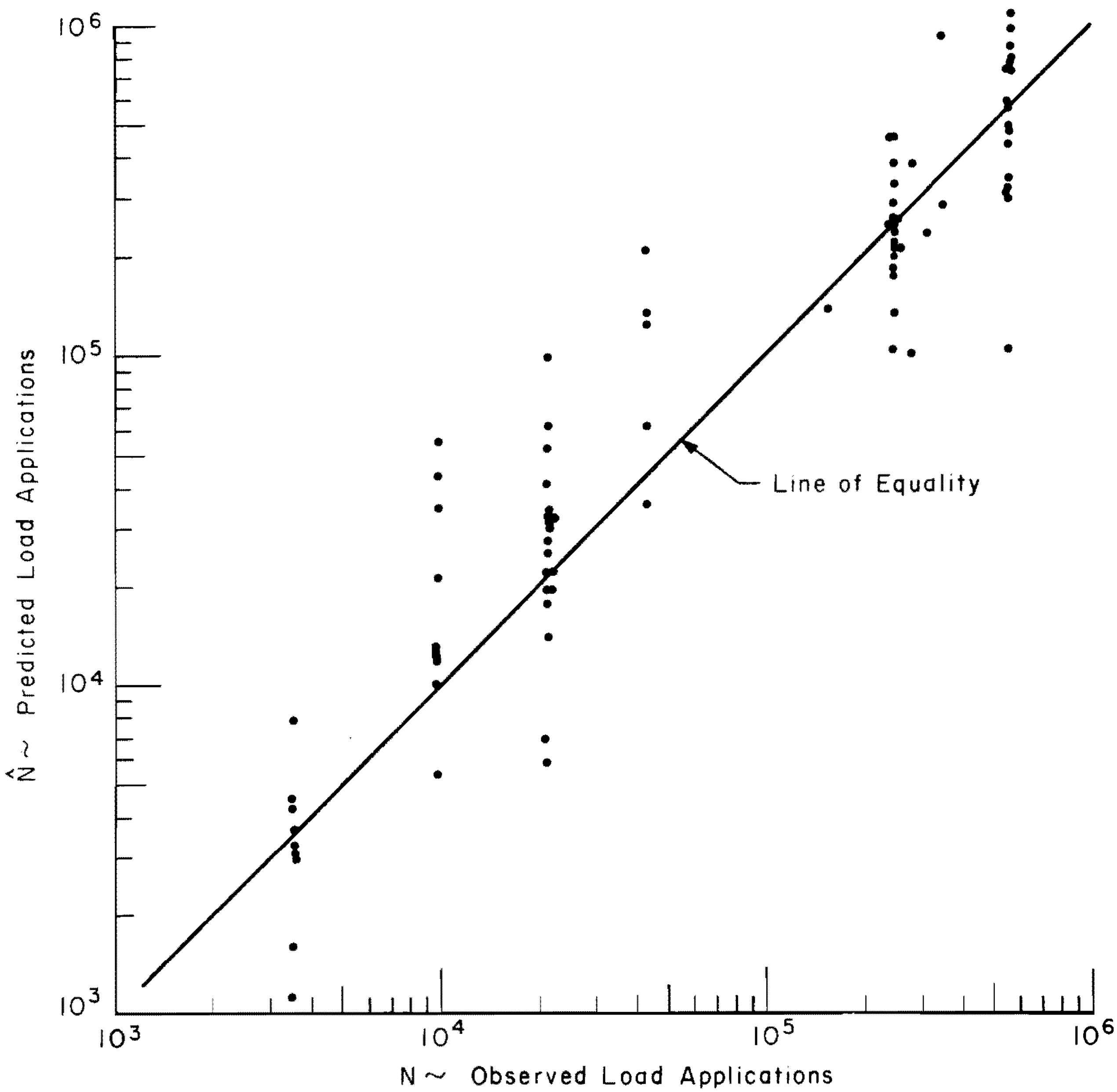


Fig 4.15. Observed N versus predicted $\hat{N}$ - AASHO Road Test data (each point represents a test section).

(1) contain all necessary terms and/or

(2) is not the correct combination of terms.

The distribution of lack-of-fit errors may be determined by observing a histogram of difference values of $\log N - \log \hat{N}$, which is shown in Fig 4.16. The χ^2 goodness of fit test failed to reject the hypothesis of normality. The skewness and kurtosis tests also fail to show significance at a level of significance of 0.05. Therefore, it appears that the lack-of-fit associated with predicting the number of load applications to failure of the AASHO Road Test sections is approximately log normally distributed.

The actual lack-of-fit of the performance model for the prediction of load applications for highways in Texas under greatly differing soil conditions, environmental conditions, and mixed traffic loadings is probably much larger than the AASHO estimate. Stabilized base courses, for example (with asphalt, cement, or lime), are used extensively at the present time in Texas. This type of base was essentially not used at the Road Test and therefore greater lack-of-fit errors may be associated with the design system when stabilized bases and subbases are used.

Determination of the actual lack-of-fit error of the performance model has been started using data obtained from pavements that have been in-service for several years. Certain data were collected from six projects in their first performance period before any overlay had been placed. Two of the projects had pavement structures that contained untreated aggregate bases and subbases. The other four projects contained either cement-treated, asphalt-treated, or lime-treated bases and subbases.

The following data were obtained from each project.

(1) initial serviceability index after construction (estimated),

(2) current serviceability index along pavement for 0.2-mile sections (measured with Mays Road Meter),

(3) total 18-kip equivalent single-axle loads since construction (estimated by the Planning Survey Division, Texas Highway Department),

(4) surface curvature index measurement along project within same 0.2-mile sections measured for serviceability (measured with Dynaflect), and

(5) district temperature parameter (estimated).

The predicted $\log N$ was calculated for each project using these data and Eq 4.4 considering the loss of serviceability from the initial to the

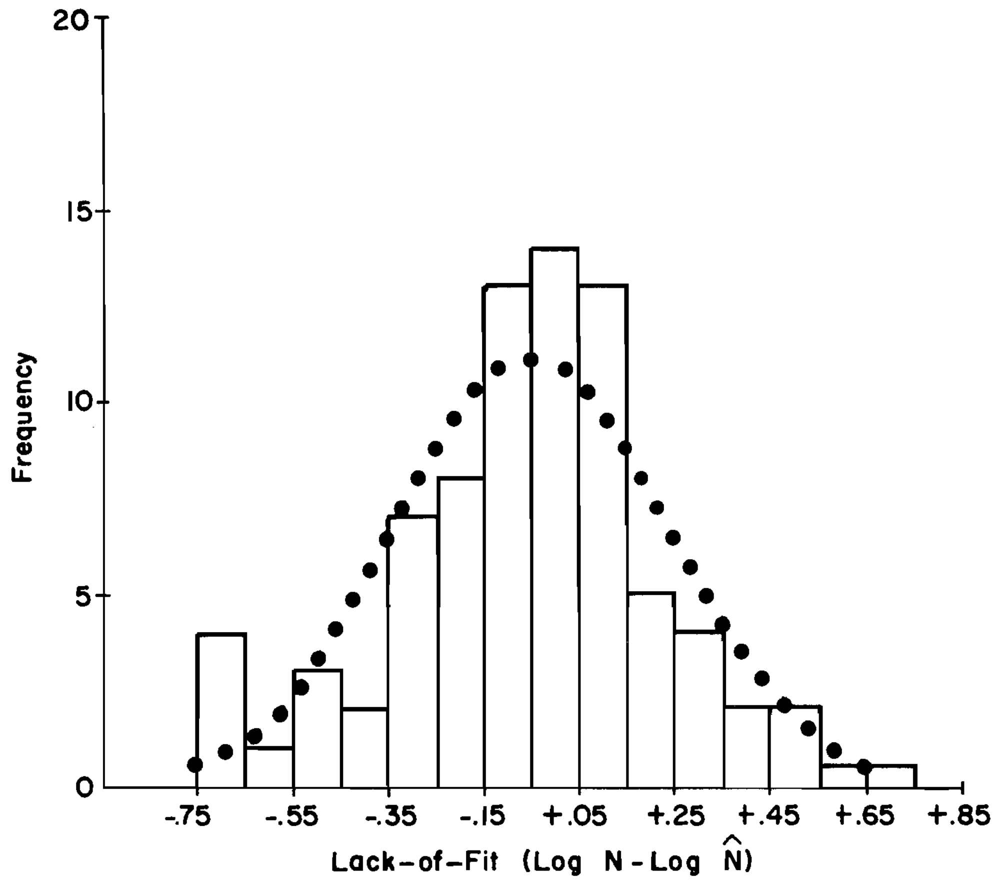


Fig 4.16. Histogram showing lack-of-fit in predicting life of AASHO Road Test sections using performance model.

present measured value. A summary of the predicted and the actual traffic load applications (as estimated by the Planning Survey Division) is given in Table 4.4. The first two projects, which contain untreated aggregate bases and subbases, show predicted log N reasonably close to the actual log N . The four projects containing treated base and subbase, however, show large differences between predicted and actual with predicted being larger than actual.

The performance model depends heavily upon the SCI of the pavement/ subgrade system due to the magnitude of its exponent. Pavement structures containing treated bases and subbases are very stiff and cause low values of SCI thereby causing the predicted log N to be large. The pavements containing treated bases and subbases are apparently not performing as well as this prediction and consequently the model shows large lack-of-fit error. Some of the data used to calculate the results shown in Table 4.4 were estimated and not actually measured and therefore these values are subject to some error. The analysis which has been briefly described will be further explained in Chapters 6 and 7. Work has been performed by Jain, McCullough, and Hudson (Ref 49) that helps explain this result. This illustrates the important need for upgrading the performance model.

TABLE 4.4. SUMMARY OF DATA FROM SIX IN-SERVICE HIGHWAYS SHOWING AVERAGE PREDICTED AND ACTUAL 18-KIP EQUIVALENT LOAD APPLICATIONS

Project	Type* Pavement Structure	Predicted Log N (Eq 4.4)	Actual Log N
US 59 (Bowie Co.)	Untreated aggregate	5.7925	6.0294
IH 35 (Bexar Co.)	Untreated aggregate	6.3404	5.9675
US 69 (Angelina Co.)	Asphalt and lime-treated	6.9275	5.6335
US 59 (Polk Co.)	Cement-treated	7.0939	5.6730
US 59 (Harrison Co.)	Cement-treated	7.0939	5.8887
US 59 (Polk Co.)	Asphalt and cement-treated	6.7525	5.9069

* All pavements contain an asphalt-concrete surfacing layer.

CHAPTER 5. TRAFFIC LOAD FORECASTING VARIATIONS

The stochastic nature of many factors associated with estimating allowable load applications of a flexible pavement/subgrade system has been summarized. This chapter presents the forecasting procedures and associated variations in estimating the applied traffic loadings to which the pavement system will be subjected. The results of this chapter, along with those from the preceding chapter, will be used as "inputs" to the probabilistic design theory incorporated into FPS in Chapter 6. The general concepts are described, followed by a description of the Texas Highway Department method for forecasting load equivalencies, and finally an estimation is made of the various uncertainties involved.

General Concepts

The accumulative damage caused by repetitive application of traffic loads and the conversion of mixed traffic loadings to a base equivalent wheel load have been discussed in many reports (Refs 1, 17, 36, 83). The analysis of complex loadings which pass over a highway is facilitated greatly by expressing the destructive effects of the many types and magnitudes in terms of equivalent numbers of applications of some base load. At the present time, 38 state highway departments along with Texas use the equivalency factors for a base 18-kip axle load which were derived at the AASHO Road Test for the design of flexible pavements.

The FPS pavement design system requires an estimation of the total number of equivalent 18-kip single axle loads which will pass over the critical pavement lane during the design analysis period. There are many uncertainties associated with estimating the total equivalent load applications which a pavement lane may carry during the design analysis period. Basically, these uncertainties can be grouped into three types:

(1) estimation of total number of axles which will pass over the pavement lane during the period,

(2) estimation of axle weight distribution, and

(3) conversion of mixed traffic to 18-kip equivalent axle loads.

As an example of the overall uncertainties involved in traffic estimation, the Kentucky Highway Department made a statistical evaluation (Ref 16) of predicted versus actual accumulations of equivalent wheel loads (EWL's). This evaluation was made on 57 projects designed and constructed between 1948 and 1957. The results indicated that 68 percent of those roads did or would accumulate their ten-year estimate of traffic between 6.8 and 16.8 years. Assuming the distribution of years to be normal, a standard deviation may be estimated from the fact that 68 percent of the data would be contained within plus and minus one standard deviation from the mean:

$$s = \frac{16.8 - 6.8}{2} = 5 \text{ years}$$

A standard deviation of five years is quite large and illustrates the magnitude of the uncertainties involved in estimating future equivalent traffic loadings.

Deacon and Lynch (Ref 16) conclude the following:

> The prediction of equivalent wheel loads is much more complicated than predicting gross traffic volume - although any error in predicting gross traffic compounds the error in equivalent loadings. For instance, the composition of traffic and the spectral distribution of truck types and axle weights are extremely elusive and variable factors - even in retrospection.

After an extensive investigation into the ability to forecast equivalent wheel loadings over the last 15 years, they state the following about the possible magnitude of error in loading prediction:

> ... the inherent or natural variability remains high and so does the error of estimate. This does not mean that all predictions will be hopelessly in error: it does mean that in some instances, the actual accumulation may be somewhere between half and twice the predicted value but in the majority of cases will conform much closer. (Ref 16).

<u>Texas Highway Department Method</u>

The basic steps used to estimate the total number of equivalent axle loads a <u>particular</u> highway pavement will carry for the design analysis period are as follows (Ref 11):

(1) total traffic volume projection and percent trucks estimation,

(2) axle factor estimation,

(3) axle load distribution estimation, and

(4) total 18-kip axle load equivalencies calculated.

Each of these steps is described below as given in Ref 33.

Total Traffic Volume Projection. The Texas Highway Department (THD) considers four major factors in estimating future traffic volumes for rural highways:

(1) Existing traffic: probably the most important function of existing traffic is in establishing growth trends, truck percentages, etc., for various types of land usage.

(2) Diverted traffic: in forming a guide for predicting the traffic that will use a specific design section of new highway, the existing traffic which will be diverted from other routes, such as major highways, arterial streets, secondary highways, etc., must be determined.

(3) Generated traffic: this additional traffic cannot be accounted for by diversion from other routes or by normal growth. Generated traffic is composed of "(1) vehicle trips which would not have been made at all, or would have been much less frequently, if a more attractive route providing better travel conditions were not available, (2) vehicle trips which would have been made to other destinations or from other origins if the route had been less attractive, and (3) vehicle trips which would have been made by other forms of transportation if a high type facility were not available" (Ref 33).

(4) Traffic growth: any increase expected in traffic volume with time must be estimated. Historical data of ADT on many Texas Highways have been kept since the middle 1930's. The Annual Report (Ref 4) published by the Planning Survey Division of THD gives yearly data from 1955 at 163 permanent automatic traffic recorders. Example plots of three highway historical ADT's are shown in Fig 5.1. Land usage in the area of the specific highway can be evaluated and projected for possible increase in the traffic growth factor. The final annual traffic growth can then be applied as a percentage to a combined figure of existing, diverted, and generated traffic at arriving at a future traffic volume for a given year.

Projections of traffic volumes for urban highways also consider many of these factors. However, volume forecasting must consider the "highway system" development within the urban area. Many highways within urban areas are filled almost to capacity soon after completion. This is usually due to the phase development of a total highway system and when other routes within the system are completed, the volume should reduce. Therefore, traffic volume projection in high density urban areas considers the factors listed above and the entire urban transportation system.

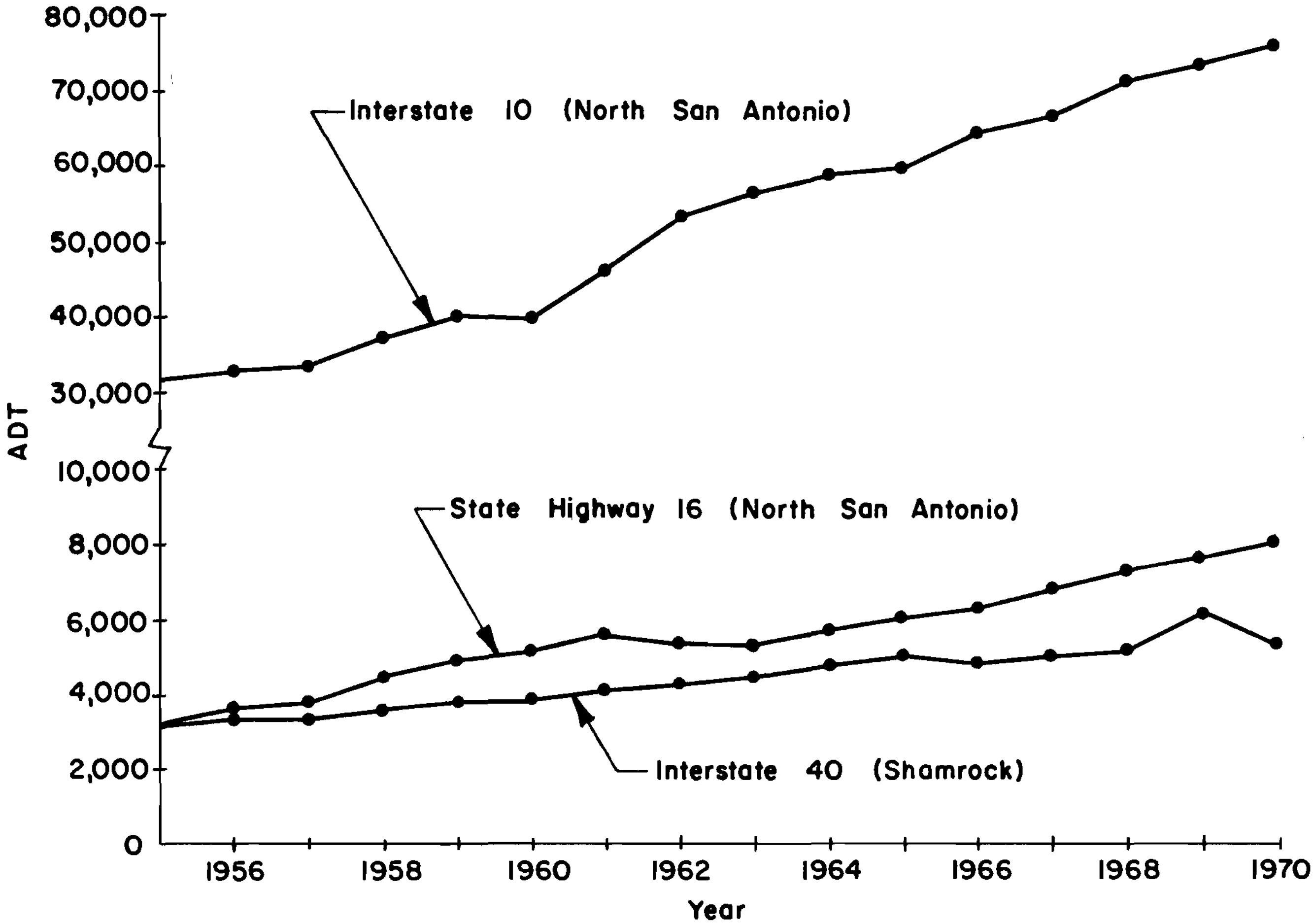


Fig 5.1. Illustration of ADT change with time for three Texas highways.

The ADT for design is determined from the following equations:

$$ADT_{design} = \left[ADT_{initial} + ADT_{final}\right] /2 \qquad (5.1)$$

where

$$ADT_{initial} = ADT_{existing} + ADT_{generated} + ADT_{diverted} ,$$

$$ADT_{final} = ADT_{initial}[1 + GM] ,$$

G = average growth percentage (in decimal form) per year, and

M = number of years in analysis period.

Percent Trucks. In Texas the percentage of trucks in the traffic flow has been found to be somewhat dependent upon the ADT. A study of all the manual classifications and count stations in Texas for the period 1951-1961 was made by Derdeyn (Ref 18) and the percent truck traffic determined. Figure 5.2 shows the highest percentage of trucks that was experienced for any given ADT on Texas highways during 1959-1961. Approximately 96 percent of all stations plotted were below the line. The majority of the stations which were above the line had less than 4000 ADT. Very few stations had less than 4 percent truck traffic. A curve depicting the change in percent trucks to ADT is established for a section of highway in question, which usually falls between a flat 4 percent and the upper line shown in Fig 5.2. All manual vehicle classification data in the general vicinity are studied in making an estimate of truck percentage.

Axle Factor Estimation. The total number of axles passing over a specific roadway must be known to estimate the total 18-kip equivalent axle loads for the design analysis period. An adjusting factor must be used to convert the number of trucks to axle loads. This adjusting factor (axles per truck) is

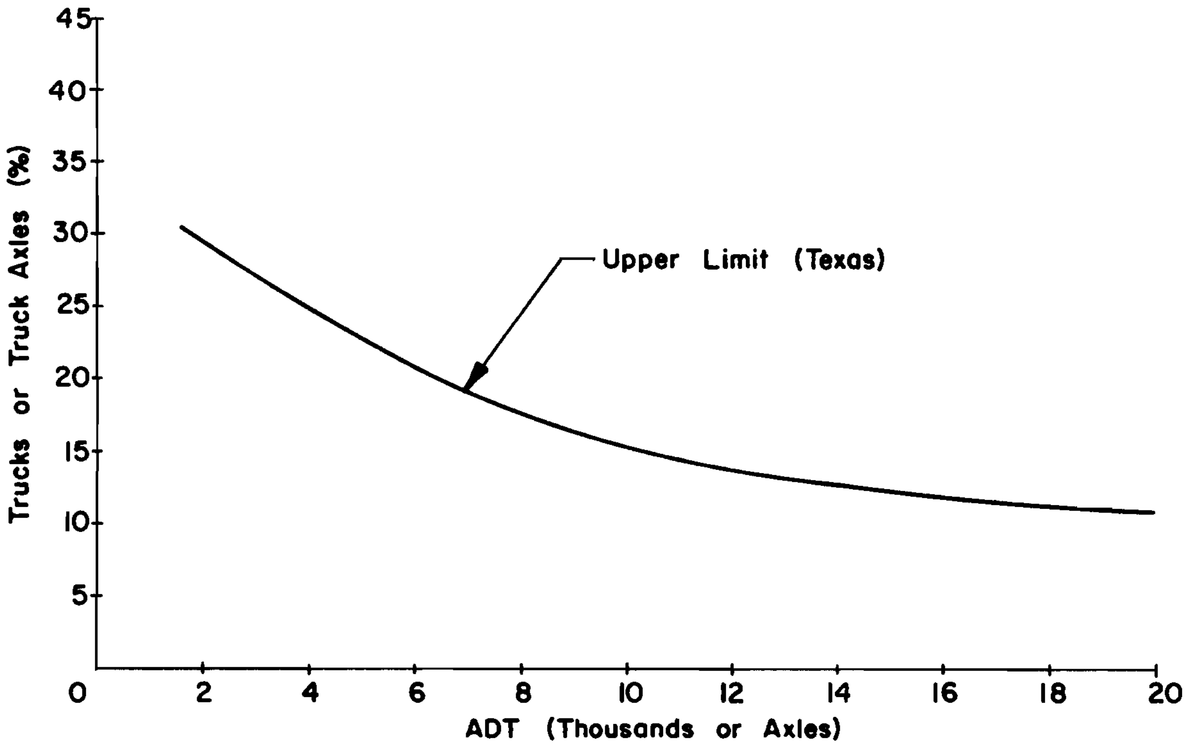


Fig 5.2. Relationship of percent trucks to ADT (Ref 18).

called an axle factor (Ref 39). For this analysis, a tandem axle is considered to be one axle, i.e., carrying one axle load, in order to conform to the AASHO load equivalency factor.

Heathington and Tutt made an investigation (Ref 39) of vehicle classification data collected on Texas highways from 1960 to 1963. They found a very good linear correlation between the number of trucks and number of axles passing a given point. The average axle factor was found to be 2.75 axles per truck. This figure is not used by THD without first reviewing the manual classification data for a given highway.

Distribution of Axle Loads. The distribution of axle loads for a given section of highway is estimated using data from loadometer stations and vehicle classification counts. The THD operates 21 loadometer stations. These stations now operate only during the summer months. A detailed description of the weighing times and procedures is given in Ref 11. Vehicle classification counts are taken at about 188 manual count stations, 21 of which are the permanent loadometer stations mentioned above. The procedures are also described in Ref 11.

Estimating the distribution of axle loads is probably the most difficult task. It is performed by experienced employees of the Planning Survey Division, based on knowledge of the area in which the new design is located. The data from loadometer stations and manual counts are used in arriving at a distribution. Manual count data are used as a reference for highway routes within this same area of the state that have similarities, such as terrain, traffic volume, and land usage. The distribution may be determined by one of the following methods:

(1) The design highway may be located at a loadometer station and/or classification location, or within the immediate area, and have basically the same traffic characteristics as the loadometer station. The distribution of the loadometer station would be used.

(2) The distribution may be estimated by grouping all 21 loadometer stations over the state by one of the following ways:

 (a) percent trucks,

 (b) highway systems, or

 (c) statewide area.

(3) Some combination of 1 and 2.

The axles are divided in single and tandem sets into weight classes of 1000 pounds to obtain the frequency distributions of axle weights for a station or group of stations. This frequency distribution expressed in percentages is used as an estimate of the mixed traffic load and is projected over the design life. It is assumed that it does not change over the design life.

Directional and Lane Distribution. There are two additional factors which are considered in forecasting the total load experience of a pavement. These factors are the directional distribution and lane distribution of trucks. These factors will vary from highway to highway and must be considered for each specific project. The directional distribution is normally a 50/50 distribution of heavy trucks in each direction (Ref 65) over the design life of a pavement (there exist notable exceptions, however). For multilane facilities, it is recognized that not all trucks use the same lane. Values currently used by various states to estimate the lane distribution factor (which is defined as the percent of the total number of trucks in the heaviest traveled truck lane) and summarized by McCullough et al (Ref 65) are as follows:

Lanes in Both Directions	Percent Trucks in Heaviest Traveled Lane
2	100
4	80 - 100
6+	60 - 80

These results were experimentally verified by a recent study by Alexander and Graves (Ref 3). For several highways in each category throughout the state of Georgia, they found that four-lane rural highways showed 93.5 ± 5.8 percent, and six-lane urban highways were 60.2 ± 8.0 percent. The standard errors are quite substantial, however, and show the variation from highway to highway.

Total 18-Kip Equivalencies Calculated. Given the design ADT, the percentage of trucks, the distribution of axle loads for trucks, and the directional and lane distribution, the total 18-kip equivalent loads can be calculated.

(1) The total number of axles in the design lane for the entire design period is determined (for trucks only):

$$E = \left[ADT_d\right]\left[A\right]\left[L\right]\left[T\right]\left[DD\right]\left[LD\right] \tag{5.2}$$

where

E = total number of axles for design period over design lane,

ADT_d = $(ADT_i + ADT_f)/2$,

ADT_i = ADT at beginning of design period,

ADT_f = ADT at end of design period,

A = average number of axles per vehicle,

T = percent trucks of ADT,

L = number of days in design period,

DD = directional distribution factor, and

LD = lane distribution factor.

(2) The total equivalent 18-kip single-axle loads may now be calculated:

$$n = \left[\sum_{i=1}^{i=k} P_i EF_i\right]E \tag{5.3}$$

where

P_i = percent of axles in i^{th} load group,

EF_i = AASHO load equivalence factor for given axle group and pavement structural number,

k = number of load groups.

The symbol ΣPE will be used to denote the overall summation of P_i and EF_i for each axle group.

Estimation of Variations

The total equivalent 18-kip single-axle loads to which a pavement may be subjected depends upon several factors which have been discussed for the THD method. Each of these factors is difficult to predict accurately and therefore has uncertainty associated with it. In statistical terms, these factors are random or stochastic variables which follow some probability distribution. As in the case of predicting the allowable load applications, the problem of determining the overall error associated with estimating the n becomes one of determining the magnitude of each of the variables included in Eq 5.3 and using statistical theory to calculate the overall variance. The latter phase is accomplished in Chapter 6. The variance of the design ADT, percent trucks, axle factor, axle weight distribution, conversion of axle weight distribution to equivalent loads, and directional lane distribution are presented below.

Variation of Design ADT, Axle Factor, and Percent Trucks. The design ADT represents the overall average ADT for the entire analysis period. It depends upon the values of initial ADT (ADT_i) and the overall growth rate (G). There is uncertainty in estimating the existing, diverted, and generated traffic, which means that there will be variation in estimating the ADT_i because this prediction must usually be made several years before the project is opened to traffic. These factors are estimated by sampling, which always has an accompanying error associated with predicting population parameters. The growth factor presents a more difficult estimation problem however. The ADT increase (or decrease) throughout an analysis period depends upon many social-economic conditions, population change, land use along the facility, and many other factors. If accurate past history of traffic growth is available (as illustrated in Fig 5.1), then the estimate of future growth will probably be much better. It appears reasonable to conclude that the magnitude of the standard error in prediction of design ADT will depend upon the magnitude of ADT, indicating a constant coefficient of variation. For existing highways where past growth data are available, the coefficient of variation in predicting design ADT may range from 10 to 20 percent. New highway locations, particularly in urban areas, may have from 15 to 30 percent coefficient of variation associated with predicting design ADT.

These estimates of variation can be illustrated by assuming that volumes are being projected for two highways and that the design ADT average is 5000 and 25,000. Assuming prediction error to be normally distributed, the 95 percent confidence interval for the mean would be, assuming a 15 and 22 percent coefficient of variation, respectively,

$$5000 \pm 1.64\ (.15 \times 5000)\text{: } 3800 \text{ to } 6200$$

$$25000 \pm 1.64\ (.22 \times 25000)\text{: } 16000 \text{ to } 34000$$

The axle factor estimation may be fairly accurate for a given time period as was shown by Heathington and Tutt (Ref 39). The average axles per truck may change with time, however, if the type of vehicle changes. Coefficient of variation may range from 5 to 15 percent.

Percent trucks is a reasonably stable value and has not, in the past, changed drastically with time. The coefficient of variation would range from 10 to 15 percent. This variation can be illustrated by assuming an average percent trucks of 13 and a coefficient of variation of 10 percent and determining the 95 percent confidence interval for the mean:

$$13 \pm 1.64\ (0.10 \times 13)\text{: } 10.9 \text{ to } 15.1 \text{ percent}$$

Variation of Axle Load Distribution. The possible error in estimating an average axle weight distribution throughout the analysis period for a given highway location could be very significant. Buffington, Schafer, and Adkins (Ref 11) investigated the accuracy of determining the axle weight distributions of Texas highways using data from 21 loadometer stations in Texas. An analysis of axle weight distributions developed from previously collected loadometer data showed the following:

(1) Significant differences exist between most of the loadometer station and highway system averages within vehicle type. Even the grouping of stations according to highway system (I.H., U.S., F.M.) failed to produce homogeneous weight distributions. Various geographical groupings of stations also showed significant differences.

(2) An analysis of axle weight samples taken at loadometer stations showed that part of the station-to-station variation in the averages of vehicle and axle weights is due to differences in the weighing schedule. Additional between-station variation is due to small samples. Therefore, samples from the 21 loadometer stations combined to produce a more accurate estimate of true population variance than samples from only one station.

> Also, the fact that at present the 21 loadometer stations only weigh vehicles in the summer may induce error in the axle weight distribution. Reference 11 concludes that none of the estimating procedures they tried produced station estimates within 10 percent of the actual value for all stations.

The study also attempted to determine just how accurate combined station weight frequency distributions, determined by grouping for various highway types (I.H., U.S., F.M.), would be in making 18-kip equivalency estimates at individual stations. The trial axle weight frequency distributions were generated from 1964-68 loadometer data. The 18-kip total axle loadings were calculated for each of the 21 stations, using the average axle distribution for that particular highway type (either I.H., U.S., or F.M.). Then the actual 18-kip load applications were calculated using the actual axle weight distribution as measured at the loadometer station.

The results of this analysis are shown in Table 5.1. The actual total 18-kip axles are plotted versus the estimated for each of the 21 loadometer stations in Fig 5.3. The percent error ranged from -31.5 to +28.7 percent. The error associated with this estimating procedure may be calculated by summing the squared difference between the actual and estimated values for all 21 loadometer stations and dividing by the degrees of freedom (n-2) (Ref 19).

$$s^2_{\log \Sigma PE} = \frac{\sum_{1}^{21} [\log(\text{actual}) - \log(\text{estimated})]^2}{21 - 2} = 0.0037 \qquad (5.4)$$

This estimate of error is lower than actual error which may occur at various random highway locations throughout the state, due to the method of analysis. Another study in Texas (Ref 39) compared the total 18-kip equivalent loads estimated at three loadometer stations to the total 18-kip axle loads calculated from the axle weight distribution at the three stations. Three methods were used to estimate the distributions.

(1) grouping of data by percent trucks,

(2) grouping of data by highway system, and

(3) grouping of data by statewide area (all loadometer stations).

TABLE 5.1. PERCENTAGE ESTIMATING ERRORS FOR HIGHWAY GROUPING SYSTEM GENERATED BY TEXAS CARGO VEHICLES WEIGHED AT EACH LOADOMETER STATION DURING 1964-1968 (Ref 11)

Loadometer Station by Highway System	Actual Total Weight in 18-Kip Axle Equivalents	Estimated Weight in 18-Kip Axle Equivalents by Highway Systems Group	Percentage Estimating Error
Interstate Rural			
10-1	816	918	12.5
10-2	2,812	2,817	0.2
20-1	3,801	3,700	-2.6
20-2	4,194	4,375	4.3
20-3	3,560	3,437	-3.5
30-1	3,384	3,610	6.7
35-1	4,728	4,915	4.0
37-1	1,595	1,848	15.9
45-2	5,625	4,893	-13.0
Other Rural			
7	750	967	28.9
16	1,503	1,845	22.8
20	2,398	2,482	3.5
42	954	895	-6.2
72	2,917	2,896	-0.7
81	2,164	1,482	-31.5
88	1,458	1,282	-12.1
145	2,359	2,599	10.2
147	788	794	0.7
149	1,176	1,228	4.4
Urban			
3	521	473	-9.1
4	266	314	17.9
Average:	2,275	2,460	10.0

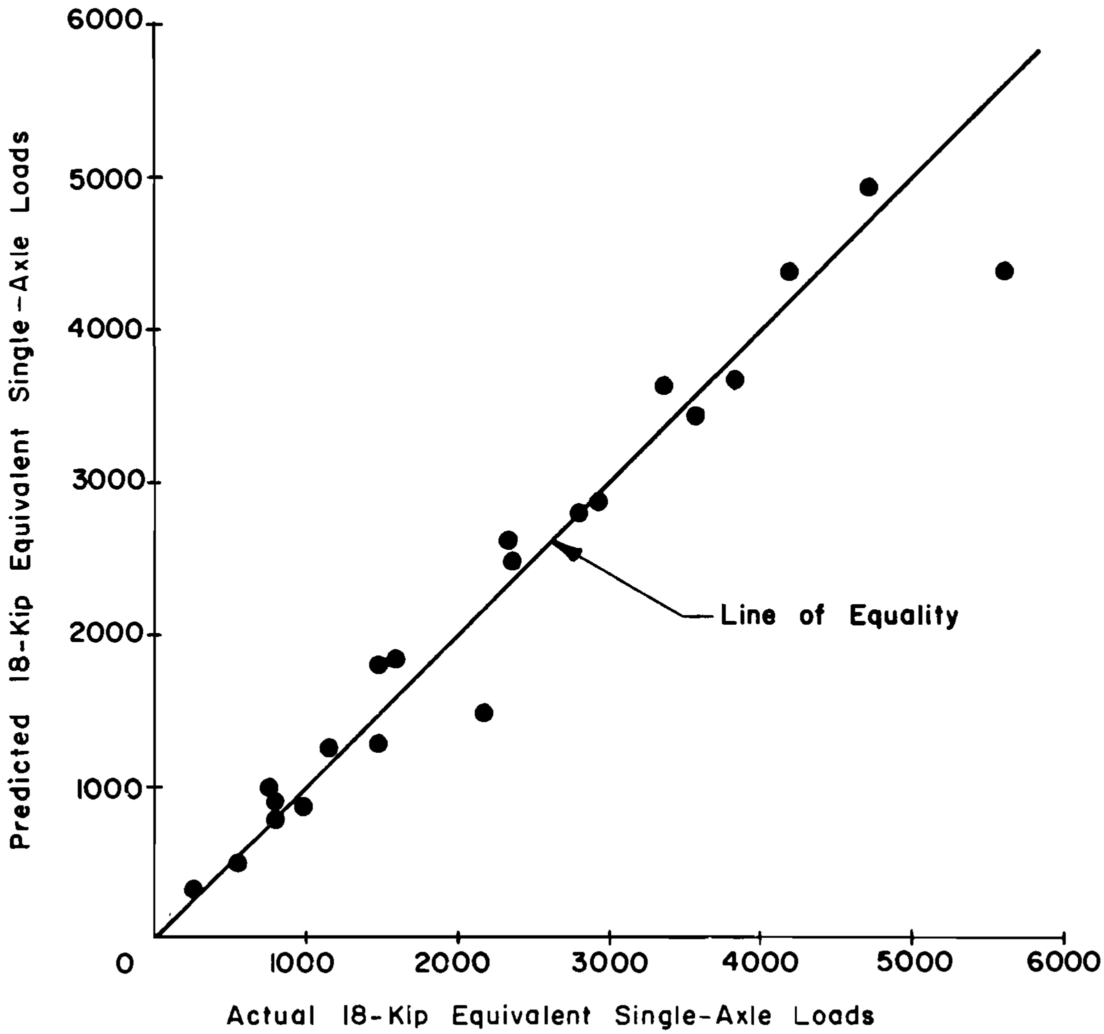


Fig 5.3. Actual versus estimated 18-kip equivalent loads at 21 loadometer stations in Texas.

The results are summarized in Table 5.2. The percent error varies from -7 percent to +50 percent. The overall variance of the three estimating procedures at the three loadometer stations can be estimated from the following:

$$s^2_{\log \Sigma PE} = \frac{\sum_{1}^{9} [\log(\text{actual}) - \log(\text{estimated})]^2}{9 - 2} = 0.0229 \qquad (5.5)$$

These variances can be compared to that obtained from the State of Kentucky. Deacon and Lynch made an extensive investigation into the prediction and projection of EWL's on Kentucky highways (Ref 16). They estimated EWL's using a new proposed method which they derived using a considerable amount of data from 1950 to 1968. EWL's were estimated using the new proposed method and compared to actual EWL's for all stations at which both vehicle classification and weight data had been obtained during the study period. For example, a plot of predicted and actual daily EWL's, shown in Fig 5.4, shows the year-to-year variations at a particular loadometer station. Figure 5.5 shows that if EWL's are accumulated over a period of years, the actual and predicted accumulations might tend to converge. This figure (from Ref 16) shows that for several stations, the results converge after longer time periods.

Deacon and Lynch (Ref 16) also extrapolated to 20-year accumulations the actual and the estimated EWL's for each of 20 locations. Figure 5.6 shows a plot of actual versus estimated EWL's. The total variance of the log of total EWL's may be calculated as for the Texas method.

$$s^2_{\log \text{EWL's}} = \frac{\sum_{1}^{20} [\log(\text{actual}) - \log(\text{predicted})]^2}{20 - 2} = 0.0453 \qquad (5.6)$$

The three estimates of the variance in predicting an axle load distribution are considerably different in magnitude. The best estimate for Texas conditions is probably that given in Eq 5.5 as this was the largest value found in Texas for three different methods of grouping.

TABLE 5.2 EQUIVALENT 18-KIP SINGLE-AXLE LOAD RESULTS OF THREE METHODS OF GROUPING DATA TO ESTIMATE AXLE-WEIGHT DISTRIBUTION (Ref 39)

Grouping Method	Loadometer Station		
	L-35-1	L-30-1	3-Up
Statewide area	10,209,820	11,220,708	6,686,146
Type highway	8,783,847	11,134,830	7,349,439
Percent trucks	11,141,503	12,089,317	4,683,927
Actual data	7,420,760	8,251,685	5,050,296

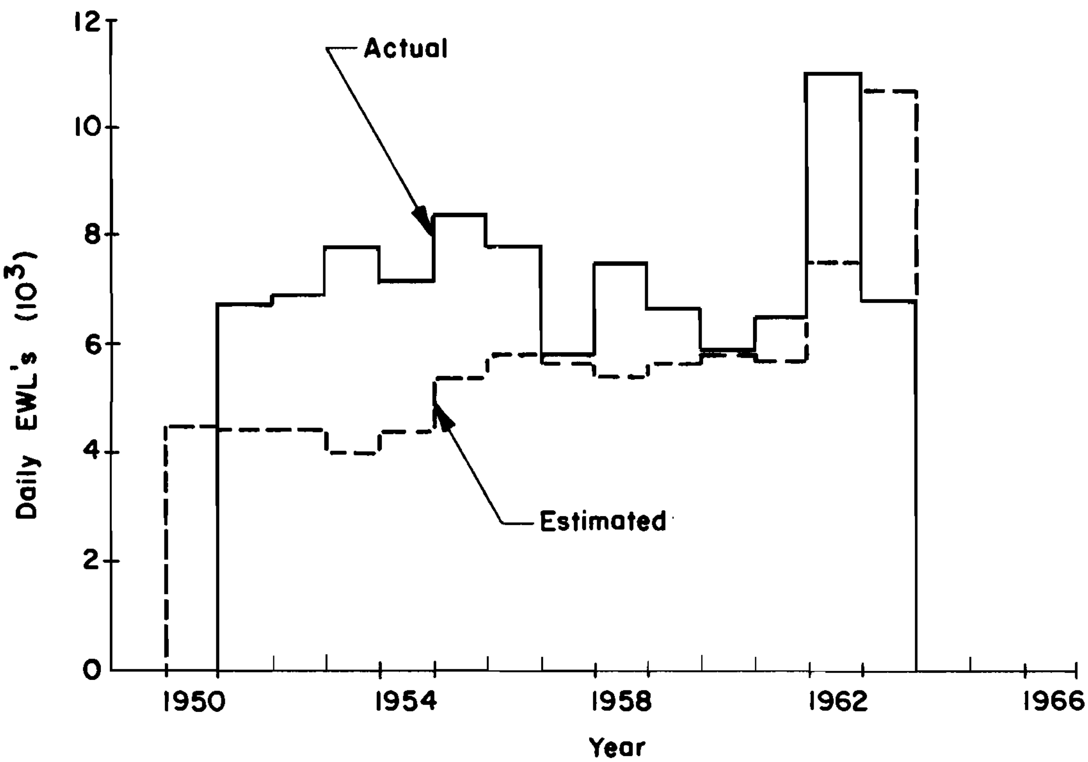


Fig 5.4. Actual and predicted daily EWL's for Kentucky loadometer station 8 (Ref 16).

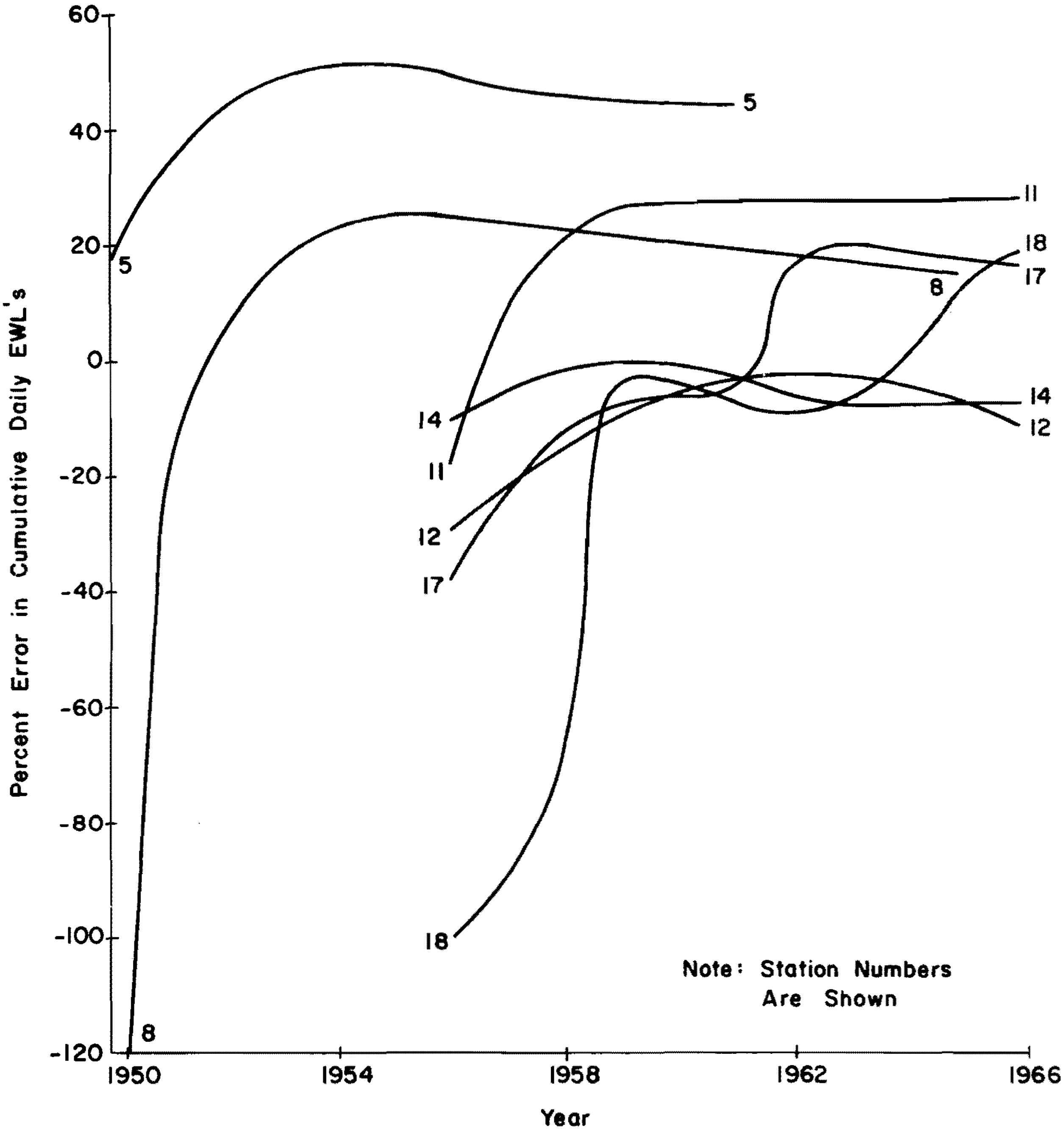


Fig 5.5. Percent error in Kentucky cumulative daily EWL's as a function of time (Ref 16).

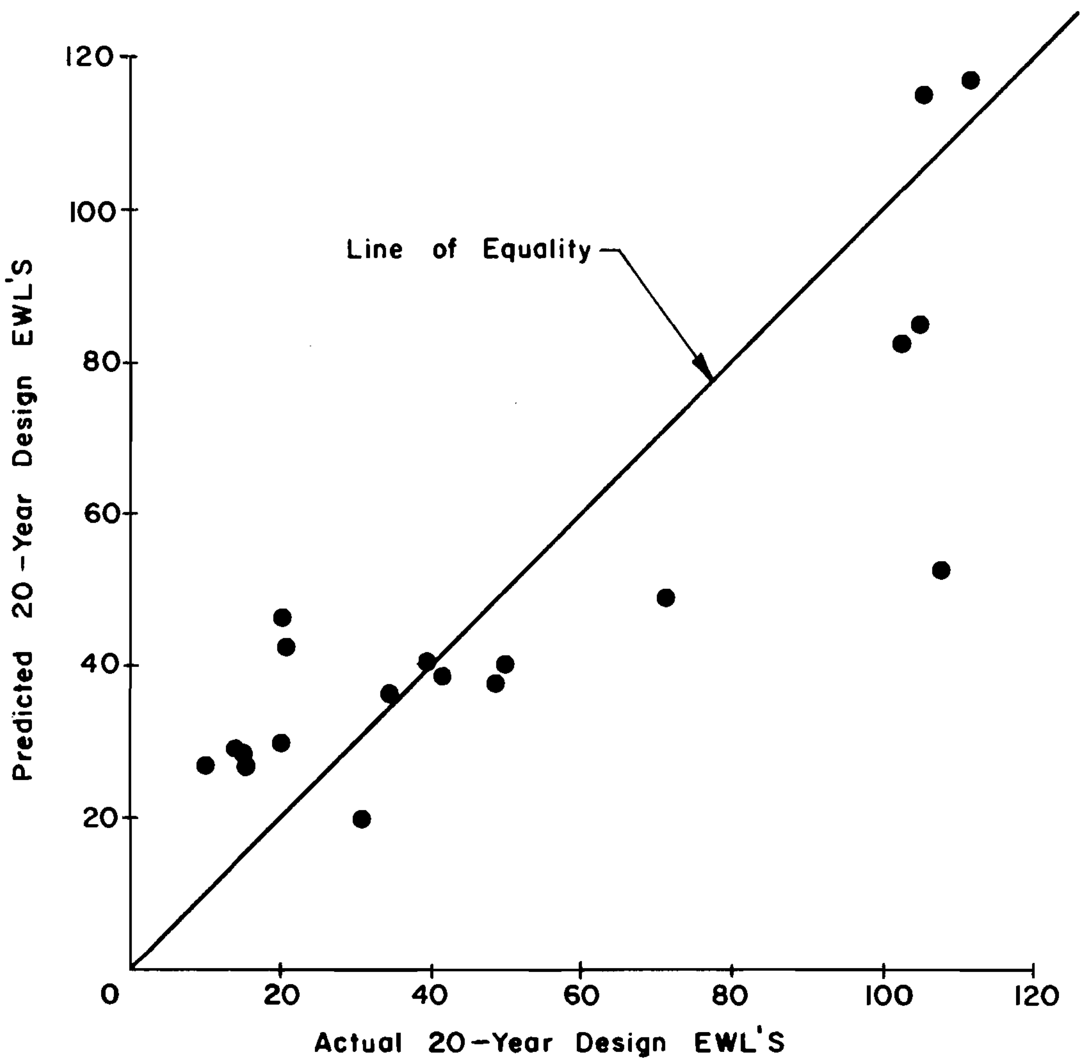


Fig 5.6. Predicted versus actual Kentucky EWL's for 20-year design analysis period (Ref 16).

This magnitude may be compared with those obtained by Shook and Lepp (Ref 92). They developed various multiple regression equations to predict 18-kip equivalent wheel loads, using such factors as number of heavy trucks, average heavy truck gross weight, and single-axle load limit. They developed predictive equations for all states in the United States. The residual variance of log n ranged from 0.0340 to 0.0025 with an average of about 0.0100.

Another significant factor which could cause serious variability in the forecasting is a change in load distribution over the analysis period. This has been measured by the Utah Highway Department and the concept of a Load Distribution Factor (LDF) has been developed (Ref 58). The LDF represents the 18-kip loading equivalency per truck for the highways and trucks for which it was computed. A linear increase of LDF with time was found for heavy and light trucks in Utah. This type of increase would cause significantly higher loadings than originally predicted.

Converting the Distribution of Axle Weights to 18-Kip Equivalent Axle Loads. Texas uses the equivalency factors derived from the AASHO Road Test. These equivalency factors are dependent upon the following:

(1) axle load magnitude,
(2) single or tandem axle,
(3) structural number of pavement,
(4) terminal serviceability level, and
(5) type of pavement (flexible or rigid).

NCHRP Report 1-11 (Ref 65) points out that substantial errors may occur in calculating the total 18-kip equivalencies if the correct equivalency factor is not used. For example, if wide ranges are used for axle weights, the equivalency factor must be averaged across the load groups involved. Another example is that if the structural number of the pavement is assumed to be, say, SN = 3 for a flexible pavement and the pavement design turns out to be SN = 6 , there will be a serious error in the estimated load equivalencies used. Many examples are found in Ref 65, with errors in traffic estimation between -50 percent and +240 percent. The THD uses essentially the recommended Method A, as explained in Ref 65 for calculating the total equivalent loads. Therefore, it is assumed that as long as this procedure is consistently followed by THD and the five factors listed are considered in determining the equivalency factor, little or no error will be induced in this calculation.

Variance of Directional and Lane Distribution. Both directional distribution and lane distribution are believed to have some uncertainty in estimation associated with them. Data are not available to evaluate the possible error in directional distribution. In most cases, it is believed that over a typical analysis period (say 20 years) the loadings will balance themselves out to a 50/50 split. Therefore, directional distribution variance will be neglected.

Lane distribution was found by Alexander and Graves (Ref 3) to have the following standard errors in the state of Georgia. Until more data are available, this variance will also be neglected.

Highway	Mean	Standard Error
4-Lane Rural	93.5%	3.0
4-Lane Urban	88.1	5.8
6-Lane Urban	60.2	8.0

Summary

The procedures used by the Texas Highway Department to predict the number of equivalent 18-kip single-axle load applications has been presented. Estimates were also made for prediction of the various uncertainties and variations associated with the forecasting procedure. In most cases, estimates of variations were based only on engineering judgement as there were no available data. Estimates which are more accurate are certainly needed, so that the overall variation of predicting 18-kip equivalent load applications may be better quantified.

CHAPTER 6. APPLICATION TO TEXAS FLEXIBLE PAVEMENT SYSTEM

The general concept of pavement system reliability has been presented, followed by the development of the basic probabilistic pavement design theory. Data illustrating the stochastic nature of many design parameters has also been summarized. A basic objective of this study is to modify the original deterministic FPS program so that the stochastic nature of the design parameters and lack-of-fit errors of the design models are considered. The designer must input the standard deviations and means of several of the design parameters and the lack-of-fit error of the design models. Thus it will be possible to design for a specified level of reliability.

This chapter first describes the development and verification of the variance models. These results are then applied to the FPS system for the new construction mode and for the overlay mode, and finally a significance study of the design factors is reported.

Variance Models

The basic theory for determining the reliability of a pavement if the log N and log n means and distributions are known has been derived in Chapter 3. The next important step is the derivation of variance models to predict variance associated with log N and log n.

The distributions of log N and log n have been assumed to approximately follow a normal distribution. Therefore, they may be completely defined by a mean and a standard deviation. These factors are functionally related to several other random variables:

$$N = f(a_i , D_i , \alpha, Pl) \tag{6.1}$$

It follows that $f(a_i , D_i , \alpha , P_1)$ is a random variable determined by the statistical characteristics of a_i , D_i , α , and Pl . The random variable $f(a_i\ ,\ D_i\ ,\ \alpha\ ,\ Pl)$ is called a multivariate. The log n is also a multivariate.

$$n = f(ADT_d\ ,\ T\ ,\ A\ ,\ \Sigma PE) \tag{6.2}$$

Since the normal distribution is uniquely defined by its mean and standard deviation, we can determine the variance of the multivariate distributions of log N and log n by any of the following methods, as described by Haugen (Ref 38):

(1) Maximum likelihood methods may be extended to functional combinations.

(2) Partial derivative methods yield good approximations to variances of functional combinations.

(3) Moment generating functions may be employed, provided that the moments of the component distributions are known.

The partial derivative method was selected due to the complexity of the equations and because of its relative ease and accuracy in application. The basic expression used in the partial derivative method to determine the variance of a function $g(x_1\ ,\ x_2\ ,\ \ldots\ x_j)$ is as follows (where $x_1\ ,\ x_2\ ,\ \ldots\ x_j$ are independent random variables):

$$V\left[g(x_1\ ,\ x_2\ ,\ \ldots\ x_j\right] \approx \sum_{i=1}^{j} \left(\frac{\partial g}{\partial x_i}\right)^2 s_{x_i}^2 \tag{6.3}$$

For example, let $g = A_0x_1 + A_1x_1^2x_2$. The variance of g may be determined as follows if x_1 and x_2 are independent random variables:

$$s_g^2 \approx \left(\frac{\partial g}{\partial x_1}\right)^2 s_{x_1}^2 + \left(\frac{\partial g}{\partial x_2}\right)^2 s_{x_2}^2$$

$$\approx \left(A_0 + 2A_1x_1x_2\right)^2 s_{x_1}^2 + \left(A_1x_1^2\right)^2 s_{x_2}^2$$

The expression given by Eq 6.3 can be derived using a Taylor's Series expansion about the mean, μ, as follows:

$$g(x) = g(\mu) + g'(\mu)[x - \mu] + \frac{1}{2}g''(\mu)[x - \mu]^2 + \ldots \text{ (higher order terms)}$$

The expected value of $g(x)$ is

$$E[g(x)] = g(\mu) + g'(\mu)[\mu - \mu] + \frac{1}{2}g''(\mu)\sigma^2 + \ldots.$$

$$E[g(x)] \approx g(\mu) + \frac{1}{2}g''(\mu)\sigma^2$$

where σ^2 = variance of x. The expected value of $g^2(x)$ is

$$E\left[g^2(x)\right] \approx g^2(\mu) + \frac{1}{2}\left[g^2(\mu)\right]''\sigma^2$$

$$\approx g^2(\mu) + \frac{1}{2}\left[2g(\mu)g'(\mu)\right]'\sigma^2$$

$$\approx g^2(\mu) + \frac{1}{2}\left[2g'(\mu)g'(\mu) + 2g(\mu)g''(\mu)\right]\sigma^2$$

The variance of $g(x)$ can now be derived.

$$V[g(x)] = E[g^2(x)] - \left[E[g(x)]\right]^2$$

Now substituting the appropriate values the variance can be evaluated.

$$V\left[g(x)\right] \approx g^2(\mu) + \left[g(\mu)^2 + g(\mu)g''(\mu)\right]\sigma^2$$

$$- \left[g^2(\mu) + g(\mu)g''(\mu)\sigma^2 + \frac{1}{4}g''(\mu)^2\sigma^4\right]$$

$$\approx g'(\mu)^2\sigma^2 - \frac{1}{4}g''(\mu)^2\sigma^4 \qquad (6.4)$$

The expression given in Eq 6.4 represents a second order approximation of the variance of a function. An expression could also be derived for $g(x)$ containing more than one variable. Equation 6.4 can be simplified further by neglecting all moments greater than second order without significant loss of accuracy.

$$V[g(x)] \approx g'(\mu)^2 \sigma^2 \tag{6.5}$$

This expression is the same as Eq 6.3. The accuracy of this equation has been verified by simulation and theory for coefficients of variation below about 0.5 by Haugen (Ref 38) and by Kher and Darter (Ref 53) for quite complex equations.

The expression given in Eqs 6.3 and 6.5 will be used to derive variance equations for the specific models used in FPS. An accuracy verification will be made by use of simulation to establish confidence in the results.

(1) Performance Equation. The performance equation was given as Eq 4.4, which is repeated below:

$$\log N = \log \left(\sqrt{5 - P2} - \sqrt{5 - P1}\right) + \log \alpha - 2 \log SCI$$

$$- \log 53.6 + 6.0$$

The random variables included are $P1$, α , and SCI, which is a function of a_i and D_i . The variance of log N can now be determined:

$$s^2_{\log N} \approx \left(\frac{\partial \log N}{\partial P1}\right)^2 s^2_{P1} + \left(\frac{\partial \log N}{\partial \alpha}\right)^2 s^2_{\alpha} + \left(\frac{\partial \log N}{\partial SCI}\right)^2 s^2_{SCI} + s^2_{lof}$$

$$\approx \frac{0.0471}{[\sqrt{5 - P2} - \sqrt{5 - P1}]^2} \left(\frac{s^2_{P1}}{5 - P1}\right) + \frac{0.189 s^2_{\alpha}}{\bar{\alpha}^2}$$

$$+ \frac{0.755\, s^2_{SCI}}{\overline{SCI}^2} + s^2_{lof} \tag{6.6}$$

where

$s^2_{\log N}$ = variance associated with log N (or allowable 18-kip equivalent single-axle load applications),

s^2_{P1} = variance of initial serviceability index of pavement,

s^2_{α} = variance of the temperature parameter, α ,

s^2_{SCI} = variance of the SCI of the pavement/subgrade, and

s^2_{lof} = variance associated with the lack-of-fit of the performance equation.

Approximate estimates of these variances were presented in Chapter 4, with the exception of SCI , which will now be derived.

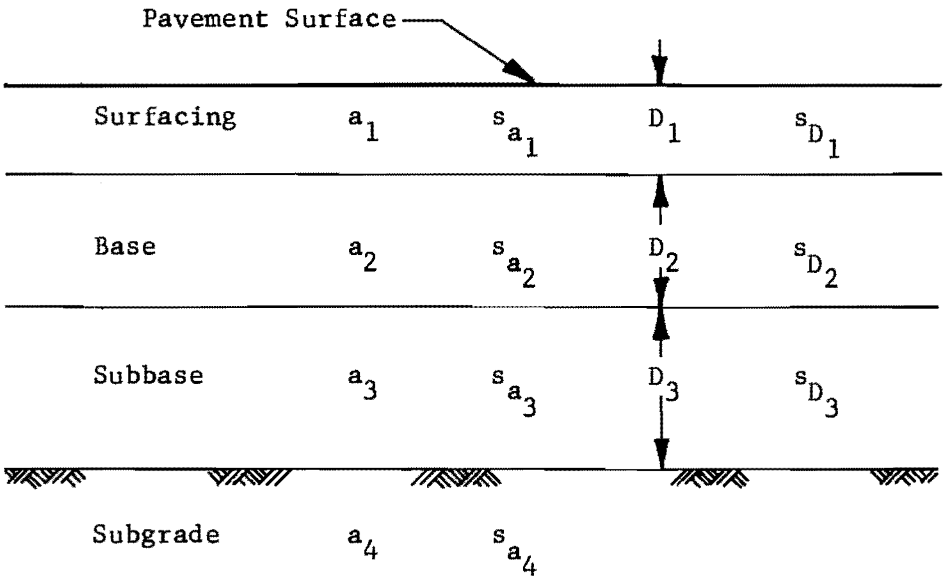
(2) Deflection Equation. The deflection model was given as Eq 4.2. The SCI is a function of a_i and D_i for each pavement layer and subgrade. The variance of SCI due to random variations of pavement and subgrade coefficients and pavement layer thickness was derived for a three-layer pavement as shown in Fig 6.1.

$$s^2_{SCI} \approx \left(\frac{\partial SCI}{\partial a_1}\right)^2 s^2_{a_1} + \left(\frac{\partial SCI}{\partial a_2}\right)^2 s^2_{a_2} + \left(\frac{\partial SCI}{\partial a_3}\right)^2 s^2_{a_3} + \left(\frac{\partial SCI}{\partial a_4}\right)^2 s^2_{a_4}$$

$$+ \left(\frac{\partial SCI}{\partial D_1}\right)^2 s^2_{D_1} + \left(\frac{\partial SCI}{\partial D_2}\right)^2 s^2_{D_2} + \left(\frac{\partial SCI}{\partial D_3}\right)^2 s^2_{D_3} \qquad (6.7)$$

The variance model resulting from Eq 6.7 was placed in Appendix 1 due to its complexity and length.

(3) Equivalent 18-kip Load Prediction. The total equivalent 18-kip single-axle loads expected over the analysis period may be calculated by the



Code: a_i = stiffness coefficient

D_i = layer thickness

s = standard deviation

Fig 6.1. Stiffness coefficient and thickness terminology for three-layer pavement structure on subgrade.

following expression, which was derived in Chapter 5 (Eq 5.3).

$$\log n = \log\left[\sum^{k} P_i EF_i\right] + \log ADT_d + \log A + \log T + \log L$$

$$+ \log DD + \log LD$$

The variance of log n may be determined considering $\Sigma P_i EF_i$, ADT_d , A , and T as independent random variables.

$$s^2_{\log n} = s^2_{\log\ [\ \Sigma PEF]} + 0.189 CV^2_{ADT_d} + 0.189 CV^2_A$$

$$+ 0.189 CV^2_T \tag{6.8}$$

where

CV = coefficient of variation

Estimates for each of these variances were given in Chapter 5 and are discussed further in this chapter.

Verification of Models

The development of variance models given by Eqs 6.6, 6.7, and 6.8 is a first attempt to use probabilistic theory to determine statistical variations in load carrying capacity and traffic forecasting for flexible pavement design. To gain confidence in this approach and to assist in verifying the models, Monte Carlo simulation techniques were used (Ref 38). Essentially this involves determining the variance of log N and log n by Eqs 6.6, 6.7, and 6.8 and comparing the results to those obtained by simulation using the original equations for log N and log n for various pavements and traffic conditions.

The manner in which this was accomplished was to select ten representative pavement/subgrade-traffic situations, using design parameters that were representative of actual conditions in Texas. A relatively high and a relatively low value of each of the design parameters, along with a corresponding standard deviation, were selected from data collected in Chapters 4 and 5 to represent a reasonable range for each parameter. These values are shown in Table 6.1. The ten representative pavement/subgrade-traffic situations were then selected by choosing randomly either a high or low value from Table 6.1 for each design parameter for the ten situations which are summarized in Table 6.2.

Each of the ten situations was then analyzed separately. For each situation the variance of log N and log n was calculated using the variance models. The variance of log N was then determined using simulation by selecting each design parameter (a_i , D_i , α , Pl , and lack-of-fit) from normal distributions having means and standard deviations shown in Table 6.1. The log N was calculated each time for 1000 trials. The variance of log N was then calculated using the array of 1000 values of log N. This same procedure was repeated for log n by selecting random values of the design parameters (ΣPE , A , T , ADT_d) from normal distributions and calculating log n 1000 times. The variance was then calculated from the 1000 values of log n.

This procedure was repeated for each of the ten pavement/subgrade-traffic situations using the computer to carry out the calculations. The results obtained from the variance models (which will be called estimated variance) and the simulated variance for log n and log N are tabulated in Table 6.3. The percent difference was determined by the following method:

$$\text{Percent Difference} = \frac{\text{Estimated Variance - Simulated Variance}}{\text{Simulated Variance}} \times 100$$

The predicted variance of log N shows an average of -7.3 percent less than the simulated variance. The predicted variance of log n shows an average of -2.2 percent less than the simulated variance. These results appear to be reasonably close considering that there is some bias in any computerized random sampling. The standard deviation, which is the square root of the variance, is actually used in design and the percent difference for the standard deviations would be about one-half that for variance. It is therefore concluded that the variance models accurately predict the variance of log N and log n due to statistical variations in the design parameters they depend upon.

TABLE 6.1. RANGES OF DESIGN PARAMETERS AND ASSOCIATED STANDARD DEVIATIONS USED IN VERIFICATION OF VARIANCE MODELS

Parameter	Lower Mean Value	Standard* Deviation	Higher Mean Value	Standard* Deviation
a_1	0.80	10%	1.20	10%
a_2	0.60	0.08	0.85	0.16
a_3	0.50	0.06	0.60	0.085
a_4	0.23	0.01	0.29	0.03
D_1	1.00	10%	4.00	10%
D_2	4.00	10%	10.00	10%
D_3	6.00	10%	12.00	10%
α	19.00	4.24	33.00	4.24
P1	3.90	0.36	4.50	0.36
LOF	0.00	0.284	0.00	0.284
ΣPE	0.10	0.151	0.50	0.151
A	2.50	15%	3.00	15%
T	0.05	15%	0.20	15%
ADT_d	1,000	15%	10,000	20%

*Either a standard deviation or coefficient of variation is given.

TABLE 6.2. SUMMARY OF DESIGN PARAMETERS USED FOR THE TEN REPRESENTATIVE PAVEMENT/SUBGRADE-TRAFFIC SITUATIONS*

Design Parameter	Design Situation 1	2	3	4	5	6	7	8	9	10
a_1	1.20	0.80	0.80	1.20	0.80	0.80	0.80	0.80	0.80	1.20
a_2	0.60	0.60	0.85	0.85	0.60	0.85	0.60	0.85	0.85	0.60
a_3	0.50	0.50	0.60	0.60	0.60	0.50	0.60	0.60	0.60	0.60
a_4	0.29	0.23	0.29	0.23	0.29	0.29	0.23	0.23	0.23	0.23
D_1	4	4	1	1	1	4	4	1	1	1
D_2	10	10	4	4	10	10	10	10	4	10
D_3	6	6	12	6	12	12	6	6	6	6
α	33	19	33	33	19	33	33	33	33	19
P1	3.9	4.5	4.5	4.5	3.9	3.9	4.5	3.9	3.9	3.9
ΣPE	0.1	0.5	0.5	0.5	0.5	0.5	0.1	0.1	0.1	0.1
A	3.0	3.0	2.5	3.0	2.5	2.5	2.5	2.5	3.0	2.5
T	0.20	0.05	0.05	0.05	0.05	0.05	0.20	0.05	0.20	0.20
ADT_d	1,000	1,000	1,000	1,000	10,000	1,000	1,000	10,000	1,000	10,000

* The corresponding standard deviations were determined using Table 6.1.

TABLE 6.3. RESULTS FROM ACCURACY VERIFICATION OF VARIANCE MODELS (Eqs 6.6, 6.7, 6.8) LOG N AND LOG n FOR TEN DESIGN SITUATIONS

Design Situation	Variance of Log N			Variance of Log n		
	Simulated	Estimated	Percent Difference	Simulated	Estimated	Percent Difference
1	0.3001	0.2992	-0.3	0.0360	0.0355	-1.3
2	0.2365	0.2212	-6.5	0.0361	0.0355	-1.5
3	0.3457	0.3272	-5.3	0.0353	0.0355	+0.7
4	0.2397	0.2266	-5.5	0.0361	0.0355	-1.5
5	0.3330	0.3008	-9.7	0.0402	0.0388	-3.5
6	0.2987	0.2634	-11.8	0.0374	0.0355	-4.9
7	0.2315	0.2156	-6.8	0.0361	0.0355	-1.5
8	0.2735	0.2455	-10.3	0.0357	0.0355	-0.4
9	0.3775	0.3630	-3.8	0.0407	0.0388	-4.6
10	0.2940	0.2560	-12.9	0.0402	0.0388	-3.5
Average			-7.3			-2.2

FPS Application - New Construction Mode

The basic theory for application of probabilistic concepts to pavement design was derived in Chapter 3. The definition of reliability was given and the mathematical equations necessary were developed to predict pavement reliability for a design system based upon the serviceability concept. Basically the reliability was defined by Eq 3.1 as

$$R = P[N > n] .$$

Each of these parameters N and n was found to be a multivariate (as they depend upon several variables), to be approximately log normally distributed, and to exhibit significant variation.

The magnitude of difference between N and n (N being the larger) and the magnitude of standard errors of N and n are directly related to the resulting reliability. The reliability of design would be 50 percent if the average N and average n were equal and would increase as N becomes larger. An expression may be obtained by rearranging Eq 3.9, that gives the design N for a specified level of reliability:

$$\overline{\log N_R} = \overline{\log n} + Z_R \sqrt{s_{\log N}^{\;2} + s_{\log n}^{\;2}} \qquad (6.9)$$

where

$\overline{\log N_R}$ = average number of 18-kip single-axle equivalent load applications to be used for design at level of reliability R

$\overline{\log n}$ = average traffic forecast of 18-kip single-axle equivalent load applications

Z_R = standardized normal deviate from normal distribution tables with mean zero and variance of one for given level of reliability R

$s_{\log N}^{\;2}$ = variance of log N determined from Eqs 6.6 and 6.7

$s_{\log n}^{\;2}$ = variance of log n determined from Eq 6.8

This concept is illustrated in Figs 6.2a and 6.2b, where typical distributions of log N and of log n are shown to overlap by various amounts. The failure probability is a function of the area of overlap and decreases as the variance of log N decreases and is illustrated by comparing Fig 6.2b and Fig 6.2c. The greater log N , the less the area of overlap and consequently the less the failure probability and the greater the reliability. The various ways in which reliability could be increased were briefly summarized in Chapter 2. The selection of design reliability is discussed in Chapter 8.

A conceptual diagram of the procedure necessary to design using FPS at a specified reliability R is shown in Fig 6.3. The three general sources of variation in the pavement design and performance process are shown with the corresponding FPS inputs which measure these variations. The conceptual procedures for determining reliability are shown across the top of the figure. The N_R is the value of 18-kip equivalent single-axle loads for which the pavement should be designed so that there will be a probability of R that the serviceability index will not fall below the minimum acceptable level throughout each design period within a limited maintenance input.

The equations and procedures that have been discussed were incorporated into the FPS program. A brief history of the development of FPS was given in Chapter 2. Basically, the FPS-7 program, which underwent trial implementation in 1970, was modified during 1971 to include some of the stochastic concepts described in this study. The new, modified program was named FPS-11 and since late 1971 has been undergoing further implementation in the Texas Highway Department. The FPS-11 version contains the stochastic inputs described in this study with the exception of the following:

(1) Traffic load forecasting variations were not considered. The 18-kip equivalent single-axle load applications were considered only as a deterministic design parameter.

(2) The variation of SCI along a pavement was considered the same for every pavement structure by using an average coefficient of variation of 34 percent. This value was an average obtained from many in-service pavements. The prediction of SCI variation from variations of pavement layer and subgrade stiffness coefficients and thicknesses was not possible.

Details of FPS-11 program documentation were given by Darter, McCullough and Brown (Ref 15) and by Orellana (Ref 74) and are also essentially documented in this study. The two factors mentioned above are believed to be important

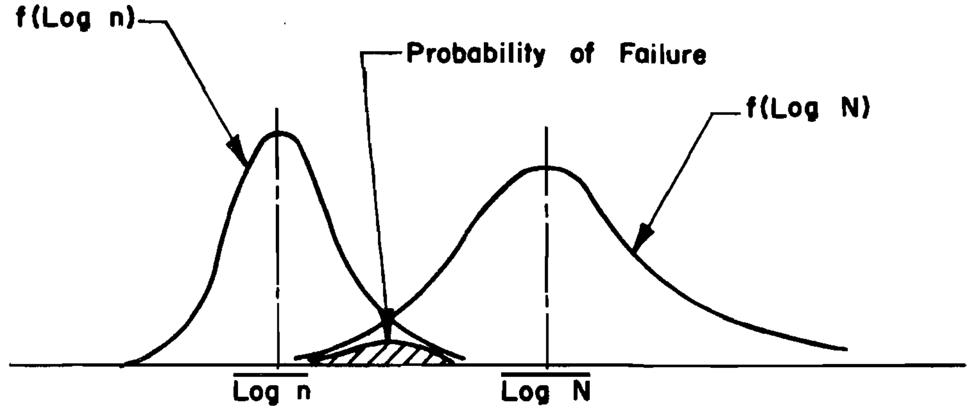


Fig 6.2(a). Illustration of overlap of distributions of log N and log n.

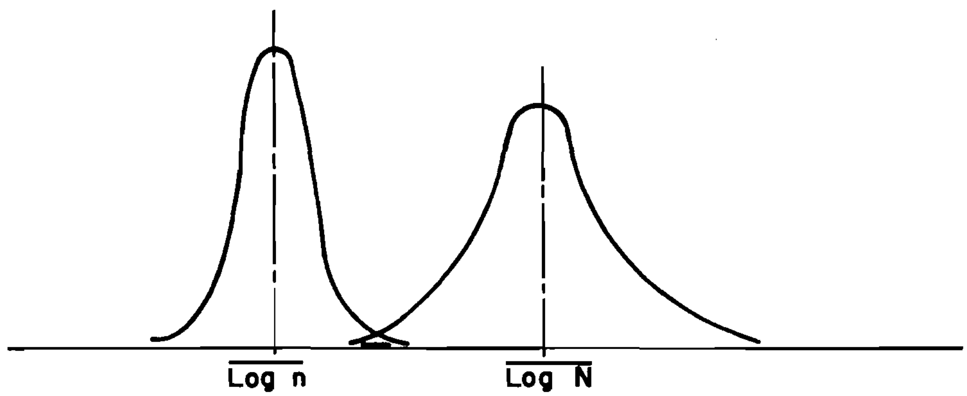


Fig 6.2(b). Reduction of probability of failure by decreasing variance of log n and log N.

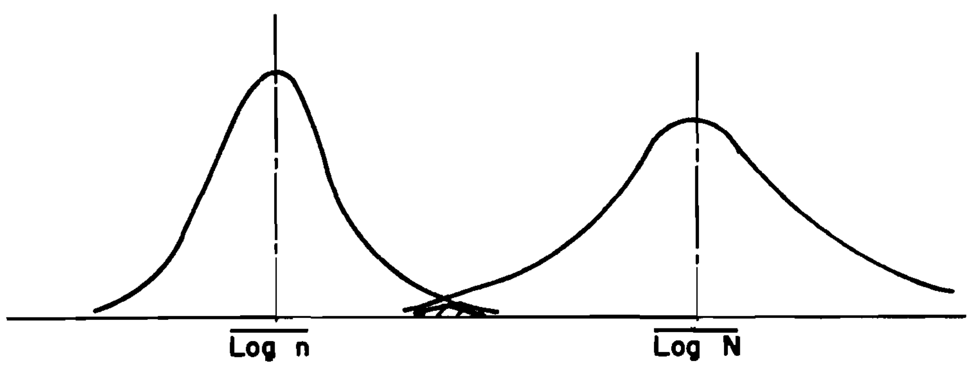


Fig 6.2(c). Reduction of probability of failure by increasing $\overline{\log N}$.

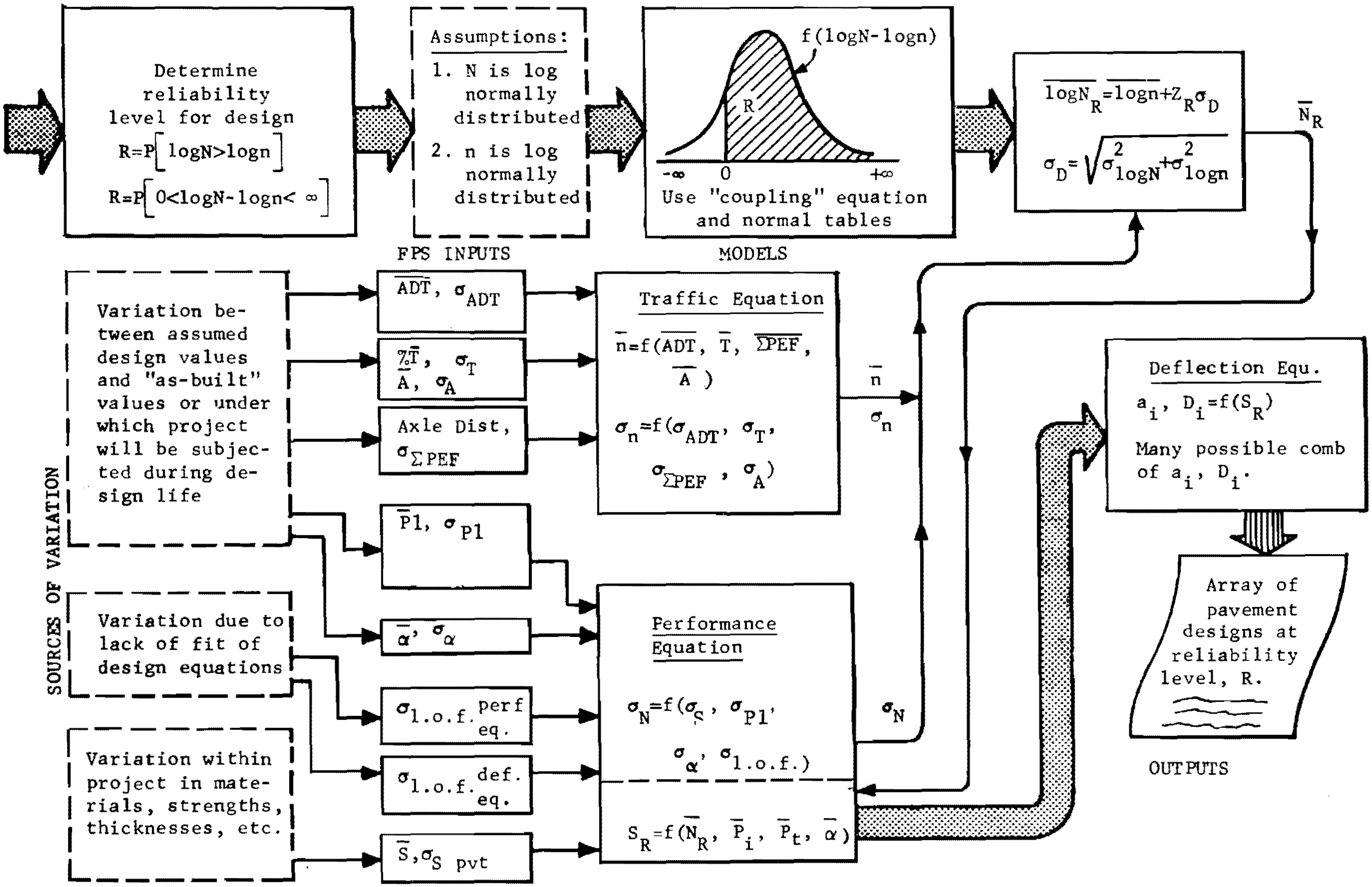


Fig 6.3. FPS pavement design procedure for desired reliability, R .

and to improve the design system and therefore have been included in this study. A new version of FPS which contains these inputs was developed and named FPS-13 (CFHR). Input guides and the computer program are included in Appendix 3. A detailed example problem for the new construction mode is given in Chapter 7.

FPS Application - Overlay Mode

The basic difference between the stochastic input for the overlay mode and the new construction mode is in the method of determining the pavement SCI and its variance. The SCI variance may be calculated from measured SCI data of an existing pavement that is to be overlayed, but must be calculated from pavement layer and subgrade stiffness coefficient variances for new construction. Another difference is that use may be made of serviceability and deflection data of the existing pavement to be overlayed to "adjust" the performance model to specific project conditions, which is now described.

The overlay subsystem has been through trial implementation in several districts and some difficulty has been experienced. It is believed that the lack-of-fit of the performance model may be quite large for some pavements (materials/environment/traffic) and that a major portion of this error can be removed by adjusting the model constant. The performance model was given in Eq 4.4. The minimum data which would be required to "adjust" this model to a specific project are as follows:

(1) actual traffic loadings since time of opening to traffic in 18-kip single-axle equivalencies,

(2) initial serviceability index for 0.2-mile sections along project,

(3) initial SCI measurement for the same 0.2-mile sections along project,

(4) measurement of temperatures through the analysis period so that α can be computed for the project location,

(5) current measurement of serviceability along project for the same 0.2-mile sections measured initially, and

(6) current SCI of the same 0.2-mile sections measured initially.

Since several of these factors were not measured in the past (such as initial conditions and temperatures) the following assumptions have been made:

(1) Use highway department estimate of equivalencies for period, obtained from measurements of past traffic conditions.

(2) Assume an average initial serviceability for all sections along the project.

(3) Assume that the final SCI is the average throughout the analysis period.

(4) Use the average district value of α for the specific project since the error in predicting α will be minimized in the adjustment process.

(5) Measure the serviceability at 0.2-mile lengths with Mays Meter or Surface Dynamics Profilometer.

(6) Measure SCI wthin 0.2-mile sections with the Dynaflect. Two or three replicates within each section should be adequate. These data must be collected during weather conditions which represent average yearly conditions so as not to bias the data.

Using these data the regression coefficient B of the performance model, Eq 4.4 may be "adjusted" so that the average predicted log N as given by Eq 4.4 will be equal to the actual log N estimated to have passed over the pavement:

$$\log B = \frac{\sum_{i=1}^{i=C} \left[\log \left(\frac{Q_i \alpha}{SCI_i^2} \right) - \log N \right]}{C} \tag{6.10}$$

where

Q_i , α , SCI_i = same as previously defined, but calculated separately for each i^{th} 0.2-mile section

log N = log of actual 18-kip equivalent load applications

C = number of 0.2-mile sections

A detailed example of this calculation is given in Chapter 7. A summary of results found for six projects is given in Table 6.4. The B determined from the projects varied from 31 to 1055. However, further examination of projects 1 and 2 shows that the pavement structure is composed of untreated aggregates with a relatively thin asphalt concrete wearing surface. The B values for these projects are 31 and 126. The 53.6 value determined from the AASHO Road Test, which also had a pavement of untreated base and subbase is within this range. The other four projects contained either asphalt-, cement-, or lime-treated materials for which the SCI was relatively small. The B ranged from 264 to 1055 for these projects. This large value of B suggests

TABLE 6.4. SUMMARY OF RESULTS OBTAINED FROM SIX IN-SERVICE HIGHWAYS ($\overline{P1} = 4.2$)

Project	County and Highway	Pavement Structure	Calculated B	$\overline{P2}$	$\overline{SCI}$	Standard Deviation log N	Number 0.2-Mile Sections
1	Bowie US 59	1" Asphalt concrete 12" Iron ore 12" Sand clay	31	3.4	0.51	0.12	8
2	Bexar IH 35	2" Asphalt concrete 12-18" Variable flexible base	126	3.4	0.29	0.24	22
3	Angelina US 69	10" Asphalt concrete 6" Lime-treated subgrade	1055	3.6	0.13	0.24	24
4	Polk US 59	6" Asphalt concrete 6" Cement-treated base 6" Road-treated base 6" Lime-treated subgrade	264	3.9	0.16	0.29	26
5	Harrison US 59	1.5" Asphalt concrete 8" Cement-treated iron ore 6" Sand clay	860	2.9	0.17	0.57	12
6	Polk US 59	6" Asphalt concrete 6" Cement-treated base 6" Road-treated base 6" Lime-treated subgrade	375	3.5	0.17	0.23	52

a rather large lack-of-fit error associated with the performance equation for pavement structures containing treated bases and subbases, as was pointed out in Chapter 4.

The data and analysis described above may be used to obtain estimates of variance of SCI between 0.2-mile sections and lack-of-fit error within the project. These specific variations are discussed in the next section.

The FPS-11 overlay mode program was modified to include the procedures described above and is included in the FPS-13 (CFHR) program. The program also contains the traffic variance models and determines the required design load applications using Eq 6.9 which are similar to the new construction mode but with different estimates of variation, as discussed in the next section.

Variance Characterization Recommendations

The designer must input the means and the standard deviations of the design factors as well as the specified level of reliability to use the new program, which has been named FPS-13 (CFHR). The magnitudes of these variations were examined in Chapters 4 and 5. These results are now summarized and recommendations are given for use of the new program. The design reliability level is quantified in Chapter 8. The variances are described separately for the new construction mode and for the overlay mode.

New Construction Mode. A summary of recommended variations is given in Table 6.5. These recommendations will be briefly discussed.

The variation of the stiffness coefficients a_i for a specific project design can be measured for similar materials in the general area of the project between 0.2-mile sections and the corresponding variations can be used. If these measurements are not possible the standard deviations for several materials can be estimated from Fig 4.9 if the mean stiffness is known.

The variation of the layer thicknesses D_i is essentially contained in the variations determined for the stiffness coefficients because the pavement is usually not cored at every location at which deflection is measured. An average thickness of pavement is usually assumed for use in the stiffness coefficient program. Therefore, if thickness is not considered in calculating the standard deviation of the stiffness coefficient, such as for the data derived in Fig 4.9, it is assumed to be contained within a_i variation. If the

TABLE 6.5. SUMMARY OF RECOMMENDED VARIATIONS FOR DESIGN FACTORS AND MODELS FOR NEW CONSTRUCTION MODE

Design Parameter	Recommended Design Variations
a_i	Measure variation of similar material in project area or use Fig 4.9 for estimate for material.
D_i	This variance is contained in variance of a_i in Fig 4.9. If included, use 10 percent coefficient of variation.
α	Variance = 18.0 for all projects.
Pl	Sum variance within project and between design and actual: $(0.3)^2 + (0.2)^2 = 0.13$.
Performance Model	Use variance = 0.0812.
Deflection Model	Use coefficient of variation of 30 percent of SCI for new construction and 38 percent for overlays. Add to variance in SCI found within a project.
ADT_d	10 to 20 percent coefficient of variation for existing highways, 15 to 30 percent coefficient of variation for new locations.
A	5 to 15 percent coefficient of variation.
T	10 to 15 percent coefficient of variation.
$\log(\Sigma PE)$	Variance = 0.0229.

variation of thickness is to be considered, a coefficient of variation of about 10 percent is recommended.

The performance model lack-of-fit was estimated from the error in predicting the life of the AASHO Road Test sections. A variance of log N of 0.0812 should be considered a minimum value and revised when adequate data have been collected from actual in-service projects.

The failure of the deflection model to accurately predict the SCI of the pavement structure was determined to be approximately equal to a coefficient of variation of 30 percent of the average SCI for initial construction and 38 percent for overlays. This variance should be added to the variation of SCI within a project or between 0.2-mile sections. This estimate should also be considered as a minimum value because it was derived from test section data that were limited in material types, thicknesses, subgrades, etc.

Overlay Mode. A summary of recommended variations is given in Table 6.6. Briefly, these recommendations are as follows.

The variation in SCI may be measured in situ for the existing pavement structure before overlay. If two or more SCI measurements are taken within each 0.2-mile section the component of variation may be determined between sections. This is done internally within the program and is not an input. The designer needs only to input the SCI values measured for each 0.2-mile section.

The variance of the performance model is calculated internally in the program. There is still lack-of-fit variance associated with the performance model and it is determined from the differences in the predicted log N between sections within a given project.

The recommended lack-of-fit coefficient of variation associated with the prediction of the SCI after the overlay has been placed is 38 percent. This variance of SCI should be added to the between section component determined above.

All variations associated with the traffic forecasting are assumed to be the same as for the new construction mode.

Significance Study of Design Factors

The probabilistic design approach provides an excellent basis for conducting a significance study of the design paramters associated with the

TABLE 6.6. SUMMARY OF RECOMMENDED VARIATIONS FOR DESIGN FACTORS AND MODELS FOR OVERLAY MODE

Parameter	Recommended Variance
SCI	Measure existing pavement and determine component of variance between 0.2-mile sections. This is done internally in program.
α	Variance assumed negligible.
P1	Sum variance within project and between design and actual: $(0.3)^2 + (0.2)^2 = 0.13$.
Performance Model	Use variance between 0.2-mile sections. Determined internally in program.
Deflection Model	Use coefficient of variation of 38 percent of SCI. Add to variance in SCI found between sections.
ADT_d	Use same as Table 6.5.
A	Use same as Table 6.5.
T	Use same as Table 6.5.
$\log(\Sigma PE)$	Use same as Table 6.5.

performance and traffic models. The ten design situations that were used to check the accuracy of the variance models can be used in the study. The manner in which the ten representative pavement/subgrade-traffic situations were obtained was explained and the levels of design factors and associated variances were given in Tables 6.1 and 6.2.

The relative effect of the natural statistical variation of each of the design parameters on the model response (log n and log N) may be estimated by determining the variance in model response due to variations of the design parameters separately. Each design factor variation was selected from the best data or experience available for in situ or actual variations. The method consists of determining the variance of log N or log n due to the variance of each of the design factors separately and then dividing this by the total variance of log N or log n when all design factors are included. For example, the variance of log N caused by the stiffness coefficient variation of the base course a_2 for design situation number 1 was 0.0798. The total variance of log N caused by all the design factors (a_1 , a_2 , 2_3 , a_4 , D_1 , D_2 , D_3 , Pl , and lack-of-fit) was 0.3772. Therefore, the percent contribution of a_2 variation to the performance model variation for this particular pavement/subgrade situation was determined as follows:

$$(0.0709/0.3772)100 = 21.2 \text{ percent}$$

This relative effect of each of the design factors was determined in the same manner. A summary of results is shown in Table 6.7, where the range and average percent effect of each factor are tabulated over the ten pavement/subgrade-traffic situations. A visual comparison is shown in Fig 6.4, in which the variance of log N and log n have been combined and the average percent of each factor was determined based upon total combined variance. This figure shows that the variance of the allowable load applications, or the performance model, is by far greater than the variance of applied load applications, or the traffic model. The variation associated with the stiffness coefficient of the base material and the lack-of-fit errors of the performance and deflection equations were the most significant. The true magnitude of lack-of-fit error is probably much larger than shown here for all types of pavement materials, environment, and traffic conditions, as was illustrated in Chapter 4.

TABLE 6.7. SUMMARY OF SIGNIFICANCE STUDY RESULTS: PERCENT EFFECT CONTRIBUTED BY EACH PARAMETER TO TOTAL VARIANCE OF PERFORMANCE AND TRAFFIC MODELS (log N and log n) FOR TEN REPRESENTATIVE DESIGN SITUATIONS

Design Parameter	Percent Range	Average Effect, Percent
I. Performance Model - log N:		
a_1	0.1 to 5.4	1.7
a_2	13.4 to 39.5	23.4
a_3	1.2 to 22.6	5.9
a_4	3.0 to 14.9	7.2
D_1	0.1 to 4.0	0.9
D_2	0.3 to 6.7	3.1
D_3	0.3 to 3.5	1.6
α	0.7 to 3.3	1.6
Pl	6.2 to 13.3	10.2
LOF (perf.)	18.7 to 28.4	24.1
LOF (def.)	15.8 to 23.9	20.3
		100.0
II. Traffic Model - log n:		
Σ PE	58.7 to 64.2	62.6
A	10.9 to 11.9	11.6
T	10.9 to 11.9	11.6
ADT_d	11.9 to 19.4	14.2
		100.0

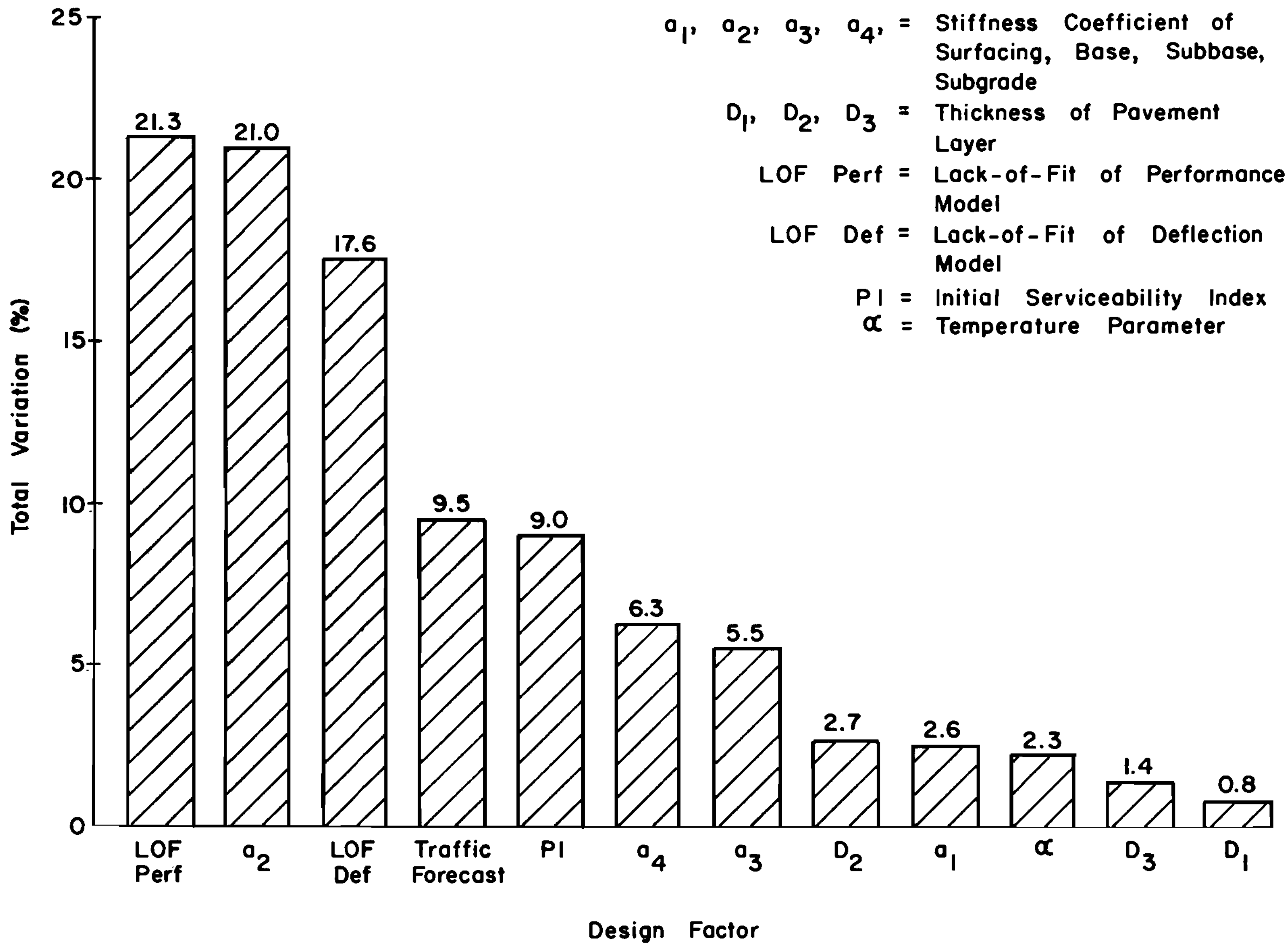


Fig 6.4. Illustration of percent effect of each design factor variation of total variance of log N and log n.

A ranking of the design parameters based upon their relative effect on total project variance is as follows:

(1) lack-of-fit variance of performance model, 21.3 percent,

(2) stiffness of base layer variation, 21.0 percent,

(3) lack-of-fit variance of deflection model, 17.6 percent,

(4) traffic load application forecasting variation, 9.5 percent,

(5) initial serviceability variation, 9.0 percent,

(6) subgrade stiffness variation, 6.3 percent,

(7) subbase stiffness variation, 5.5 percent, and

(8) sum of variation of all other factors, 9.8 percent.

Figure 6.5 shows a comparison of the relative percent effect of the variation of lack-of-fit of the performance and deflection models, the stiffness of the pavement/subgrade, the thicknesses of the pavement layers, serviceability, and the temperature parameter. The pavement/subgrade stiffness and the lack-of-fit error variation are the largest sources of variation.

Other combinations of materials and traffic conditions may give a different breakdown in percent effect of the variation of each parameter. However, this example is representative of many typical pavement/subgrade-traffic situations and points out that the variability of a few design parameters accounts for a large part of the total variation. The effects of this natural variability on life of a pavement are illustrated in Chapter 7. These results show that variations caused by lack-of-fit of the models is very significant. Also, the variability of the stiffness, or modulus, of the pavement/subgrade system is very significant. The traffic forecasting error and initial smoothness variation of the pavement also had significant effect.

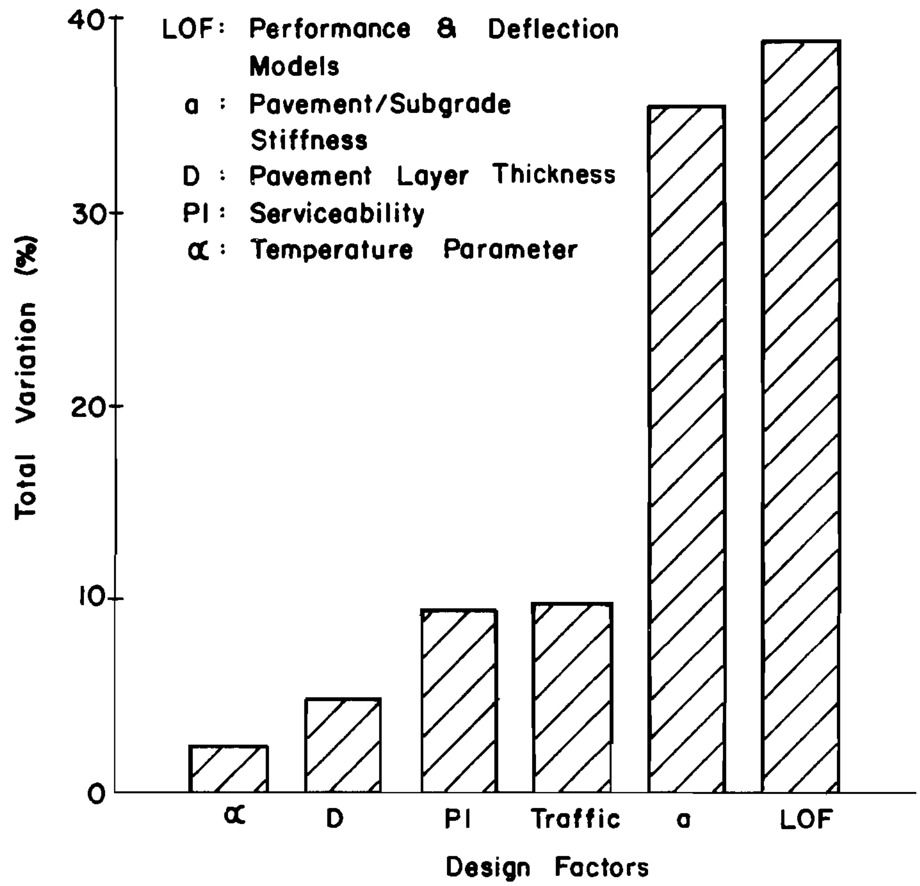


Fig 6.5. Comparison of combined design factors and percent effect of total variance.

CHAPTER 7. PROBABILISTIC DESIGN ILLUSTRATION

A description of the application of probabilistic design theory to the FPS design system has been presented. The illustration and usage of the new probabilistic system through two example designs are the subject of this chapter. The first example is the pavement design for a new urban freeway. The second example is an asphalt concrete overlay design for a primary highway. Illustrations are given to show the effect of variations on the economics of pavement design and other illustrations of the benefits and capabilities of the new system for each example.

Example Design: New Construction

The new construction design mode of FPS may be used for design of any pavement structure for a new location or for completely rebuilding an existing pavement structure. The example selected is the pavement design for an urban freeway through Austin, Texas, named Loop 1, MOPAC. The facility will be six lanes with an estimated initial ADT of nearly 40,000. The one-direction expected equivalent 18-kip single-axle loads are estimated at about seven million for a 20-year design period. The alignment of the highway passes over a subgrade that has some swelling tendency. The effect of swelling subgrade will be neglected in this example problem so that the results of the probabilistic concepts can more easily be illustrated.

The various inputs necessary for the pavement design were estimated and are summarized in Table 7.1, which is an input listing of the FPS-13 (CFHR) program. A description of many of the inputs is given in the Texas Highway Department User's Manual for FPS (Ref 22). There are several inputs, however, which relate specifically to the probabilistic design method and their selection is discussed now.

Traffic Data. The estimates of variation for design ADT, percent trucks, axles per truck, and axle load distribution were made according to recommendations given in Chapter 5 and summarized in Table 6.5, since no better data

TABLE 7.1. ILLUSTRATIVE PROBLEM OF FLEXIBLE PAVEMENT DESIGN FOR URBAN FREEWAY - INPUTS TO FPS PROGRAM

TEXAS HIGHWAY DEPARTMENT
FPS - 13 CFHR
FLEXIBLE PAVEMENT DESIGN

PROB	DIST.	COUNTY	CONT.	SECT.	HIGHWAY	DATE	IPE	PAGE
18	14	TRAVIS	3136	01	LP 1 MOPAC	12/1/72	238	1

COMMENTS ABOUT THIS PROBLEM

EXAMPLE DESIGN PROBLEM USING PROBABILISTIC DESIGN METHOD
LOOP 1 MOPAC, AUSTIN, TEXAS

BASIC DESIGN CRITERIA

LENGTH OF THE ANALYSIS PERIOD (YEARS)	20.0
MINIMUM TIME TO FIRST OVERLAY (YEARS)	6.0
MINIMUM TIME BETWEEN OVERLAYS (YEARS)	6.0
MINIMUM SERVICEABILITY INDEX P2	3.0
DESIGN RELIABILITY LEVEL	E
INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)	7.0

PROGRAM CONTROLS AND CONSTRAINTS

NUMBER OF SUMMARY OUTPUT PAGES DESIRED (8 DESIGNS/PAGE)	3
NUMBER OF MATERIALS	3
MAX FUNDS AVAILABLE PER SQ.YD. FOR INITIAL DESIGN (DOLLARS)	8.00
MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)	36.0
ACCUMULATED MAX DEPTH OF ALL OVERLAYS (INCHES) (EXCLUDING LEVEL-UP)	6.0

TRAFFIC DATA

ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)	39330
ADT AT END OF TWENTY YEARS (VEHICLES/DAY)	64752
ONE-DIRECTION 20.-YEAR ACCUMULATED NO. OF EQUIVALENT 18-KSA	6894000
AVERAGE APPROACH SPEED TO THE OVERLAY ZONE(MPH)	50.0
AVERAGE SPEED THROUGH OVERLAY ZONE (OVERLAY DIRECTION) (MPH)	20.0
AVERAGE SPEED THROUGH OVERLAY ZONE (NON-OVERLAY DIRECTION) (MPH)	50.0
PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)	5.5
PERCENT TRUCKS IN ADT	8.0
DESIGN ADT COEFFICIENT OF VARIATION(PERCENT)	15.0
PERCENT TRUCKS COEFFICIENT OF VARIATION	15.0
AXLES PER TRUCK COEFFICIENT OF VARIATION(PERCENT)	10.0
VARIANCE OF AXLE LOAD/EQUIVALENCY PARAMETER	.0229

ENVIROMENT AND SUBGRADE

DISTRICT TEMPERATURE CONSTANT	31.0
SWELLING PROBABILITY	0.00
POTENTIAL VERTICAL RISE (INCHES)	0.00
SWELLING RATE CONSTANT	0.00
SUBGRADE STIFFNESS COEFFICIENT	.26
DISTRICT TEMPERATURE CONSTANT STANDARD DEVIATION	4.24
SUBGRADE STIFFNESS COEFFICIENT STANDARD DEVIATION	.02

(Continued)

TABLE 7.1. Continued.

TEXAS HIGHWAY DEPARTMENT
FPS - 13 CFHR
FLEXIBLE PAVEMENT DESIGN

PROB	DIST.	COUNTY	CONT.	SECT.	HIGHWAY	DATE	IPE	PAGE
18	14	TRAVIS	3136	01	LP 1 MOPAC	12/1/72	238	2

INPUT DATA CONTINUED

CONSTRUCTION AND MAINTENANCE DATA
****

SERVICEABILITY INDEX OF THE INITIAL STRUCTURE	4.0
SERVICEABILITY INDEX P1 AFTER AN OVERLAY	3.9
MINIMUM OVERLAY THICKNESS (INCHES)	.8
OVERLAY CONSTRUCTION TIME (HOURS/DAY)	7.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)	1.26
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)	75.0
WIDTH OF EACH LANE (FEET)	12.0
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE-MILE)	100.00
INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE-MILE)	10.00
INITIAL STRUCTURE AND OVERLAY SERVICIBILITY INDEX STANDARD DEVIATION	.36

DETOUR DESIGN FOR OVERLAYS
****

TRAFFIC MODEL USED DURING OVERLAYING	3
TOTAL NUMBER OF LANES OF THE FACILITY	6
NUMBER OF OPEN LANES IN RESTRICTED ZONE (OVERLAY DIRECTION)	1
NUMBER OF OPEN LANES IN RESTRICTED ZONE (NON-OVERLAY DIRECTION)	3
DISTANCE TRAFFIC IS SLOWED (OVERLAY DIRECTION) (MILES)	1.00
DISTANCE TRAFFIC IS SLOWED (NON-OVERLAY DIRECTION) (MILES)	0.00
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)	0.00

MODEL LACK OF FIT VARIANCE
*************************/

PERFORMANCE MODEL LACK OF FIT VARIANCE	.0812
DEFLECTION MODEL LACK OF FIT COEFFICIENT OF VARIATION(PERCENT)	
INITIAL STRUCTURE	30.0
OVERLAY	38.0

PAVING MATERIALS INFORMATION
****

LAYER	MATERIALS CODE	NAME	COST PER CY	STR. COEFF.	STD. DEV. STR. COEFF.	MIN. DEPTH	MAX. DEPTH	COEFF. VAR. LAYER THICK.	SALVAGE PCT.
1	A	ACP	15.48	.96	.10	2.00	4.00	10.00	30.00
2	B	BLACK BASE	13.93	.96	.10	2.50	10.00	10.00	40.00
3	C	CRUSHED STONE	4.40	.60	.08	5.00	18.00	10.00	75.00

were available. The combined effect of these sources of error would give a variance of 0.0333 for log n . The 95 percent confidence interval for the mean estimate of 6,894,000 applications would be as follows, assuming log n to be normally distributed:

$$\begin{aligned} \log n &= \log (6{,}894{,}000) \pm 1.64\sqrt{0.0333} \\ &= 6.8385 \pm 0.3000 \end{aligned}$$

or

$$n = 3{,}455{,}000 \text{ to } 13{,}755{,}000$$

There is a chance of 95 in 100 that the mean number of 18-kip equivalent single-axle load applications actually applied to the pavement over the design analysis period will fall within this range.

Pavement/Subgrade. Estimates of variation were made for initial serviceability index, pavement layer and subgrade stiffness coefficients, and layer thicknesses. The initial serviceability index standard deviation of a 0.2-mile design section was estimated using the recommendations given in Table 6.5.

The standard deviations for the pavement materials were obtained using Fig 4.9 as a guide, supplemented by actual measurements. The subgrade average stiffness and its variation were estimated from stiffness coefficients, calculated from deflection data taken on several paved streets near the freeway alignment. The crushed stone subbase mean stiffness coefficient was obtained from in situ measurements on typical crushed stone aggregates of the quality specified in the plans and specifications and existing in pavments in the general area. The variation of the stiffness coefficient was estimated from Fig 4.9. The blackbase specified was essentially of asphalt concrete quality, and the stiffness coefficient was assumed to be similar. The standard deviation of stiffness was obtained from Fig 4.9. The stiffness of asphalt concrete was taken to be the recommended average with a standard deviation of 10 percent of the mean because good quality control was expected.

Design Model Lack-of-Fit. The lack-of-fit variance of the design models was used as recommended in Table 6.5. These estimates for the deflection model and performance model are reasonable and applicable to this design problem.

Calculations. The following calculations show the method for design at R = 99.6 percent:

$$\overline{\log N_R} = \overline{\log}_n + Z_R\sqrt{s^2_{\log n} + s_{\log}{}^2_n}$$

where

$$\overline{\log n} = \log(6{,}894{,}000) = 6.8385$$

$$Z_R = 2.65 \text{ (from normal tables)}$$

$$s^2_{\log n} = \text{Eq } 6.8$$

$$= 0.0229 + 0.189\ [0.15^2 + 0.15^2 + 0.10^2]$$

$$= 0.0333$$

$$s^2_{\log N} = \text{Eq } 6.6$$

$$= \frac{0.0471}{[\sqrt{5.0 - 3.0} - \sqrt{5.0 - 4.2}]^2} \left(\frac{(0.09 + 0.04)}{5.0 - 4.2}\right)$$

$$+ 0.189(18.0)/(31.0)^2 + \frac{0.755\ s^2_{SCI}}{\overline{SCI}^2} + 0.0812$$

$$= 0.3271$$

$$\overline{SCI} = \text{Eq. } 4.2 = 0.030$$

$$s^2_{SCI} = \text{Eq } 6.7 + (\overline{SCI})^2 (0.30)^2$$

$$= 0.000174 + 0.000081 = 0.000255$$

therefore

$$\log N_R = 6.8385 + 2.65\sqrt{0.0333 + 0.3271} = 8.4294$$

The log N_R was then used for design of the pavement structure using the various models in FPS that have been given.

Outputs. The FPS-13 (CFHR) program outputs an array of feasible designs which are sorted by total cost. The program may be run at any desired level of reliability. A summary of the optimum design at six levels of reliability is given in Table 7.2. The selection of design reliability level is discussed in Chapter 8. For this urban freeway example, a design reliability of 99.6 percent was chosen by the design engineer. The reasons for this choice are discussed in Chapter 8.

The optimum design strategy for lowest total cost at $R = 99.9$ percent shows, for example, an initial design life of about nine years. An overlay of 2.3 inches of asphalt concrete is then scheduled and it is predicted to last through the 20-year design life. An early overlay was found more economical because the large traffic volume of later years would cause excessive user delay costs due to overlay operations.

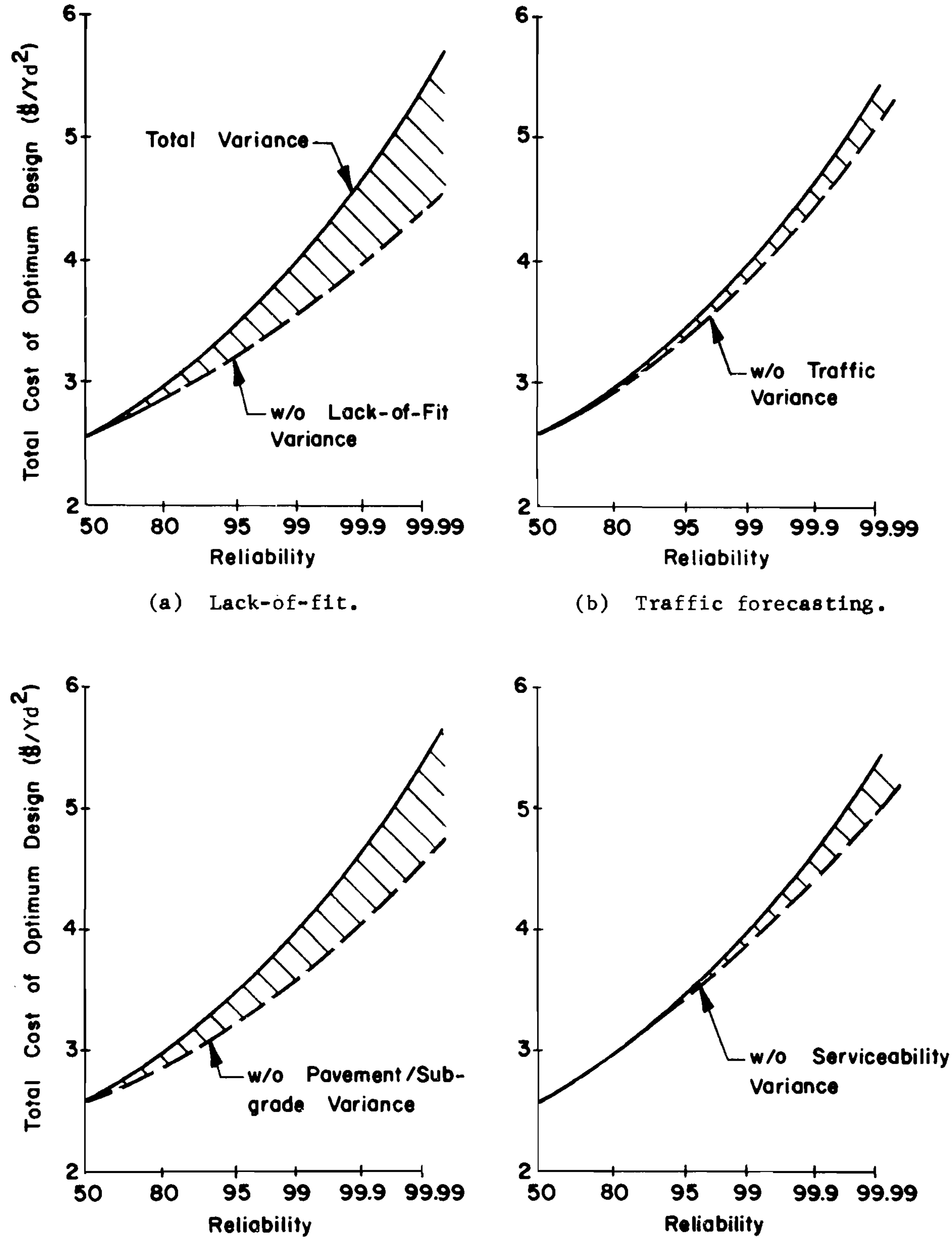
The design reliability level is correct only if all of the variances have been correctly modeled. The level of R represents the maximum possible value. The true R would be somewhat less than this if all variations were known. While this value may seem quite high, it must be realized that this is an expected percentage of pavements reaching minimum serviceability over all projects designed with FPS for this reliability level. This high level of reliability also reflects the designer's concern over the consequences of premature reduction in serviceability level of a high-volume urban freeway.

The effect of reliability level and the magnitude of variation of the design factors are illustrated in Fig 7.1. As the specified level of reliability increases, the corresponding total cost of the optimum pavement design also increases. The upper curve on each plot represents optimum pavement design with all of the variations considered. The lower curve represents the optimum pavement design obtained without considering the various sources of variation shown. These plots give quantitative data about the effect of variations on pavement costs. For example, if there were no lack-of-fit associated with the design models of FPS, the total pavment costs could be reduced about

TABLE 7.2. SUMMARY OF OPTIMUM (TOTAL COST) DESIGNS FOR MOPAC DESIGN PROBLEM AT VARIOUS LEVELS OF RELIABILITY

Design Criteria	Reliability Level, percent					
	50	80	95	99	99.9	99.99
Initial cost	2.66	3.20	3.68	3.73	4.41	5.19
Routine maintenance	.28	.28	.28	.23	.22	.22
Overlay	.00	.00	.00	.27	.54	.50
User-delay	.00	.00	.00	.29	.16	.24
Salvage	-.32	-.39	-.49	-.55	-.69	-.77
Total ($/SY)*	2.62	3.09	3.47	3.97	4.64	5.38
Thickness (inches)						
Asphalt concrete	2.00	2.25	2.00	2.00	2.00	2.00
Black base	2.75	3.25	3.50	3.00	3.50	5.55
Crushed stone	6.00	8.00	12.00	14.00	18.00	18.00
Initial life (years)	21.6	21.4	21.0	10.5	9.2	9.5
Overlay thickness (inches)				1.3	2.3	2.3
Life (years)				20.0	22.0	22.0

* SY = square yard



(a) Lack-of-fit.

(b) Traffic forecasting.

(c) Pavement/subgrade.

(d) Initial serviceability.

Fig 7.1. Total cost versus reliability for specific pavement design problem (Mopac design).

15 percent for the same level of reliability. A reduction of costs could also be achieved if there were fewer variations associated with the pavement/subgrade, traffic estimation, and initial pavement serviceability.

The change in total pavement costs for a change in variance of several of the design factors is illustrated in Figs 7.2 and 7.3. As subgrade variation is increased, for example, the total cost of the optimum pavement design increases at an increasing rate. These examples are given to illustrate the concepts involved and should not be considered as exact predictions. There are many other implications that could be discussed, but those presented illustrate the possibilities of the method.

Example Design: Asphalt Concrete Overlay

The overlay mode of FPS can be used to design an asphalt concrete overlay for an existing pavement. The overlay design mode utilizes most of the usual subsystems in FPS and provides for an overall design strategy over some analysis period. The example problem selected for illustration of probabilistic overlay design is US 59 from north of Sulphur River to south of FM 989. The existing pavement is about 11 years old and has an average present serviceability index of 3.2. The design period is from 1975 to 1995, with an estimated average 18-kip equivalent single-axle load of nearly four million. The pavement is not located in a swelling clay area.

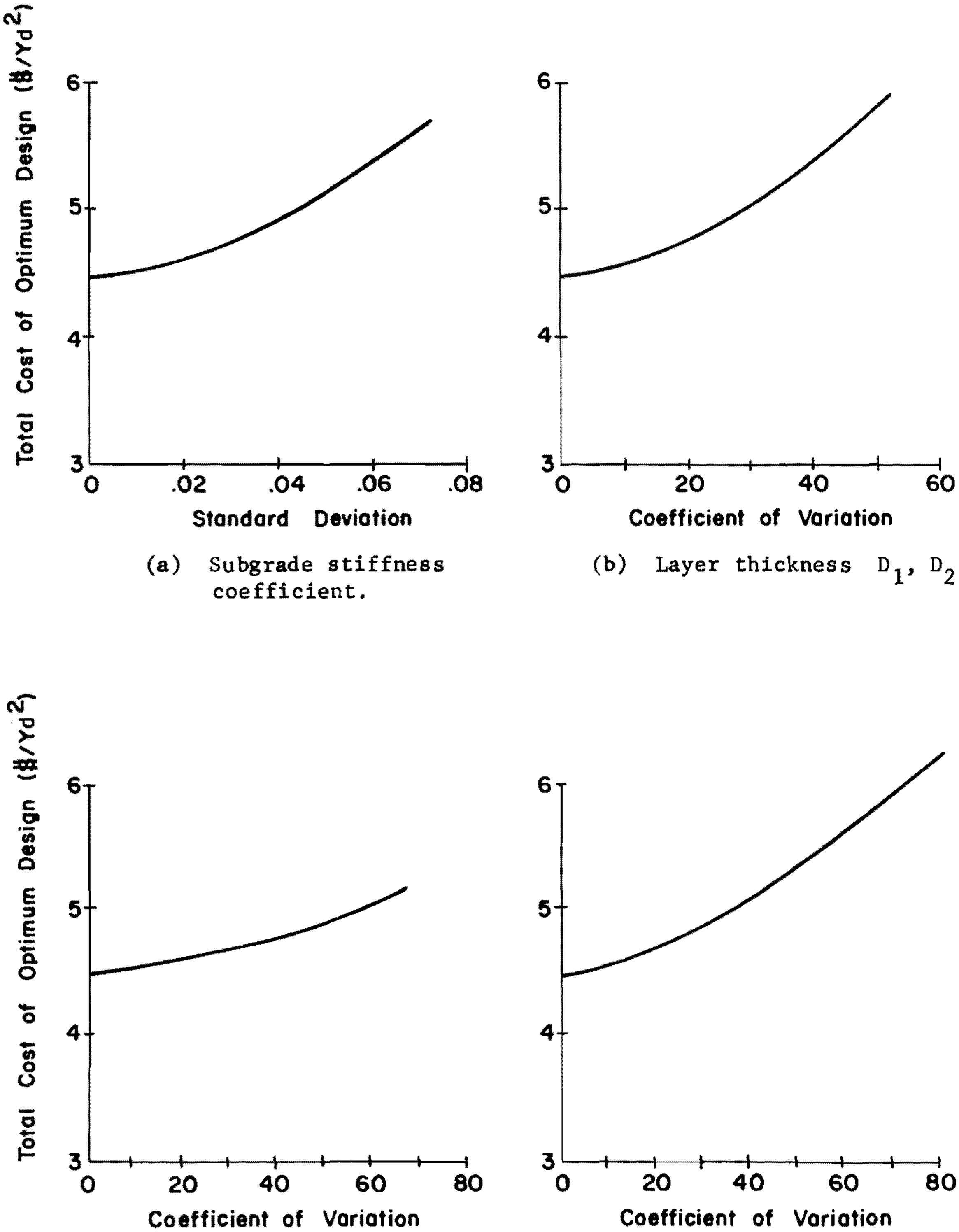
A summary of the inputs is given in Table 7.3. A few additional inputs to this program which are not described in the Texas Highway Department Flexible Pavement Design Manual (Ref 22) are briefly explained here.

Traffic Data. Estimates of design variances for the several traffic parameters were made using the recommendations of Chapter 5 and Table 6.6. The overall variance in estimating log n is 0.0333. The 95 percent confidence limits for the mean would be as follows, assuming log n to be normally distributed:

$$\text{Log } n = \log(3{,}976{,}000) \pm 1.64\sqrt{0.0333}$$
$$= 6.5994 \pm 0.3000$$

or

$$n = 1{,}993{,}000 \text{ to } 7{,}932{,}000$$



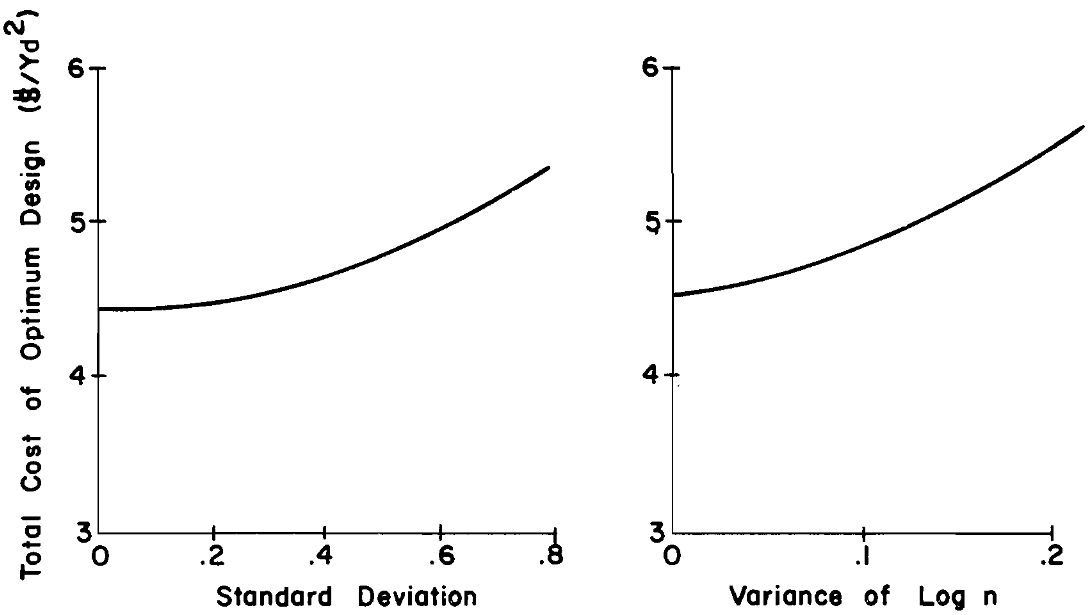
(a) Subgrade stiffness coefficient.

(b) Layer thickness D_1, D_2, D_3.

(c) Black base stiffness coefficient.

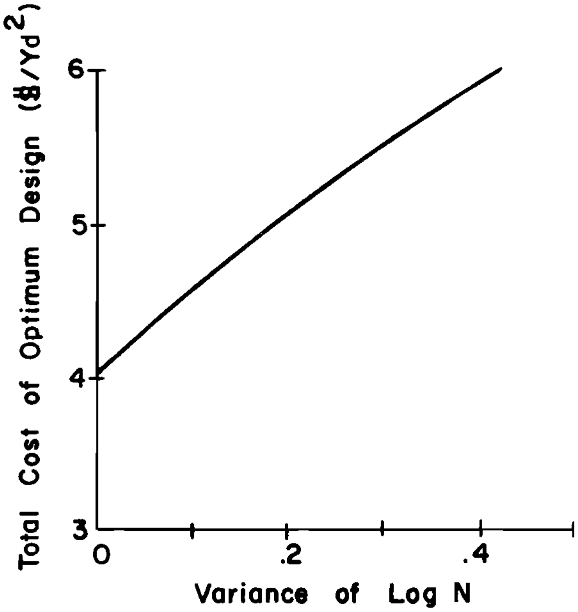
(d) Crushed stone subbase stiffness coefficient.

Fig 7.2. Change in total cost of pavement with corresponding change in variance of specific design factor for R = 99.9 percent (Mopac design).



(a) Initial serviceability index.

(b) Traffic forecast (18-kip applications).



(c) Lack-of-fit: performance and deflection models.

Fig 7.3. Change in total cost of pavement with corresponding change in variance of specific design factors for R = 99.9 percent (Mopac design).

TABLE 7.3. ILLUSTRATIVE PROBLEM OF ASPHALT CONCRETE OVERLAY DESIGN FOR PRIMARY HIGHWAY - INPUTS TO FPS PROGRAM

```
                         TEXAS HIGHWAY DEPARTMENT
                              FPS - 13  CFHR
                            ACP OVERLAY DESIGN

PROB   DIST.      COUNTY       CONT.  SECT.    HIGHWAY     DATE      IPE   PAGE
 6R     19     BOWIE           218      1      US-59     12/1/72              1
******************************************************************************
                     COMMENTS ABOUT THIS PROBLEM

EXAMPLE PROBLEM USING PROBABILISTIC DESIGN METHOD
ASPHALT CONCRETE OVERLAY MODE
HIGHWAY US 59    DESIGN PERIOD 1975 TO 1995
******************************************************************************

BASIC DESIGN CRITERIA
*********************

  LENGTH OF THE ANALYSIS PERIOD (YEARS)                                   20.0
  MINIMUM TIME BETWEEN OVERLAYS (YEARS)                                    6.0
  MINIMUM SERVICEABILITY INDEX P2                                          3.0
  DESIGN RELIABILITY LEVEL                                                  C
  INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)                           7.0

PROGRAM CONTROLS AND CONSTRAINTS
********************************

  NUMBER OF SUMMARY OUTPUT PAGES DESIRED ( 8 DESIGNS/PAGE)                  3
  MAX FUNDS AVAILABLE PER SQ.YD. FOR FIRST OVERLAY (DOLLARS)               9.99
  ACCUMULATED MAX DEPTH OF ALL OVERLAYS (INCHES) (EXCLUDING LEVEL-UP)     11.0

TRAFFIC DATA
************

  ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)                     7800
  ADT AT END OF TWENTY YEARS (VEHICLES/DAY)                             14500
  ONE-DIRECTION 20.-YEAR ACCUMULATED NO. OF EQUIVALENT 18-KSA         3976000
  AVERAGE APPROACH SPEED TO THE OVERLAY ZONE(MPH)                        70.0
  AVERAGE SPEED THROUGH OVERLAY ZONE (OVERLAY DIRECTION) (MPH)           20.0
  AVERAGE SPEED THROUGH OVERLAY ZONE (NON-OVERLAY DIRECTION) (MPH)       30.0
  PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)          5.6
  PERCENT TRUCKS IN ADT                                                  17.0
  DESIGN ADT COEFFICIENT OF VARIATION(PERCENT)                           15.0
  PERCENT TRUCKS COEFFICIENT OF VARIATION                                15.0
  AXLES PER TRUCK COEFFICIENT OF VARIATION(PERCENT)                      10.0
  VARIANCE OF AXLE LOAD/EQUIVALENCY PARAMETER                             .0229

ENVIRONMENT AND SUBGRADE
************************

  DISTRICT TEMPERATURE CONSTANT                                          25.0
  SWELLING PROBABILITY                                                    0.00
  POTENTIAL VERTICAL RISE (INCHES)                                        0.00
  SWELLING RATE CONSTANT                                                  0.00
```

(Continued)

TABLE 7.3. Continued.

INPUT DATA CONTINUED

CONSTRUCTION AND MAINTENANCE DATA
****

SERVICEABILITY INDEX P1 AFTER AN OVERLAY	4.2
MINIMUM OVERLAY THICKNESS (INCHES)	1.0
OVERLAY CONSTRUCTION TIME (HOURS/DAY)	10.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)	1.98
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)	75.0
WIDTH OF EACH LANE (FEET)	12.0
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE-MILE)	30.00
INCREMENTAL INCREASE IN MAINT. COST PER YEAR (DOLLARS/LANE-MILE)	20.00
SERVICIBILITY INDEX P1 AFTER AN OVERLAY STANDARD DEVIATION	.36

DETOUR DESIGN FOR OVERLAYS
****

TRAFFIC MODEL USED DURING OVERLAYING	3
TOTAL NUMBER OF LANES OF THE FACILITY	4
NUMBER OF OPEN LANES IN RESTRICTED ZONE (OVERLAY DIRECTION)	1
NUMBER OF OPEN LANES IN RESTRICTED ZONE (NON-OVERLAY DIRECTION)	2
DISTANCE TRAFFIC IS SLOWED (OVERLAY DIRECTION) (MILES)	.50
DISTANCE TRAFFIC IS SLOWED (NON-OVERLAY DIRECTION) (MILES)	.50
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)	0.00

EXISTING PAVEMENT AND PROPOSED ACP
****

THE COMPOSITE THICKNESS OF THE EXISTING PAVEMENT (INCHES)	13.0
THE IN-PLACE COST/COMPACTED C.Y. OF PROPOSED ACP (DOLLARS)	23.00
SALVAGE VALUE OF PROPOSED ACP AT END OF ANALYSIS PERIOD (PERCENT)	30.0
IN-PLACE VALUE OF EXISTING PAVEMENT (DOLLARS/C.Y.)	6.00
SALVAGE VALUE OF EXISTING PAVT. AT END OF ANALYSIS PERIOD (PERCENT)	75.0
LEVEL-UP REQUIRED FOR THE FIRST OVERLAY (INCHES)	.70
ONE DIRECTION ACCUMULATED NO. OF EQUIVALENT 18-KSA THAT HAVE PASSED OVER PAVEMENT SINCE CONSTRUCTION OR LAST OVERLAY	1072000.00
INITIAL SERVICILITY INDEX OF ORIGINAL PAVEMENT	4.20

MODEL LACK OF FIT
****

DEFLECTION MODEL LACK OF FIT COEFFICIENT OF VARIATION(PERCENT)	38.0

SERVICIBILITY AND SCI OF EXISTING PAVEMENT FOR 0.2 MILE SECTIONS
****

0.2 MILE SECTIONS	SERVICIBILITY INDEX	SURFACE CURVATURE INDEX		
1	3.60	.51	.57	.50
2	3.40	.50	.58	.52
3	3.60	.52	.31	.29
4	3.20	.29	.60	.60
5	2.90	.72	.72	.69
6	3.40	.69	.54	.60
7	3.60	.60	.40	.48
8	3.90	.35	.34	.28

If these estimations of variance and the assumption of normality are correct, the probability of the mean falling between these limits is 0.95. The number of 18-kip equivalent single-axle load applications since construction is also required and was estimated by the Texas Highway Department.

Pavement/Subgrade. The pavement/subgrade variations are characterized by variations of serviceability and SCI along the existing pavement. The 0.2-mile section serviceability and SCI replications within a section are shown in Table 7.3 for eight 0.2-mile sections. These values are used in the program as described in Chapter 7 to "adjust" the performance model to the specific pavement being designed. The average serviceability level of the pavement just after its initial construction must also be estimated. A value of 4.2, which is the overall state average, was assumed.

Design Model Lack-of-Fit. The lack-of-fit of the performance model is the mean square residual between sections as described in Chapter 6. The lack-of-fit of the deflection equation is the same as previously used for overlays where the coefficient of variation is 0.38.

Calculations. The following calculations illustrate the procedure of applying the probabilistic concepts to overlay design for the example problem.

(1) Deflection analysis: The analysis necessary to determine the component of variance of SCI between 0.2-mile sections is shown in Table 7.4.

(2) Performance equation analysis: Calculations showing the determination of adjusted B are shown in Table 7.5. The B will be adjusted so that the average $\log \hat{N}$ predicted equals the actual $\log N_A$ that has passed over the pavement. The B will be calculated using Eq. 6.10.

Using the adjusted regression coefficient, the performance equation now predicts average pavement performance of the particular project *if* all other assumptions were correct.

The total variation associated with prediction of allowable N load applications for the overlayed pavement may be summarized by Eq 6.6:

$$s_{\log N}^{2} = \frac{0.0471}{\bar{Q}^{2}}\left[\frac{s_{P1}^{2}}{5-P1}\right] + \frac{0.755 s_{SCI}^{2}}{\overline{SCI}^{2}} + 0.189 s_{\alpha}^{2}/\bar{\alpha}^{2} + s_{l.o.f.}^{2}$$

TABLE 7.4. DETERMINATION OF SCI COMPONENT OF VARIANCE BETWEEN 0.2-MILE SECTIONS - ANALYSIS OF VARIANCE

Source Variation	df	SS	MS	F	Expected Mean Square
Between sections	7	0.3143	0.0449	5.23*	$\sigma^2 + m\ \sigma_B^2$
Within sections	16	0.1380	0.0086		σ^2
Total	23	0.4523			

*Significant at level of significance < 0.01. Therefore, there is a significant difference in deflection between sections.

σ^2 = expected <u>within</u> <u>section</u> variation of SCI

σ_B^2 = expected component of variance <u>between</u> <u>sections</u>

m = number of replicates within a section

therefore

$$\sigma_B^2 = (0.0449 - 0.0086)/3 = 0.0121$$

TABLE 7.5. DETERMINATION OF "ADJUSTED" B COEFFICIENT FOR PERFORMANCE MODEL

Section	$\overline{SCI}$	P2	Q	$\frac{Q\alpha}{\overline{SCI}^2}$	$\log\left(\frac{Q\alpha}{\overline{SCI}^2}\right)$	$\log N_A$	Diff.
1	0.527	3.6	0.289	26.03	1.4154	0.0294	1.3860
2	0.533	3.4	0.370	32.56	1.5127		1.4833
3	0.373	3.6	0.289	51.80	1.7143		1.6849
4	0.497	3.2	0.447	45.32	1.6563		1.6269
5	0.710	2.9	0.555	27.51	1.4395		1.4101
6	0.610	3.4	0.370	24.89	1.3960		1.3666
7	0.493	3.6	0.289	29.66	1.4722		1.4428
8	0.323	3.9	0.154	36.92	1.5672	0.0294	1.5378
						Average =	1.4923
						B =	31.0

where

Q = $\sqrt{5-P2} - \sqrt{5-P1}$

N_A = $1{,}072{,}000 \times 10^{-6}$

Diff. = $\log\left(\frac{Q\alpha}{\overline{SCI}^2}\right) - \log N_A$

$\overline{SCI}$ = mean SCI, 0.508×10^{-3} inch.

B = 31.0, which represents the new regression coefficient to be used for this project in place of 53.6.

P1 = 4.2

α = 25.0

where

$$s^2_{P1} = (0.3)^2 + (0.2)^2 = 0.13$$

$\overline{SCI}$ = average SCI of overlayed pavement, 0.236 for R = 95 percent

$$s^2_{SCI} = (0.236)^2[0.0121/(0.508)^2] + (0.236^2)(0.38)^2$$

$$= 0.0107$$

$$s^2_{\alpha} = 0$$

$$s^2_{l.o.f.} = 0.0134$$

therefore

$$s^2_{\log N} = 0.1867$$

The design $\log N_R$ may now be determined where $R = 0.95$

$$\log N_{95} = \log(3{,}976{,}000) + 1.64\sqrt{0.1867 + 0.0333}$$

$$= 7.3687$$

Outputs. The FPS overlay mode also outputs an array of possible design strategies sorted by total cost. The program may be run at various levels of design reliability. A summary of the optimum design (minimum total cost) at several levels is shown in Table 7.6. As the reliability increases, the total cost of optimum designs also increases. The design tentatively selected for construction is at R = 95 percent. Criteria for selection of design reliability are detailed in Chapter 8.

The selected design strategy calls for 4.2 inches of asphalt concrete to be placed initially, which yields life of about 8 years. An overlay of 1.5 inches that will last to 15 years is then scheduled. Another overlay of

TABLE 7.6. SUMMARY OF OPTIMUM (TOTAL COST) DESIGNS FOR EXAMPLE OVERLAY DESIGN FOR US 59 AT VARIOUS RELIABILITY LEVELS

Design Criteria	Reliability Level, percent					
	50	80	95	99	99.9	99.99
Initial overlay	1.09	1.72	2.68	3.64	4.28	4.92
User delay	.08	.13	.21	.28	.33	.38
Future overlay	.46	.97	.91	.70	.77	.77
User delay	.05	.11	.10	.09	.09	.09
Routine maintenance	.18	.13	.14	.18	.16	.16
Salvage	-.52	-.62	-.69	-.77	-.82	-.87
Total cost ($\$/Yd^2$)	1.34	2.45	3.34	4.13	4.82	5.46
Overlay policy (inches)						
Initial overlay	1.7	2.7	4.2	5.7	6.7	7.7
Second overlay	1.5	1.5	1.5	1.5	1.5	1.5
Third overlay		1.5	1.5	1.5	1.5	1.5
Performance time (years)						
Initial overlay	11.1	7.3	7.9	10.7	9.8	9.5
Second overlay	20.5	14.1	15.0	19.9	18.3	17.8
Third overlay		20.6	22.0	28.5	26.4	25.7

1.5 inches is then required, and it should last to the end of the analysis period.

The effect of reliability level and the magnitude of variation of the design parameters for this specific overlay design problem are shown in Fig 7.4. As the reliability level increases the total cost of the optimum design also increases. The increase in cost is due to the increase in asphalt concrete overlay thickness required with increasing reliability. The upper curves in Fig 7.4 represent the relationship when the variation of all design parameters are considered. The lower curve represents the cost of the optimum design without the variance of each of the sources of variation shown. The difference between the upper and lower curves represents the additional costs due to veriations of each parameter. The variations due to lack-of-fit of the performance and deflection models appear to have the largest effect of all the types.

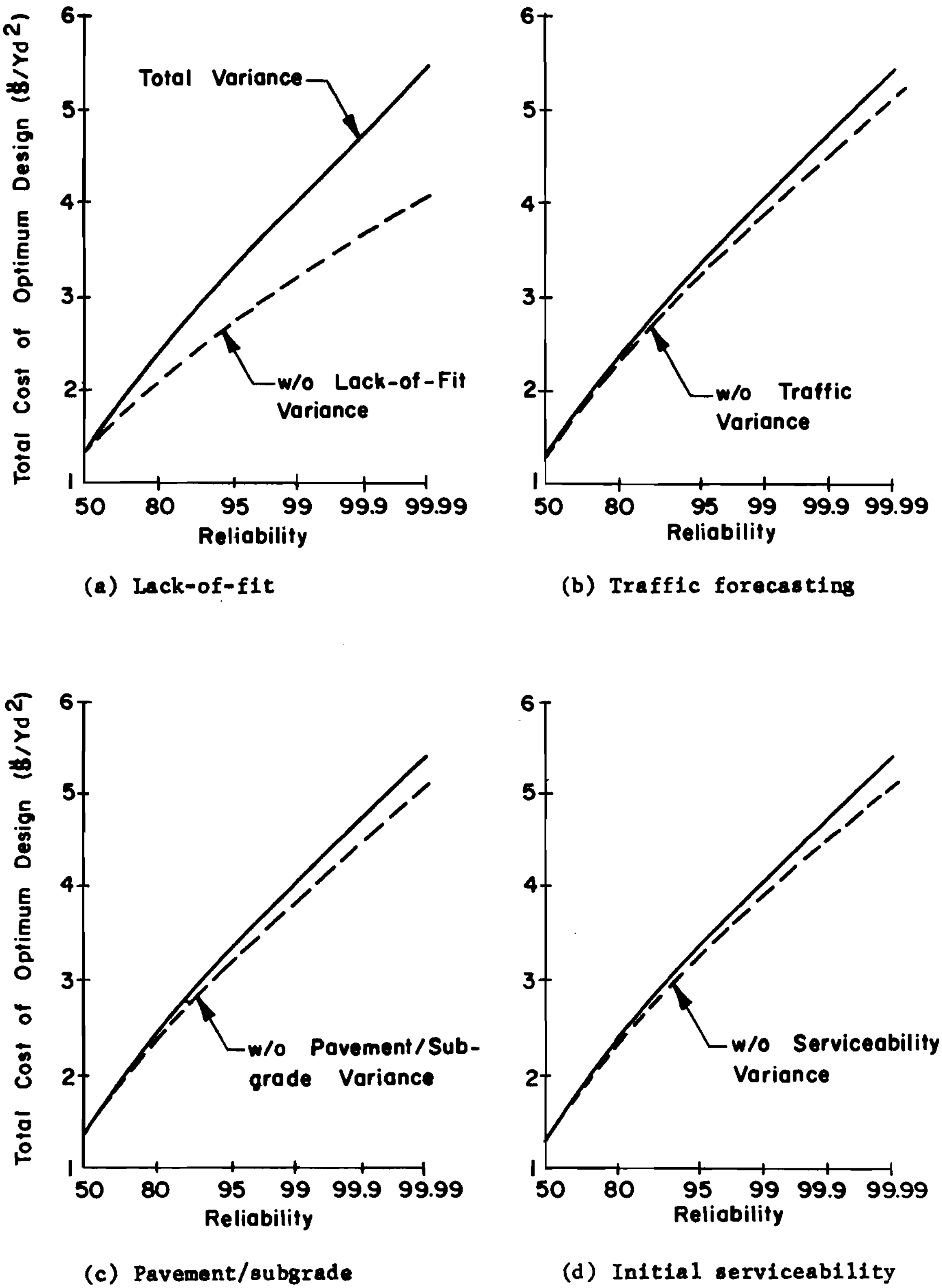


Fig 7.4. Total cost versus reliability for specific asphalt concrete overlay design problem (US 59).

CHAPTER 8. RELIABILITY LEVEL FOR DESIGN

The concept of pavement systems reliability was introduced in Chapter 2. A definition of reliability was given along with the conceptual relationships between reliability R , performance P , and costs C . The probabilistic nature of the design parameters and the necessary theory to consider them in design have also been presented so that a pavement can be designed for a desired level of reliability. A very important design input, which has not been discussed yet, is the level of reliability to be used in design of various types of pavements. The analysis of this most important aspect with recommendations is the subject of this chapter. A conceptual analysis is given first, followed by the practical method used to develop tentative levels for use by the Texas Highway Department, and, finally, specific recommendations.

Conceptual Analysis

Reliability was defined as the probability that the pavement system will perform its intended function over its design life and under the conditions (or environment) encountered during operation. To "perform its intended function" was defined as an expected percentage of pavement sections showing an adequate serviceability within a limited maintenance cost over a specified period. Performance for a specific pavement can be defined as the area under the serviceability history curve (the integral of the serviceability) over the design analysis period. The total cost associated with the pavement system facility is the sum of initial construction, overlay, routine maintenance, user-delay due to planned overlay operations, and salvage value, with future costs discounted to a present worth. There are also motorists costs due to rough pavements such as vehicle maintenance and operation, accidents, and delay time due to decreased speeds.

The higher the reliability level; the higher the performance or average serviceability level throughout the design life, the higher the associated facility costs, and the lower the motorist costs due to rough pavement. In the final analysis, the reliability requirements of a pavement are determined by its user's requirements. If the reliability level is set too low, the pavement will operate at low serviceability levels resulting in high motorist costs. This will include numerous complaints from the traveling public. If an extremely high reliability level is used, total facility costs will also be excessive and fewer pavements can be provided due to scarcity of funds. It is believed that pavements should be designed for just the level of reliability that will provide the level of performance desired by the user and no greater, because

(1) pavements upon reaching minimum acceptable serviceability do not seriously endanger human lives
(2) there is an increasing scarcity of highway construction funds.

This level of performance has never been quantified for actual in-service highway pavements and probably varies depending upon highway type, traffic volume, desired traveling speed, type of terrain, comfort, and other factors.

The level of design reliability is a function of similar factors such as (1) type of highway - interstate, primary, or secondary; (2) traffic volume and character; (3) adequate detour availability; (4) available funds; and (5) confidence in design procedure. It is apparent that setting the design reliability level is a complex problem but guidelines must be developed so that pavements of similar characteristics throughout the state can be designed for the same reliability using the FPS program. This will provide for uniformity of design and also for minimization of costs.

An important point which must be reemphasized is that the R level as defined in this study for a particular project is only accurate if the corresponding variances of the design factors and models are accurate. Due to limitations in obtaining data from which to estimate these variances, the R is not to be considered a precise value but only approximate. Recommended future work would include better quantification of the variations of the design factors and models.

A conceptual understanding of the nature of relationships between the reliability, performance, and costs may be helpful in setting guidelines for determining the design reliability level. To accomplish this, the FPS-13 (CFHR) program was run at varying levels of R for the MOPAC design project. The optimum design was selected in each case as is summarized in Table 7.2 in

Chapter 7. For each reliability level the total facility cost was obtained and the performance calculated as the area contained between the serviceability-load applications curve and the minimum acceptable serviceability level. This was determined by integration of the performance model as shown in Fig 2.6. The relationships between R , P , and C are shown in Figs 8.1 and 8.2. The cost and performance scales are shown as percentage increase over the 50 percent reliability level values. There is no available method by which to determine motorist costs and hence they are not shown in Fig 8.1.

The relationships between R and C and between R and P , shown in Fig 8.1, are curvilinear with no abrupt breaking points. The relationship between P and C , shown in Fig 8.2, is the most important and shows a fairly abrupt change in slope, indicating a significant difference occurs in the rate of change of costs and performance. The increase in performance with increase in cost increases rapidly for reliability levels up to about 90 percent. This value may change for other problems however and should only be considered as illustrative. The important concept that this analysis illustrates is that there is considerable increase in the level of performance with increase in costs until a certain reliability level is reached, where the benefit of higher reliability does not result in much increase in performance.

Practical Method to Determine Design Reliability

The selection of a design reliability level must be a practical matter. The state-of-the-art of pavement design has not progressed to a point at which the complete relationships between R , P , and C can be exactly determined analytically. Since it is believed that the reliability level of design should be determined by the user, the following procedure was developed. It is believed that considerable interaction with experienced pavement designers of the Texas Highway Department would be the best way to establish reliability factors which provide for an economical balance between performance and costs. The designers are faced with the dual problems of answering to the consequence of failure and also the consequence of spending too much money on a few projects and thereby not having enough funds for other needed construction.

Design data were obtained for 12 projects ranging from farm to market roads to urban freeways from five districts and pavement designs were made using the FPS-11 program. Pavement designs for each of the 12 projects were

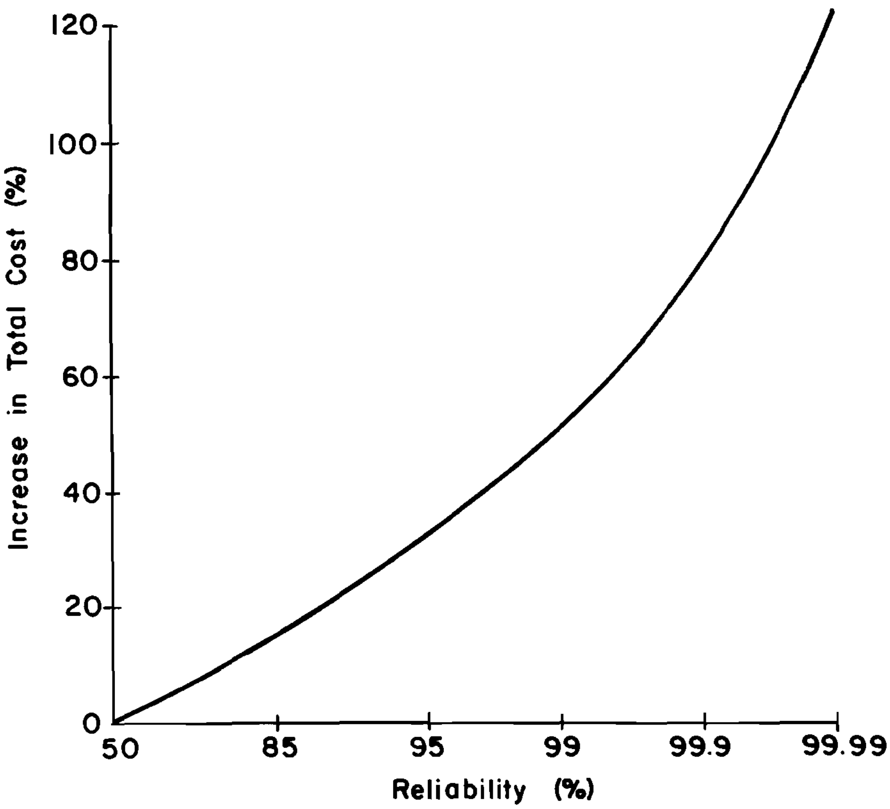


Fig 8.1(a). Reliability versus percent increase in total cost for Mopac design project.

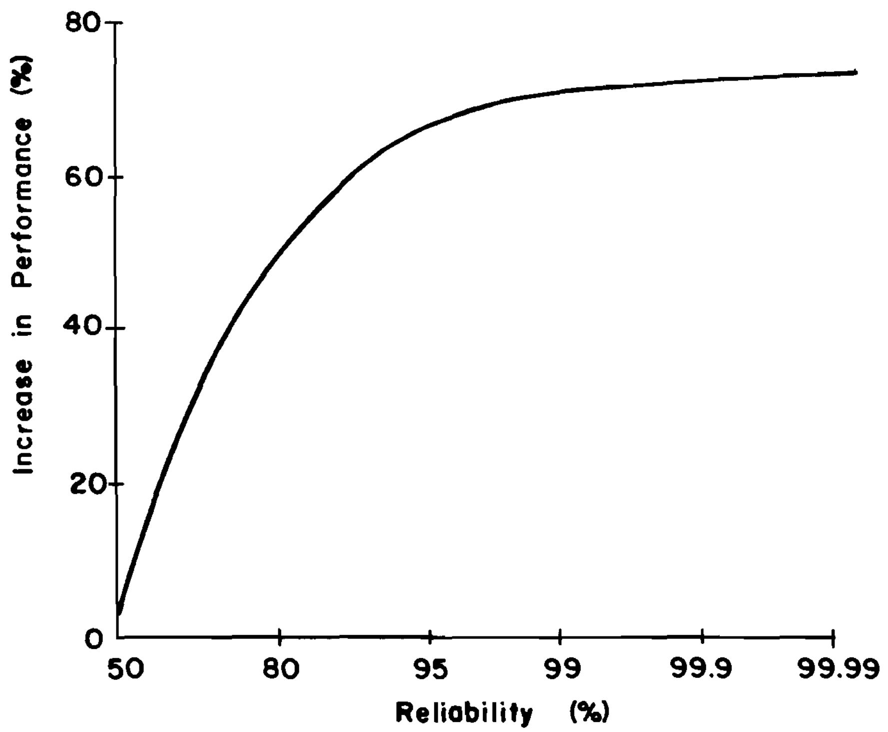


Fig 8.1(b). Reliability versus percent increase in performance for Mopac design project.

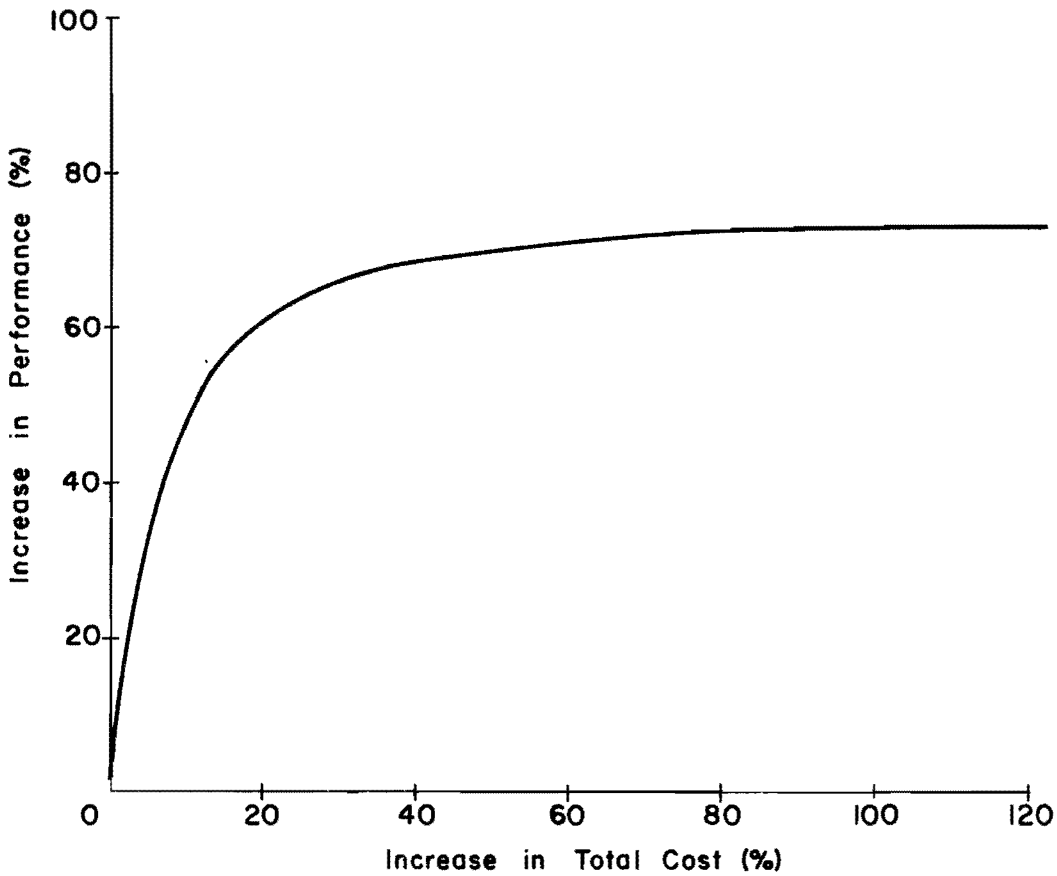


Fig 8.2. Percent increase in total cost versus percent increase in performance for Mopac design project.

obtained at coded reliability levels of A , B , C , D , E , and F and were analyzed by experienced design engineers of the district in which the projects were located. The designers then selected the design strategy that they believed to be adequate (or the one that they would construct) from among the six designs at varying reliability levels. A summary of the basic characteristics of these projects and the level selected for design using the FPS-11 program are summarized in Table 8.1. The design inputs/outputs of the optimum design are summmarized in Appendix 2. The specific letter codes represent the following reliabilities:

A = 50 percent	D = 99 percent
B = 80 percent	E = 99.9 percent
C = 95 percent	F = 99.99 percent

The reliability level chosen for design appears to increase in general with the functional classification, traffic volume, and 18-kip equivalent single-axle load applications. There were undoubtedly other factors which entered into the decision, such as the magnitude of congestion if the pavement should require early maintenance inputs. A general conclusion is that the Texas Highway Department designers are inherently considering the consequences of failure, whether the consequences be political or economic, from their experience and providing a smaller risk of premature failure for pavements that are of greater importance to the highway user. The recommended levels for design of different pavements are presented in the next section.

Recommendations

The level of reliability represents the expected level of pavement performance that will be obtained. The higher the level, the greater will be the average serviceability history throughout the design period and the greater will be the associated facility costs involved. The design level should be a direct function of the problems or consequences of failure that will occur if the pavement must be overlaid or reconstructed prematurely. The user's manual for the FPS-11 program states the following:

> The problems arising because of failure to provide the specified quality throughout the analysis period depend upon the type of repair required to restore serviceability, the relative amount of traffic using the facility during this repair, and the availability of a detour route for this traffic (Ref 22).

TABLE 8.1. SUMMARY OF PROJECTS AND SELECTED DESIGN RELIABILITY LEVEL USING FPS-11 PROGRAM

Design Number	District	Highway	Functional Classification+	ADT	Equivalent 18-kip Single-Axle Loads	Selected Reliability Level
1	19	FM 2625	Minor Collector	135*/260**	30,000	C
2	19	FM 1840	Minor Collector	730/1215	289,000	C
3	5	US 70 & 84	Principal Arterial	955/1950	658,000	C
4	19	SH 300	Major Collector	1100/1850	637,400	D
5	2	SH 24	Minor Arterial	1600/3200	800,000	D
6	5	US 84	Principal Arterial	1670/2750	1,069,000	D
7	19	US 271	Minor Arterial	1350/2150	1,450,000	D
8	14	SH 71	Principal Arterial	2800/4900	1,562,000	D
9	17	US 290	Principal Arterial	2130/6200	3,661,000	D
10	5	Loop 289	Principal Arterial	2725/16400	2,840,000	E
11	2	SH 360	Principal Arterial	6800/15100	4,657,000	E
12	14	Loop 1	Principal Arterial	19660/32380	6,894,000	E

+From Ref 32

*Initial one-direction ADT

**End one-direction ADT

Recommended levels of design reliability for the FPS-11 program as contained in the user's manual (Ref 22) are shown in Table 8.2. Only three levels are recommended: C , D , and E . The decision cirteria consist of (1) whether or not the project is located in an urban or rural area and (2) whether or not the highway will be operating at less than or greater than 50 percent capacity throughout the analysis period. The higher reliability is associated with the urban area location and with the traffic volume greater than 50 percent of the capacity.

The results obtained from the 12 projects were analyzed further and additional recommendations were developed to supplement those contained in Table 8.2. Proposed criteria are as follows:

(1) number of 18-kip equivalent single-axle loads;

(2) the degree to which traffic congestion will be a problem during overlay operation, which depends upon traffic volume and available detours;

(3) highway functional classification, arterial or collector; and

(4) location of highway, urban or rural.

The procedure is shown in Table 8.3. The design reliability level can be selected using Table 8.3 if the above criteria are known. If the road is in an urban area, the higher reliability level should be used wherever alternate levels are given. These recommended design reliability values are tentative only and usage and experience with the FPS design system will provide verification and improvement. The recommended levels of C , D , and E represent reliabilities greater than 90 percent which lie above the break point in the C versus P curve shown in Fig 8.2 but not too far out on the curve. They therefore seem to be reasonable values according to the previous analysis.

The selected design reliability levels for the 12 projects were for new pavement or reconstruction of existing pavements. There are no data available concerning recommended design reliability for overlay of existing pavements. It would seem that a somewhat lower level of reliability could be used for most pavements because the designer would expect less risk of failure of an overlay than of completely new construction or reconstruction of a pavement structure. In some cases in which a pavement has deteriorated very rapidly the opposite may be true and the designer would then design for higher reliability as that the pavement would not show the same failure rate. Therefore, it is recommended that the same reliability levels be used for the overlay mode of

TABLE 8.2. GUIDELINES FOR SELECTING THE DESIGN CONFIDENCE LEVEL (from FPS User's Manual, Ref 22)

	The highway will remain rural throughout the analysis period	The highway is or will become urban before the end of the analysis period
The highway will be operating at greater than 50% of capacity sometime within the analysis period	C or D	E
The highway will be operating at less than 50% of capacity throughout the analysis period	C	D or E

TABLE 8.3. RECOMMENDED DESIGN RELIABILITY LEVELS FOR FPS PROGRAM

Traffic Handling situation	Functional classification	Equivalent 18-kip single-axle loads: <500,000	500,000 to 2,000,000	>2,000,000
Satisfactory	Collector	C	C or D*	D
	Arterial	C	C or D	D or E
Some Problems	Collector	C	C or D	D
	Arterial	C or D	D	D or E
Considerable Problems	Collector	C or D	D or E	E
	Arterial	D	D or E	E

*Note: If pavement is located in urban area, use higher reliability level wherever range is given.

the FPS program as are used for the new or reconstruction mode, as given in Tables 8.2 and 8.3. However, the reliability level for the overlay mode may be decreased by one letter for a pavement that has not shown abnormal deterioration during the past performance period. These recommendations are tentative until additional data can be obtained.

Setting Reliability Levels for Modified Programs

The design input and output data obtained from these 12 projects represent information that can be used as a standard of correct solutions by which future revisions of the FPS program can be compared. The letter codes representing reliability levels were used so that changes in the variance models or probabilistic system could be handled by changing the reliability levels that the letter codes represent. The required change can be determined by comparing the output of the new program with the output of the original 12 projects. The FPS-13 (CFHR) program contains several design considerations which the FPS-11 program did not consider, and which were discussed in Chapter 6. It is therefore desirable to obtain new estimates of the design reliability level for each of the 12 projects so that the FPS-13 (CFHR) program can be used for design using the same recommended codes. The determination of a new design reliability level for each of the 12 projects can be done by equating the basic probabilistic design models as used in FPS-11 and FPS-13 (CFHR) and solving for the new reliability level.

The design model used in the FPS-11 program is as follows:

$$\overline{\log N}_{FPS-11} = \overline{\log n} + Z_{FPS-11}\, s_{FPS-11} \tag{8.1}$$

where

$\overline{\log N}_{FPS-11}$ = number of 18-kip equivalent single-axle loads used for design in the FPS-11 program

$\overline{\log n}$ = average number of 18-kip equivalent single-axle loads predicted to pass over pavement

Z_{FPS-11} = standardized normal deviate (mean = 0, standard deviation = 1) representing the level of reliability used in the FPS-11 program

s_{FPS-11} = standard devision associated with $\log N$ as predicted by the FPS-11 variance model

The probabilistic design model used in the new FPS-13 (CFHR) program is as follows:

$$\overline{\log N}_{FPS-13} = \overline{\log n} + Z_{FPS-13}(s_{FPS-13}) \tag{8.2}$$

where

$\overline{\log N}_{FPS-13}$ = number of 18-kip equivalent single-axle load applications used for design in FPS-13 (CFHR)

Z_{FPS-13} = standardized normal deviate representing the new level of reliability to be used in the FPS-13 (CFHR) program

s_{FPS-13} = standard deviation associated with $\log N$ and $\log n$ as predicted by the FPS-13 (CFHR) variance models

Similar designs may be obtained if $\overline{\log N}_{FPS-11}$ is equal to $\overline{\log N}_{FPS-13}$ and therefore Eq 8.1 and Eq 8.2 may be set equal and the Z_{FPS-13} obtained.

$$Z_{FPS-13} = Z_{FPS-11}\left[\frac{s_{FPS-11}}{s_{FPS-13}}\right] \tag{8.3}$$

The Z_{FPS-13} was calculated for each of the 12 projects given in Table 8.1. The reliability level corresponding to Z_{FPS-13} was then obtained from the distribution tables and is shown in Table 8.4. In all cases, the new design reliability determined for FPS-13 (CFHR) is less than the reliability used for FPS-11. This is due to the consideration of additional sources of variance in the FPS-13 (CFHR) program such as traffic estimation error.

The average value of reliability for each level of design was obtained by averaging the Z_{FPS-13} values for each project in the C, D, and E categories. The recommended design reliabilities to be used in the new program are given in Table 8.5. If these values are used in the FPS-13 (CFHR) program to design the 12 projects, the outputs would be similar to the original outputs selected by the design engineers for construction. If the new

TABLE 8.4. DESIGN RELIABILITY LEVEL FOR FPS-11 PROGRAM AND FOR FPS-13 (CFHR) PROGRAM

Design Number	Highway	Reliability FPS-11	Reliability FPS-13 (CFHR)
1	FM 2625	95.0	93.7
2	FM 1840	95.0	92.4
3	US 70 & 84	95.0	92.1
4	SH 300	99.0	97.3
5	SH 24	99.0	96.5
6	US 84	99.0	98.2
7	US 271	99.0	98.3
8	SH 71	99.0	97.9
9	US 290	99.0	96.8
10	Loop 289	99.9	99.3
11	SH 360	99.9	99.7
12	Loop 1	99.9	99.6

program is adopted by the Texas Highway Department for use in design, the Z values corresponding to the C , D , and E levels would be modified to those shown in Table 8.5. This method provides a workable method of determining the design reliabilities to be used in design when any new program is to be implemented.

TABLE 8.5. RECOMMENDED DESIGN RELIABILITY FOR THE FPS-13 (CFHR) PROGRAM

Code	FPS-11 Level	FPS-13 Level
A	50.00	50.00
B	80.00	*
C	95.00	92.90
D	99.00	97.60
E	99.90	99.60
F	99.99	*

* Could not be determined due to lack of data.

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

This chapter briefly summarizes the significant conclusions reached in this study and makes specific recommendations to the sponsor, the Texas Highway Department, as to future research work in this area.

Conclusions

A brief summary of the results and basic conclusions reached in this study is as follows:

(1) A major problem which exists in pavement design at the present time is the consideration in design of the inherent uncertainty and variation of the design parameters and of the design models. Many of these existing variations have been illustrated in this study. Empirical safety and judgement factors have been applied in the past to "adjust" for the many uncertainties involved. These safety factors usually do not depend upon the magnitude of variations involved and therefore have resulted in much overdesign and underdesign. A significant need was found to develop a method which would consider the associated variations and uncertainties of pavement design on a <u>quantitative</u> basis whereby designs can be made to specified levels of adequacy or reliability.

(2) As a basic start towards the solution of this problem, the theory and procedures were developed, based upon classical reliability theory, to apply probabilistic design concepts to flexible pavement system design. This method makes it possible to design for a desired level of reliability through the consideration of the variabilities and uncertainties associated with pavement design. The probabilistic theory has been applied to the Texas flexible pavement system (FPS), which was originally a deterministic method. The system considers the following variations:

(a) variations within a design project length,
(b) variations between design and actual values, and
(c) variations due to lack-of-fit of the design models.

Approximate estimates of these variations were made for specific design parameters and models of the Texas FPS system, which included pavement layer and subgrade stiffness coefficients, initial serviceability, termperature parameters, performance model, deflection model, and traffic forecasting.

(3) The probabilistic theory and procedures have been shown to be both practical and implementable by being actually incorporated into the daily operations of the Texas Highway Department. The procedures were originally implemented into the deterministic FPS-7 version during 1971 and the new program was called FPS-11. This version has been used by ten districts of the Texas Highway Department since late 1971. The FPS program consists of a new or reconstruction mode and an overlay of existing pavement mode. During 1972 the FPS-11 program has been further developed to include variations occurring in individual pavement layers and the subgrade and consideration of traffic forecasting errors. The overlay mode was also improved by making it possible to adjust the performance model to a specific pavement by considering its past performance history. A new program called FPS-13 (CFHR) was developed to include these variations, which add new capabilities to the system. Design examples are given of actual projects and the results illustrate the potential of the probabilistic method.

A practical method was developed to set tentative design reliability levels. Recommended levels have been developed based upon specific characteristics of the pavement being designed; this will assist in producing uniform reliability in pavement design. The recommendations may also result in more optimized designs.

A procedure was also developed using the probabilistic approach to perform sensitivity or significance analysis of design models. This makes it possible to determine the relative effect of each design parameter on pavement performance and to determine research priorities.

(4) Basically, the probabilistic design procedures documented in this study provide a first order approach to a practical and implementable method to quantify adequacy of designs by considering uncertainties and variations and designing for specified levels of reliability. The method may be applied to existing pavement design procedures if the variation and uncertainties associated with the method can be quantified.

Recommendations

The following recommendations concerning the probabilistic design method are made in light of the results of this study:

(1) The method which has been applied to the Texas flexible pavement system has proved successful in many ways. Considerable work remains however in the quantification of variations and uncertainties involved. A major study is recommended to investigate and quantify the variations of the FPS design parameters and models. Such a study will

greatly improve the estimates now available and make the system more closely predict actual pavement performance.

(2) This study has concentrated upon the structural aspects of the pavement design system. Other subsystems which deserve attention are the economic, the safety, and the user delay.

(3) The basic probabilistic concepts should be applied to any mechanistic design subsystem which may be implemented into the FPS system. It is believed that this would assist greatly in the developement and implementation of such a procedure.

(4) Further study of the probabilistic design approach is needed to improve or supplement the basic method developed in this study. The improvements must be practical, however, and capable of implementation into the operations of the Texas Highway Department.

(5) The FPS-11 program is currently being implemented and used by ten districts of the Department. The FPS-13 (CFHR) version is believed to contain improvements which would be beneficial in design. It is recommended that the FPS-13 (CFHR) program be considered for implementation by the Department to make available the added capabilities to the pavement designers.

REFERENCES

1. "AASHO Interim Guide for the Design of Flexible Pavement Structures," AASHO Committee on Design, October 1961.

2. Advisory Committee of FHWA-HRB Workshop on Structural Design of Asphalt Concrete Pavement Systems, Special Report 126, Highway Research Board, 1971, p 8.

3. Alexander, M., and R. A. Graves, III, "Determination of the Lanal Distribution of Truck Traffic on Freeway Facilities," Final Report, Project No. 7001, State Highway Department of Georgia, 1971.

4. Annual Report 1970, "Permanent Automatic Traffic Recorders," Planning Survey Division, Texas Highway Department, 1970.

5. Au, Tung, and T. E. Stelson, Introduction to Systems Engineering, Deterministic Models, Addison-Wesley Publications, 1969.

6. Benjamin, J. R., and H. C. Lind, "A Probabilistic Basis for a Deterministic Code," Proceedings of the American Concrete Institute, November 1969.

7. Brown, C. B., "Concepts of Structural Safety," Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol 86, No. ST12, December 1960, pp 39-57.

8. Brown, James L., Larry J. Buttler, and Hugo E. Orellana, "A Recommended Texas Highway Department Pavement Design Systems Users Manual," Research Report No. 123-2, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, March 1970.

9. Brown, J. L., and H. E. Orellana, "An Asphaltic Concrete Overlay Design Subsystem," Research Report 101-1F, Texas Highway Department, 1970.

10. Brunk, H. D., An Introduction to Mathematical Statistics, Second Edition, Blarsdell Publishing Co., 1965, pp 33-34.

11. Buffington, J. L., D. L. Schafer, and W. G. Adkins, "Procedures for Estimating the Total Load Experience of a Highway as Contributed by Cargo Vehicles," Research Report 131-2F, Texas Transportation Institute, Texas A&M University, 1970.

12. Carey, W. N., Jr., and P. E. Irick, "The Pavement Serviceability-Performance Concept," Bulletin 250, Highway Research Board, 1960, pp 40-58.

13. Carey, W. N., Jr., "Some Remarks on Research for Structural Design of Asphalt Concrete Pavement Systems," Special Report 126, Highway Research Board, 1971, pp 61-62.

14. Cornell, C. A., "A Probability-Based Structural Code," Proceedings of the American Concrete Institute, December 1969.

15. Darter, M. I., B. F. McCullough, and J. L. Brown, "Reliability Concepts Applied to the Texas Flexible Pavement Design System," paper submitted to Highway Research Board for presentation at the Annual Meeting in January 1972, Washington, D. C., Center for Highway Research, August 1971.

16. Deacon, J. A., and R. L. Lynch, "Determination of Traffic Parameters for the Prediction, Projection, and Computation of EWL's," Final Report KYHPR-64-21 HPR-1(4), Kentucky Highway Department, August 1968.

17. Deacon, J. A., "Load Equivalency in Flexible Pavements," Proceedings of the Association of Asphalt Paving Technologists, Vol 38, 1969, pp 465-494.

18. Derdeyn, C. J., "A New Method of Traffic Evaluation for Pavement Design," Highway Research Record No. 46, Highway Research Board, 1964, pp 1-10.

19. Draper, N. R., and H. Smith, Applied Regression Analysis, John Wiley and Sons, Inc., 1966.

20. Feigenbaum, A. V., Total Quality Control, McGraw-Hill Book Co., Inc., New York, 1961.

21. Finn, F. N., "Observations of Distress in Full Scale Pavements," Special Report 126, Highway Research Board, 1971, pp 86-90.

22. "Flexible Pavement Designer's Manual - Part I, Texas Highway Department Pavement Design System," Highway Design Division, Texas Highway Department, 1972.

23. Freudenthal, A. M., and M. Shinozuka, "Structural Safety Under Conditions of Ultimate Load Failure and Fatigue," WADD Technical Report 61-177, October 1961.

24. Freudenthal, A. M., "Critical Appraisal of Safety Criteria and Their Basic Concepts," Preliminary Publication, 8th Congress, International Association for Bridge and Structural Engineering, New York, New York, 1968.

25. Freudenthal, A. M., J. M. Garretts, and M. Shinozuka, "The Analysis of Structural Safety," Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol 92, No. ST1, Proceedings Paper 4682, February 1966, pp 267-325.

26. Freudenthal, A. M., "Load Factors," Journal of the American Concrete Institute, March 1960.

27. Freudenthal, A. M., "Prediction of Fatigue Failure," Journal of Applied Physics, Vol 31, December 1960.

28. Freudenthal, A. M., "Reliability Analysis of Fatigue-Sensitive Structure," ACTA Technica, Academiac Scientiarum Hungaricai, 1961.

29. Freudenthal, A. M., "Safety and the Probability of Structural Failure," Transactions, Proceedings of the American Society of Civil Engineers, Vol 121, Paper 2843, 1956, pp 1337-1375.

30. Freudenthal, A. M., "Safety of Structures," Transactions, Proceedings of the American Society of Civil Engineers, Vol 112, 1947, pp 125-180.

31. Freudenthal, A. M., "Safety, Reliability and Structural Design," Transactions, Proceedings of the American Society of Civil Engineers, Vol 127, Part II, Paper 3372, 1962, p 304.

32. Functional Classification Map, Texas Highway Department, 1969.

33. General Outline of the Method Used by the Texas Highway Department in Predicting Future Traffic on Rural State Highways, Planning Survey Division, Texas Highway Department, 1971.

34. Granley, Edwin C., "Quality Assurance in Highway Construction, Part 4 - Variations of Bituminous Construction," Public Roads, Vol 35, No. 9, August 1969.

35. Granley, Edwin C., "Quality Assurance in Highway Construction, Part 6 - Control Charts," Public Roads, Vol 35, No. 11, December 1969.

36. Grumm, F. J., "Designing Foundation Courses for Highway Pavements and Surfaces," California Highways and Public Works, Vol 20, No. 3, March 1942, pp 6-9, 20.

37. Haas, R. G., "Developing a Pavement Feedback Data System," Research Report 123-4, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, 1971.

38. Haugen, E. B., Probabilistic Approaches to Design, John Wiley and Sons, Inc., New York, 1968.

39. Heathington, K. W., and P. R. Tutt, "Estimating the Distribution of Axle Weights for Selected Parameters," Highway Research Record No. 189, Highway Research Board, 1967.

40. Hudson, W. Ronald, B. Frank McCullough, F. H. Scrivner, and James L. Brown, "A Systems Approach Applied to Pavement Design and Research," Research Report 123-1, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, March 1970.

41. Hudson, W. R., and B. F. McCullough, "Translating AASHO Road Test Findings, Basic Properties of Pavement Components, Systems Formulation," Final Report, NCHRP Project 1-10 and 1-10/1, Materials Research and Development, Inc., December 1970.

42. Hudson, W. R., B. F. McCullough, and F. N. Finn, "Factors Affecting Performance of Pavement Systems," Transportation Engineering Journal, Vol 95, No. TE3, American Society of Civil Engineers, August 1969.

43. Hudson, W. R., F. N. Finn, B. F. McCullough, K. Nair, and B. A. Vallerga, "Systems Approach to Pavement Design, Systems Formulation, Performance Definition and Materials Characterization," Final Report, NCHRP Project 1-10, Materials Research and Development, Inc., March 1968.

44. Hudson, W. R., R. K. Kher, and B. F. McCullough, "Automation in Pavement Design and Management Systems," Special Report 128, Highway Research Board, 1972.

45. Hudson, W. R., "Relating Distress to Pavement Performance," Group 1, Special Report 126, Highway Research Board, 1971.

46. Huculak, N. A., "Quality Control of Asphalt Pavement Construction," Proceedings of the Association of Asphalt Paving Technologists, Vol 37, 1968.

47. Hutchinson, B. G., and R. C. G. Haas, "A Systems Analysis of the Highway Pavement Design Process," Highway Research Record No. 239, Highway Research Board, 1968.

48. Hutchinson, B. G., "A Conceptual Framework for Pavement Design Decisions," Highway Research Record No. 121, Highway Research Board, 1966, pp 1-14.

49. Jain, S. P., B. F. McCullough, and W. R. Hudson, "Flexible Pavement System - Second Generation Incorporating Fatigue and Stochastic Concepts," Research Report 123-10 (preliminary), published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, December 1971.

50. Kelley, James A., "Quality Assurance in Highway Construction, Part 5 - Summary of Research for Quality Assurance of Aggregate," Public Roads, Vol 35, No. 10, October 1969.

51. Kersten, M. S., and E. L. Skok, Jr., "Application of AASHO Road Test Results to Design of Flexible Pavements in Minnesota," Interim Report, University of Minnesota, 1968.

52. Keyser, J. H., and P. I. Waell, "Montreal's Experience in the Application of Statistics in the Specification and Control of Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 37, 1968.

53. Kher, R. K., and M. I. Darter, "Probabilistic Concepts and Their Applications to AASHO Interim Guide for Design of Rigid Pavement," a paper prepared for presentation at the 52nd Annual Meeting of the Highway Research Board, January 1973.

54. Kher, R. K., B. F. McCullough, and W. R. Hudson, "A Sensitivity Analysis of Flexible Pavement System FPS2," Research Report No. 123-8, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, August 1971.

55. Kher, R. K., W. R. Hudson, and B. F. McCullough, "A Comprehensive Systems Analysis for Rigid Pavements," Highway Research Record No. 362, Highway Research Board, pp 9-20, 1971.

56. Kher, R. K., W. R. Hudson, and B. F. McCullough, "A Systems Analysis of Rigid Pavement Design," Research Report No. 123-5, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, November 1970.

57. Kilpatrick, M. J., and R. C. Mequate, "Bituminous Pavement Construction," U. S. Department of Transportation, Federal Highway Administration, BPR, June 1967.

58. Lefevre, D. M., "Analysis of Traffic Load Characteristics Applicable to Pavement Design," Utah Highway Department, 1969.

59. Lemer, A. C., and F. Moavenzadeh, "Reliability of Highway Pavements," Highway Research Record No. 362, Highway Research Board, 1971.

60. Lemer, A. C., and Fred Moavenzadeh, "The Analysis of Highway Pavement Systems," Highway Research Record No. 337, Highway Research Board, 1970.

61. Levey, J. R., and E. J. Barenberg, "A Procedure for Evaluating Pavements With Non-uniform Paving Materials," Highway Research Record No. 337, Highway Research Board, 1970.

62. Lloyd, D. K., and M. Lipow, Reliability/Management, Methods, and Mathematics, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1962.

63. Liddle, W. J., "Evaluation of Pavement Serviceability on Utah Highways," Interim Report, Utah State Highway Department, 1969.

64. McCullough, B. F., "Distress Mechanisms - General," Special Report 126, Highway Research Board, 1971, pp 77-85.

65. McCullough, B. F., C. J. Van Til, B. A. Vallerga, and R. G. Hicks, "Evaluation of AASHO Interim Guides for Design of Pavement Structures," Final Report, NCHRP Project 1-11, December 1968.

66. McMahon, Thurmul F., and Wesley M. Baker, "Quality Assurance in Highway Construction, Part 3 - Quality Assurance of Portland Cement Concrete," Public Roads, Vol 35, No. 8, June 1969.

67. McMahon, Thurmul F., and Woodrow J. Halstead, "Quality Assurance in Highway Construction, Part 1 - Introduction and Concepts," Public Roads, Vol 35, No. 6, February 1969.

68. McMahon, Thurmul F., "Quality Assurance in Highway Construction, Part 2 - Quality Assurance of Embankments and Base Courses," Public Roads, Vol 35, No. 7, April 1969.

69. Moavenzadeh, F., and A. C. Lemer, "An Integrated Approach to the Design and Analysis of Highway Pavement," Report R68-S8, Massachusetts Institute of Technology, Department of Civil Engineering, Cambridge, 1968.

70. Moavenzadeh, Fred, "Damage and Distress in Highway Pavements," Special Report 126, Highway Research Board, 1971, pp 114-139.

71. Moore, R. K., and T. W. Kennedy, "Tensile Behavior of Stabilized Subbase Materials Under Repetitive Loadings," Research Report No. 98-12, Center for Highway Research, The University of Texas at Austin, 1971.

72. Nair, K., "Solutions and Solution Techniques for Boundary Value Problems," Special Report 126, Highway Research Board, 1971, pp 103-113.

73. Oglesby, C. H., and L. I. Hewes, Highway Engineering, John Wiley and Sons, Inc., Second Edition, 1961.

74. Orellana, H. E., "Flexible Pavement Design System Computer Program Documentation - FPS-11," Research Report 123-15, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, November 1972.

75. Pell, P. S., and I. F. Taylor, "Asphaltic Road Materials in Fatigue," Proceedings of the Association of Asphalt Paving Technologists, Vol 38, 1969, pp 371-422.

76. Peterson, D. E., W. R. Delis, D. Mangrum, and C. C. Sy, "Utah's System for Planning Roadway Improvements," Materials and Tests Division, Utah State Highway Department, 1971

77. Phang, W. A., and R. Slocum, "Pavement Investment Decision-Making and Management Systems," a paper presented at the 51st Annual Meeting of the Highway Research Board, January 1972.

78. Pister, K. S., "Some Remarks on Research for Structural Design of Asphalt Concrete Systems," Special Report 126, Highway Research Board, 1971, pp 77-85.

79. Poehl, R., and F. H. Scrivner, "Seasonal Variations of Pavement Deflections in Texas," Research Report No. 136-1, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1971.

80. Schiffman, Robert L., "Solutions to Boundary Value Problems," Special Report 126, Highway Research Board, 1971.

81. Scrivner, F. H., and W. M. Moore, "An Empirical Equation for Predicting Pavement Deflections," Research Report No. 32-12, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1968.

82. Scrivner, F. H., and W. M. Moore, "Flexible Pavement Performance Related to Deflections, Axle Applications, Temperature, and Foundation Movements," Research Report No. 32-13, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1969.

83. Scrivner, F. H., "A Theory for Transforming the AASHO Road Test Pavement Performance Equations Involving Mixed Traffic," Special Report 66, Highway Research Board, 1961.

84. Scrivner, F. H., C. H. Michalak, and W. M. Moore, "Calculation of the Elastic Moduli of a Two-Layer Pavement System from Measured Surface Deflections," Research Report 123-6, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, 1971.

85. Scrivner, F. H., G. Swift, and W. M. Moore, "A New Research Tool for Measuring Pavement Deflection," Highway Research Record No. 129, Highway Research Board, 1966.

86. Scrivner, F. H., W. M. Moore, W. F. McFarland, and G. R. Carey, "A Systems Approach to the Flexible Pavement Design Problem," Research Report No. 32-11, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1968.

87. Sexsmith, R. G., and M. F. Nelson, "Limitations in Application of Probabilistic Concepts," Proceedings of the American Concrete Institute, October 1969.

88. Shah, H. C., "The Rational Probabilistic Code Format," Proceedings of the American Concrete Institute, September 1969.

89. Sherman, G. B., R. O. Watkins, and R. Prysoek, "A Statistical Analysis of Embankment Compaction," California Division of Highways, May 1966.

90. Sherman, G. B., and R. O. Watkins, "A Statistical Analysis of Untreated Base and Subbase Materials," California Division of Highways, March 1967.

91. Sherman, G. B., "In Situ Materials Variability," Special Report 126, Highway Research Board, 1971, pp 180-190.

92. Shook, J. F., and T. Y. Lepp, "A Method for Calculating Equivalent 18-kip Load Applications," Highway Research Record No. 362, Highway Research Board, 1971.

93. Skok, E. L., Jr., "Load and Environmental Variables," Special Report 126, Highway Research Board, 1971.

94. Snedcor, G. W., and W. G. Cochran, "Statistical Methods," Sixth Edition, Iowa State University Press, 1967.

95. Steitle, D. C., and B. F. McCullough, "Skid Resistance Considerations in the Flexible Pavement Design System," Research Report No. 123-9 (preliminary), published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, 1972.

96. Strom, O. G., W. R. Hudson, and J. L. Brown, "A Pavement Feedback Data System," Research Report No. 123-12, published jointly by the Texas Highway Department; Texas Transportation Institute, Texas A&M University; and the Center for Highway Research, The University of Texas at Austin, 1972.

97. Task Committee on Structural Safety, A Literature Review, Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol 98, No. ST4, April 1962.

98. "The AASHO Road Test, Report 5 - Pavement Research," Special Report 61E, Highway Research Board, 1962.

99. "Traffic-Induced Fracture Committee Report," Special Report 126, Highway Research Board, 1971, pp 42-45.

100. "Unexpected Traffic Volume Cuts Pavement Life," Engineering News Record, Vol 18B, No. 20, May 20, 1971, p 25.

101. Walker, R. S., W. R. Hudson, and F. L. Roberts, "Development of a System for High Speed Measurement of Pavement Roughness," Final Report, Research Report No. 73-5F, Center for Highway Research, The University of Texas at Austin, 1970.

102. Welborn, J. York, "Asphalt Hardening - Fact and Fallacy," Public Roads, Vol 35, No. 12, February 1970, pp 279-285.

103. Westmann, R. A., "Committee on Mechanics of Earth Masses and Layered Systems," Special Report 126, Highway Research Board, 1971, pp 197-198.

APPENDIX 1

VARIANCE MODEL OF SCI

APPENDIX 1

THIS APPENDIX SUMMARIZES THE VARIANCE MODEL OF SCI FOR A ONE, TWO, AND THREE-LAYER PAVEMENT AND SUBGRADE SYSTEM. THE COMPLETE MODELS FOR THE PREDICTION OF SCI FOR A ONE, TWO, AND THREE-LAYER PAVEMENT ARE ALSO GIVEN

DEFINITION OF SYMBOLS

A1,A2,A3,A4 = STRENGTH COEFFICIENTS
VA1,VA2,VA3,VA4 = STANDARD DEVIATION OF A1,A2,A3,A4
D1,D2,D3, = PAVEMENT LAYER THICKNESS
VD1,VD2,VD3 = STANDARD DEVIATION OF D1,D2,D3
VVA1,VVA2,VVA3,VVA4,VVD1,VVD2,VVD3 = COMPONENTS OF VARIANCE OF SCI ASSOCIATED WITH A1,A2,A3,A4,D1,D2,D3
SCI= SURFACE CURVATURE INDEX
V(SCI)= VARIENCE OF SCI OF PAVEMENT / SUBGRADE SYSTEM

CONSTANTS

```
C0 = .891087
C1 = 4.50292
C2 = 6.25
R1 = 10.00
R2 = 15.62
```

THE MEAN SCI FOR ONE, TWO, AND THREE-LAYER PAVEMENT ARE AS FOLLOWS

ONE-LAYER PAVEMENT

```
G1 =   A1**C1*R1**2
G2 =   (A1**C1*R1**2) + (A1**C1*C2*(A1*D1)**2)
G3 =   (A2**C1*R1**2) + (A2**C1*C2*(A1*D1)**2)
G11=   A1**C1*R2**2
G22=   (A1**C1*R2**2) + (A1**C1*C2*(A1*D1)**2)
G33=   (A2**C1*R2**2) + (A2**C1*C2*(A1*D1)**2)
SCI1    = C0/G1 - C0/G2 + C0/G3 - C0/G11 + C0/G22 - C0/G33
```

TWO-LAYER PAVEMENT

```
G4 =   (A2**C1*R1**2) + (A2**C1*C2*(A1*D1 + A2*D2)**2)
G5 =   (A3**C1*R1**2) + (A3**C1*C2*(A1*D1 + A2*D2)**2)
G44=   (A2**C1*R2**2) + (A2**C1*C2*(A1*D1 + A2*D2)**2)
G55=   (A3**C1*R2**2) + (A3**C1*C2*(A1*D1 + A2*D2)**2)
SCI2    = SCI1   + C0/G5 - C0/G4 - C0/G55 + C0/G44
```

THREE-LAYER PAVEMENT

```
G6 =   (A3**C1*R1**2) + (A3**C1*C2*(A1*D1 + A2*D2 + A3*D3)**2)
G7 =   (A4**C1*R1**2) +  (A4**C1*C2*(A1*D1 + A2*D2 + A3*D3)**2)
G66=   (A3**C1*R2**2) + (A3**C1*C2*(A1*D1 + A2*D2 + A3*D3)**2)
G77=   (A4**C1*R2**2) +  (A4**C1*C2*(A1*D1 + A2*D2 + A3*D3)**2)
SCI3    = SCI2   - C0/G6 + C0/G7 + C0/G66 - C0/G77
```

THE VARIANCE MODELS FOR SCI FOR ONE, TWO, AND THREE-LAYER PAVEMENT ARE AS FOLLOWS

ONE-LAYER PAVEMENT

```
A110 = C1*R1**(-2)*A1**(-C1-1.)
A120 = C1*A1**(C1-1.)*R1**2 + (C1+2.)*A1**(C1+1.)*C2*D1**2
A130 = A2**C1*C2*2.*A1*D1**2
A111 = C1*R2**(-2)*A1**(-C1-1.)
A122 = C1*A1**(C1-1.)*R2**2 + (C1+2.)*A1**(C1+1.)*C2*D1**2
A230 = C1*A2**(C1-1.)*R1**2 +C1*A2**(C1-1.)*C2*(A1*D1)**2
A233 = C1*A2**(C1-1.)*R2**2 +C1*A2**(C1-1.)*C2*(A1*D1)**2
D120 = 2.*A1**C1*C2*A1**2*D1
D130 = 2.*A2**C1*C2*A1**2*D1
CVA1 =  -C0*A110 +C0*G2**(-2)*A120 -C0*G3**(-2)*A130
     +C0*A111-C0*G22**(-2)*A122+C0*G33**(-2)*A130
CVA2 = -C0*G3**(-2)*A230 + C0*G33**(-2)*A233
CVD1 = C0 *G2**(-2)*D120 - C0*G3**(-2)*D130 - C0*G22**(-2)*D120
     + C0*G33**(-2)*D130

VVA1 = (CVA1**2) * VA1 **2
VVA2 =(CVA2**2) * VA2**2
VVD1 = (CVD1**2)* VD1 **2
V(SCI1) = VVA1 + VVA2 + VVD1
```

TWO-LAYER PAVEMENT

```
A140 = 2.*A1*D1**2*A2**C1*C2+2.*A2*D1*D2*A2**C1*C2
A150 = 2.*A1*D1**2*A3**C1*C2+2.*A2*D1*D2*A3**C1*C2
A240 = C1*A2**(C1-1.)*R1**2 + C1*A2**(C1-1.)*C2*A1**2*D1**2
                            + (C1+1.)*A2**C1*C2*2.*A1*D1*D2 +(C1
     +2.)*A2**(C1+1.)*C2*D2**2
A250 = A3**C1*C2*(2.*A1*D1*D2 +2.*A2*D2**2)
A244 = C1*A2**(C1-1.)*R2**2+ C1*A2**(C1-1.)*C2*A1**2*D1**2
                            + (C1+1.)*A2**C1*C2*2.*A1*D1*D2 +(C1+2.)
     *A2**(C1+1.)*C2*D2**2
A350 = C1*A3**(C1-1.)*(R1**2+C2*A1**2*D1**2 +2.*C2*A1*D1*A2*D2
     +C2*A2**2 *D2**2)
A355 = C1*A3**(C1-1.)*(R2**2+C2*A1**2*D1**2 +2.*C2*A1*D1*A2*D2
     +C2*A2**2*D2**2)
D140 =2.*D1*A2**C1*C2*A1**2+2.*A1*A2*D2*A2**C1*C2
D150 =2.*A3**C1*C2*A1**2*D1+2.*A3**C1*C2*A1*A2*D2
D240 =2.*A2**C1*C2*A1*D1*A2+2.*A2**C1*C2*A2**2*D2
D250 =2.*A3**C1*C2*A1*D1*A2+2.*A3**C1*C2*A2**2*D2
CVA1 = CVA1 + C0*G4**(-2) * A140 - C0*G5**(-2) * A150 - C0*G44
     **(-2) * A140 + C0*G55**(-2) * A150
CVA2 = CVA2 + C0*G4**(-2)*A240 - C0*G5**(-2)*A250 - C0*G44**(-2)
     *A244 + C0*G55**(-2)*A250
CVA3 = CVA3 - C0*G5**(-2)*A350 + C0*G55**(-2)*A355
CVD1 = C0*G4**(-2)*D140 -C0*G5**(-2)*D150 +CVD1
   -C0*G44**(-2)*D140 + C0*G55**(-2)*D150
CVD2 = C0*G4**(-2)*D240-C0*G5**(-2)*D250 - C0*G44**(-2)*D240
     + C0*G55**(-2) *D250

VVA1 = (CVA1**2) * VA1 **2
VVA2 =(CVA2**2) * VA2**2
VVA3 = (CVA3**2) * VA3**2
VVD1 = (CVD1**2)* VD1 **2
VVD2 = (CVD2**2)*VD2**2

V(SCI2) = VVA1 + VVA2 + VVA3 + VVD1 + VVD2
```

THREE-LAYER PAVEMENT

```
A160 = A3**C1*C2*(2.*A1*D1**2+2.*D1*A2*D2 + 2.*D1*A3*D3)
A170 = A4**C1*C2*(2.*A1*D1**2+2.*D1*A2*D2 + 2.*D1*A3*D3)
A260 = A3**C1*C2*(2.*A2*D2**2 + 2.*A1*D1*D2 +2.*D2*A3*D3)
A270 = A4**C1*C2*(2.*A2*D2**2+2.*A1*D1*D2+2.*D2*A3*D3)
A360 = C1*A3**(C1-1.)*(R1**2+C2*A1**2*D1**2 +2.*C2*A1*D1*A2*D2+C2*
       A2**2*D2**2) +(C1+2.)*A3**(C1+1.)*C2*D3**2
       +2.*(C1+1.)*A3**C1*C2*(A1*D1*D3+A2*D2*D3)
A370 = 2.*A3*D3**2*A4**C1*C2 +2.*A2*D2*D3*A4**C1*C2 +2.*A1*D1
       *D3*A4**C1*C2
A366 = C1*A3**(C1-1.)*(R2**2+C2*A1**2*D1**2+2.*C2*A1*D1*A2*D2 +C2
       *A2**2*D2**2)+(C1+2.)*A3**(C1+1.)*C2*D3**2
       +2.*(C1+1.)*A3**C1*C2*(A1*D1*D3+A2*D2*D3)
A470 = C1*A4**(C1-1.)*(R1**2+C2*(A1**2*D1**2+A2**2*D2**2 +A3**2*D3
       **2+2.*A1*D1*A2*D2 +2.*A2*D2*A3*D3+2.*A1*D1*A3*D3))
A477 = C1*A4**(C1-1.)*(R2**2+C2*(A1**2*D1**2+A2**2*D2**2 +A3**2*D3
       **2+2.*A1*D1*A2*D2 +2.*A2*D2*A3*D3+2.*A1*D1*A3*D3))
D160 =2.*A3**C1*C2*A1**2*D1+2.*A3**C1*C2*A1*A2*D2+2.*A3**(C1+1.)*C
       2*A1*D3
D170 =2.*A4**C1*C2*A1**2*D1+2.*A4**C1*C2*A1*A2*D2+2.*A4**C1*C2*A1*
       A3*D3
D260 =2.*A3**C1*C2*A2**2*D2+2.*A3**C1*C2*A1*D1*A2+2.*A3**(C1+1.)*
       C2*A2*D3
D270 =2.*A4**C1*C2*A2**2*D2+2.*A4**C1*C2*A1*D1*A2+2.*A4**(C1)*C2
       *A2*A3*D3
D360 =2.*A3**(C1+2.)*C2*D3+2.*A3**C1*C2*A2*D2*A3+2.*A3**(C1+1.)*A1
       *D1*C2
D370 =2.*A4**C1*C2*A3**2*D3+2.*A4**C1*C2*A2*D2*A3+2.*A4**C1*C2*
       D1*A1*A3


CVA1 = CVA1 + C0*G6**(-2)*A160 - C0*G7**(-2)*A170 - C0*G66**(-2)
     *A160 + C0*G77**(-2)*A170
CVA2 = CVA2 +C0*G6**(-2)*A260 - C0*G7**(-2)*A270 - C0*G66**(-2)*
     A260 + C0*G77**(-2)*A270
CVA3 = CVA3 + C0*G6**(-2)*A360 - C0*G7**(-2)*A370 - C0*G66**(-2)
     *A366 + C0*G77**(-2)*A370
CVA4 =   -C0*G7**(-2)*A470 + C0*G77**(-2)*A477
CVD1 = CVD1 + C0*G6**(-2)*D160 - C0*G7**(-2)*D170
            - C0*G66**(-2)*D160 + C0*G77**(-2)*D170
CVD2 = CVD2 + C0*G6**(-2)*D260 - C0*G7**(-2)*D270 - C0*G66**(-2)
     *D260 + C0*G77**(-2)*D270
CVD3 =    C0*G6**(-2)*D360 -C0*G7**(-2)*D370 -C0*G66**(-2)*D360 +
       C0*G77**(-2)*D370

VVA1 = (CVA1**2) * VA1 **2
VVA2 =(CVA2**2) * VA2**2
VVA3 = (CVA3**2) * VA3**2
VVA4 = (CVA4**2)*VA4**2
VVD1 = (CVD1**2)* VD1 **2
VVD2 = (CVD2**2)*VD2**2
VVD3 = (CVD3**2)*VD3**2

V(SCI3) = VVA1 + VVA2 + VVA3 + VVA4 + VVD1 + VVD2 + VVD3
```

APPENDIX 2

SUMMARY OF INPUT/OUTPUT DATA FOR 12 PROJECTS

USING FPS-11 PROGRAM

APPENDIX 2. SUMMARY OF INPUT/OUTPUT DATA FOR 12 PROJECTS USING FPS-11 PROGRAM

Design Inputs	Projects					
	FM 2625	FM 1840	US 70 & 84	SH 300	SH 24	US 84
Length of Analysis period	10.0	10.0	20.0	20.0	20.0	20.0
Minimum time to first overlay (years)	10.0	10.0	6.0	8.0	5.0	6.0
Minimum time between overlays (years)	10.0	10.0	6.0	8.0	5.0	6.0
Design confidence level	C	C	C	D	D	D
Interest rate or time value of money (percent)	7.0	7.0	7.0	7.0	7.0	7.0
Minimum serviceability index - P2	2.5	2.5	3.0	3.0	3.0	3.0
Maximum funds available per square yard for initial design (dollars)	2.25	1.75	7.00	4.00	7.50	7.00
Maximum allowed thickness of initial construction (inches)	20.0	22.0	38.0	28.0	24.0	38.0
Accumulated maximum depth of all overlays (inches)	0.0	0.0	6.0	5.0	4.5	6.0
One-direction ADT at beginning of analysis period (vehicles/day)	135	730	955	1,100	1,600	1,670
One-direction ADT at end of 20 years (vehicles/day)	260	1,215	1,950	1,850	3,200	2,750
One-direction 20-year accumulated number of equivalent 18-kip axles	30,000	289,000	658,000	637,400	8,800,000	1,069,000
Average approach speed to overlay zone (MPH)	0.0	0.0	60.0	70.0	60.0	60.0
Average speed through overlay zone (O.D.) (MPH)	0.0	0.0	35.0	30.0	20.0	35.0

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	FM 2625	FM 1840	US 70 & 84	SH 300	SH 24	US 84
Average speed through overlay zone (N.O.D.) (MPH)	0.0	0.0	50.0	30.0	30.0	50.0
Proportion of ADT arriving each hour of construction (percent)	5.0	6.0	6.0	6.0	5.0	6.0
The road is in a rural/urban area	Rural	Rural	Rural	Rural	Rural	Rural
District temperature constant	25.0	25.0	16.0	25.0	22.0	16.0
Swelling probability	0.0	1.00	0.0	0.05	0.0	0.0
Potential vertical rise (inches)	0.0	1.50	0.0	3.00	0.0	0.0
Swelling rate constant	0.0	0.08	0.0	0.02	0.0	0.0
Subgrade stiffness constant	0.26	0.25	0.25	0.28	0.28	0.28
Serviceability index of the initial structure	3.8	3.8	4.3	4.2	4.2	4.3
Serviceability index P1 after an overlay	4.2	4.2	4.2	4.2	4.0	4.2
Minimum overlay thickness (inches)	0.0	0.0	0.5	0.8	0.5	0.5
Overlay construction time (hours/day)	0.0	0.0	10.0	10.0	10.0	10.0
Asphaltic concrete compacted density (tons/C.Y.)	1.98	0.0	1.82	1.98	2.0	1.82
Asphaltic concrete production rate (tons/hour)	80.0	0.0	120.0	80.0	80.0	120.0
Width of each lane (feet)	10.0	12.0	12.0	12.0	12.0	12.0
First year cost of routine maintenance (dollars/lane-mile)	50.00	10.00	50.00	50.00	50.00	50.00

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	FM 2625	FM 1840	Us 70 & 84	Sh 300	SH 24	US 84
Annual incremental increase in maintenance cost (dollars/lane-mile)	20.00	15.00	20.00	10.00	20.00	20.00
Traffic model used during overlaying	2	0	3	2	1	3
Number of open lanes in restricted zone (overlay direction)	1	0	1	1	1	1
Number of open lanes in restricted zone (non-overlay direction)	1	0	1	1	1	1
Distance traffic is slowed (overlay direction)(miles)	0.0	0.0	1.50	1.0	1.0	1.50
Distance traffic is slowed (non-overlay direction)(miles)	0.0	0.0	1.00	1.0	1.0	1.0
Detour distance around overlay zone (miles)	0.0	0.0	0.0	0.0	0.0	0.0
First layer material	1-course surface treatment	Surface treatment	ACP	ACP	ACP	ACP
Cost per cubic yard	14.88	14.88	16.60	21.78	16.00	16.60
Structural coefficient	0.46	0.55	0.96	0.96	0.95	0.96
Minimum depth	0.25	0.25	1.50	1.50	1.50	1.50
Maximum depth	0.25	0.25	4.00	3.00	1.50	4.00
Salvage percent	11.00	76.00	25.00	18.00	0.0	25.00

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	FM 2625	FM 1840	US 70 & 84	SH 300	SH 24	US 84
Second Layer Material	Iron ore topsoil	Bank run gravel	Flexible base	I.O. & Del aggregate	Asphalt stabilized base	Flexible base
Cost per cubic yard	4.29	1.54	5.65	6.30	11.00	5.65
Structural coefficient	0.50	0.50	0.62	0.65	0.85	0.62
Minimum depth	4.00	8.00	6.00	6.00	3.00	6.00
Maximum depth	15.00	8.00	10.00	15.00	8.50	10.00
Salvage percent	59.00	192.00	75.00	57.00	0.00	75.00
Third layer material		Lime-treated material	Caliche	Iron ore topsoil		Caliche
Cost per cubic yard		4.28	5.65	2.50		5.65
Structural coefficient		0.40	0.62	0.45		0.62
Minimum depth		4.00	6.00	4.00		6.00
Maximum depth		8.00	30.00	8.00		30.00
Salvage percent		100.00	100.00	56.00		

(Continued)

APPENDIX 2. (Continued)

Design Outputs	Projects					
	FM 2625	FM 1840	US 70 & 84	SH 300	SH 24	US 84
Material arrangement	AB	ABF	ACG	ADH	AB	ACG
Initial construction cost	0.58	1.28	2.57	2.51	2.81	2.93
Overlay construction cost	0.0	0.0	0.0	0.0	0.26	0.0
User cost	0.0	0.0	0.0	0.0	0.02	0.0
Routine maintenance cost	0.17	0.07	0.32	0.20	0.24	0.32
Salvage value	-0.15	-0.80	-0.47	-0.28	0.00	-0.56
Total cost	0.60	0.55	2.42	2.43	3.32	2.68
Number of layers	2	3	3	3	2	3
Layer depth (inches)						
D (1)	0.25	0.25	1.50	1.50	1.50	1.50
D (2)	4.00	8.00	6.00	6.00	7.00	6.00
D (3)		7.00	6.00	8.00		8.25
D (4)						
Number of performance periods	1	1	1	1	2	1
Performance time (years)						
T (1)	10.1	10.4	20.4	21.9	13.7	21.2
T (2)					20.1	

(Continued)

APPENDIX 2. (Continued)

Design Outputs	Projects					
	FM 2625	FM 1840	US 70 & 84	SH 300	SH 24	US 84
Overlay policy inch (including level up)						
0 (1)					1.5	
Swelling clay loss						
SC (1)	0.0	0.28	0.0	0.05	0.0	0.0
SC (2)					0.0	

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Length of analysis period	20.0	20.0	20.0	20.0	20.0	20.0
Minimum time to first overlay (years)	8.0	5.0	4.0	6.0	5.0	4.0
Minimum time between overlays (years)	8.0	6.0	6.0	6.0	5.0	6.0
Design confidence level	D	D	D	E	E	E
Interest rate or time value of money (percent)	7.0	6.5	7.0	7.0	7.0	7.0
Minimum serviceability index - P2	3.0	3.0	3.0	3.0	3.0	3.0
Maximum funds available per square yard for initial design (dollars)	4.00	4.0	9.99	7.00	9.99	8.00
Maximum allowed thickness of initial construction (inches)	28.00	30.0	36.0	38.0	30.0	36.0
Accumulated maximum depth of all overlays (inches)	4.0	4.0	8.0	6.0	4.5	6.0
One-direction ADT at beginning of analysis period (vehicles/day)	1,350	2,800	2,130	2,725	6,800	19,665
One-direction ADT at end of 20-years (vehicles/day)	2,150	4,900	6,200	16,400	15,100	32,376
One-direction 20-year accumulated number of equivalent 18-kip axles	1,450,000	1,562,000	3,661,000	2,840,000	4,657,000	6,894,000
Average approach speed to overlay zone (MPH)	70.0	60.0	60.0	60.0	60.0	50.0
Average speed through overlay zone (O.D.)(MPH)	30.0	40.0	40.0	35.0	30.0	20.0

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Average speed through overlay zone (N.O.D.)(MPH)	50.0	50.0	50.0	50.0	60.0	50.0
Proportion of ADT arriving each hour of construction (percent)	6.0	7.0	7.0	6.0	6.0	5.5
The road is in a rural/urban area	Rural	Rural	Rural	Rural	Rural	Urban
District temperature constant	25.0	31.0	30.0	16.0	22.0	31.0
Swelling probability	0.0	0.25	1.00	0.0	1.0	0.85
Potential vertical rise (inches)	0.0	2.00	5.00	0.0	4.0	5.0
Swelling rate constant	0.0	0.08	0.15	0.0	0.1	0.08
Subgrade stiffness constant	0.30	0.25	0.24	0.28	0.25	0.26
Serviceability index of the initial structure	4.2	4.2	4.2	4.3	4.2	4.0
Serviceability index PI after an overlay	4.2	4.4	4.2	4.2	4.0	3.9
Minimum overlay thickness (inches)	0.5	0.5	0.3	0.5	0.5	0.8
Overlay construction time (hours/day)	10.0	10.0	10.0	10.0	10.0	7.0
Asphaltic concrete compacted density (tons/cubic yard)	1.98	2.00	1.80	1.82	2.0	1.26
Asphaltic concrete production rate (ton/hour)	75.0	100.0	90.0	120.0	80.0	75.0
Width of each lane (feet)	12.0	12.0	12.0	12.0	12.0	12.0
First year cost of routine maintenance (dollars/lane-mile)	50.0	50.0	50.0	50.0	50.0	100.0

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Annual incremental increase in maintenance cost (dollars/lane-mile)	20.0	20.0	20.0	20.0	20.0	10.0
Traffic model used during overlaying	3	3	3	3	3	3
Number of open lanes in restricted zone (O.D.)	1	1	1	1	1	1
Number of open lanes in restricted zone (N.O.D.)	2	2	2	2	2	3
Distance traffic is slowed (overlay direction)(miles)	1.00	1.00	2.00	1.50	1.0	1.0
Distance traffic is slowed (non-overlay direction)(miles)	0.10	0.50	0.20	1.00	0.0	0.0
Detour distance around overlay zone (miles)	0.0	0.0	0.0	0.0	0.0	0.0
First layer material	ACP	ACP	ACP	ACP	ACP	Lt. Wt. ACP
Cost per cubic yard	21.80	16.00	13.99	16.60	14.00	21.42
Structural coefficient	0.96	0.96	0.96	0.96	0.85	0.96
Minimum depth	1.50	1.50	1.50	1.50	4.50	1.00
Maximum depth	4.00	2.50	1.50	1.50	10.00	1.00
Salvage percent	10.00	25.00	20.00	25.00	0.0	10.00

(Continued)

APPENDIX 2. (Continued)

Design Inputs	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Second layer material	IOTS	Crushed stone	Asphalt stabilized base	Flexible base	Stabilized flexible base	ACP
Cost per cubic yard	5.54	5.10	15.46	5.65	6.00	15.48
Structural coefficient	0.46	0.60	0.90	0.62	0.60	0.96
Minimum depth	6.00	4.00	5.00	6.00	8.00	1.50
Maximum depth	18.00	10.00	8.00	9.00	20.0	1.50
Salvage percent	50.00	70.00	15.00	75.00	0.0	10.00
Third layer material		Flint gravel	Crushed limestone	Caliche		Black base
Cost per cubic yard		2.50	7.13	5.65		13.93
Structural coefficient		0.55	0.60	0.62		0.96
Minimum depth		4.00	5.00	4.00		2.50
Maximum depth		6.00	10.00	6.00		10.00
Salvage percent		100.00	75.00	100.00		30.00

(Continued)

APPENDIX 2. (Continued)

Design Inputs	US 271	SH 71	US 290	Loop 289	Sh 360	Loop 1
	Projects					
Fourth layer material						Crushed stone
Cost per cubic yard						4.40
Structural coefficient						0.60
Minimum depth						10.00
Maximum depth						18.00
Salvage percent						80.00
Fifth layer material						Lime-treated subgrade
Cost per cubic yard						2.40
Structural coefficient						0.40
Minimum depth						6.00
Maxiuum depth						6.00
Salvage percent						100.00

(Continued)

APPENDIX 2. (Continued)

Design Output	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Material arrangement	AB	ABC	ABC	ACG	AB	ABCDE
Initial construction cost	3.96	2.39	4.71	3.44	5.43	5.13
Overlay construction cost	0.40	0.29	0.55	1.13	0.57	0.91
User cost	0.01	0.01	0.03	0.09	0.10	0.15
Routine maintenance cost	0.22	0.24	.16	0.20	0.17	0.22
Salvage value	-0.29	-0.44	-0.51	-0.80	0.00	-0.71
Total cost	4.31	2.50	4.94	4.06	6.27	5.71
Number of layers	2	3	3	3	2	5
Layer depth (inches)						
D (1)	3.75	1.50	1.50	1.50	6.25	1.00
D (2)	11.00	9.25	5.00	6.00	18.00	1.50
D (3)		6.00	10.00	11.50		3.50
D (4)						17.50
D (5)						6.00
Number of performance periods	2	2	3	2	3	2

(Continued)

APPENDIX 2. (Continued)

Design Output	Projects					
	US 271	SH 71	US 290	Loop 289	SH 360	Loop 1
Performance time (years)						
T (1)	12.1	12.6	5.5	9.5	8.0	9.1
T (2)	20.3	21.6	13.1	20.0	14.2	21.0
T (3)			20.8		20.2	
T (4)						
Overlay policy (inch)(including level-up)						
O (1)	1.5	1.5	1.3	4.5	1.5	2.8
O (2)			1.3		1.5	
O (3)						
Swelling clay loss						
SC (1)	0.0	0.11	0.94	0.0	0.74	0.73
SC (2)	0.0	0.03	0.50	0.0	0.27	0.42
SC (3)			0.16		0.15	
SC (4)						

APPENDIX 3

INPUT DATA GUIDE TO FPS-13(CFHR)

FPS13 (CFHR) - GUIDE FOR DATA INPUT

PROJECT IDENTIFICATION

PROB	D	COUNTY	CONT	S	HWY	DATE	IPE	NCOM
A3	A2	3A4, A2	A4	A2	2A4, A2	2A4	A4	I3

1 3 5 19 23 25 35 43 47 50

PROB - Problem identification or number

D - District number

COUNTY - County name

CONT - Control

S - Section

HWY - Highway

DATE - Date of construction

IPE - Investigation and planning expense number

NCOM - Number of comment cards ($0 \leq NCOM \leq 7$)

PROJECT COMMENTS (NCOM CARDS)

20A4

1 80

Project Comments - Space provided so the designer may include the most relevant information concerning the project.

FPS13 (CFHR) - GUIDE FOR DATA INPUT

BASIC DESIGN CRITERIA

CL	XTTO	XTBO	P2	PLEVEL	PCTRAT
F5.2	F5.2	F5.2	F5.2	A1	F5.2
1 5	10	15	20	21	26

CL - Length of the analysis period, in years

XTTO - Minimum allowed time to the first overlay

XTBO - Minimum allowed time between overlays

P2 - Minimum allowed value of the serviceability index (point at which an overlay must be applied)

PLEVEL - Alphabetic character used to determine confidence level

PLEVEL = A corresponds to confidence level of 50 percent
PLEVEL = B corresponds to confidence level of 80 percent
PLEVEL = C corresponds to confidence level of 95 percent
PLEVEL = D corresponds to confidence level of 99 percent
PLEVEL = E corresponds to confidence level of 99.9 percent
PLEVEL = F corresponds to confidence level of 99.99 percent
PLEVEL = G corresponds to confidence level of 99.999 percent

PCTRAT - Interest rate or time value of money expressed as percentage

FPS13 (CFHR) - GUIDE FOR DATA INPUT

PROGRAM CONTROLS AND CONSTRAINTS

IPTYPE	NMB	NM	CMAX	TMAXIN	OMAXIN
I5	I5	I5	F5.0	F5.0	F5.0

1 5 10 15 20 25 30

IPTYPE = 1 for a new pavement construction

= 2 for an ACP overlay

NMB - Number of output pages for the summary table (eight designs per page)

NM - Number of materials (not including the subgrade)

CMAX - Maximum cost per square yard allowed for initial construction

TMAXIN - Maximum allowed total thickness of initial construction

OMAXIN - Accumulated maximum thickness of all overlays

TRAFFIC VARIABLES

RB	RE	XN20	AAS	ASO	ASN	PROPCT	PTRUCK
F10.2	F10.2	F10.2	F5.2	F5.2	F5.2	F5.2	F5.2

1 10 20 30 35 40 45 50 55

RB - Average daily traffic at the beginning of the analysis period

RE - ADT at the end of 20 years

XN20 - 20-year accumulated 18-kip axle equivalencies

AAS - Average approach speed to the overlay area, assumed to be the same for both directions

ASO - Average speed through the overlay area in the overlay direction

ASN - Average speed through the overlay area in the non-overlay direction

FPS13 (CFHR) - GUIDE FOR DATA INPUT

PROPCT - Percent of ADT which will pass through the overlay zone during each hour while overlaying takes place

PTRUCK - Percentage of trucks in ADT

ENVIRONMENT AND SUBGRADE

ALPHA	PROBSW	PVR	SWRATE	SCOS	SSCOS
F5.2	F5.2	F5.2	F5.2	F5.2	F 5.3

1 5 10 15 20 25 30

ALPHA - District or regional temperature constant

PROBSW - Probability of swell

PVR - Potential vertical rise due to swelling clay (input in inches)

SWRATE - Swelling clay constant for the swelling rate

SCOS - Stiffness coefficient of the subgrade

SSCOS - Subgrade stiffness coefficient standard deviation

CONSTRUCTION PARAMETERS

PSI	P1	OMININ	HPD	ACCD	ACPR	XLW	CM1	CM2
F5.2	F5.2	F5.2	F5.2	F5.2	F5.2	F5.2	F6.2	F6.2

1 5 10 15 20 25 30 35 41 47

PSI - Serviceability index of the initial structure

P1 - Beginning serviceability index of the pavement after an overlay

OMININ - Minimum thickness of an individual overlay

HPD - Number of hours per day that overlay construction takes place

FPS13 (CFHR) - GUIDE FOR DATA INPUT

ACCD - Asphaltic concrete compacted density (tons per compacted cubic yard)

ACPR - Asphaltic concrete production rate (tons per hour)

XLW - Width of each lane (feet)

CM1 - Annual routine maintenance cost per lane mile for the first year after construction or an overlay

CM2 - Annual incremental increase in routine maintenance cost per lane mile

DETOUR DESIGN FOR OVERLAYS

MODEL	NLANES	NLRO	NLRN	XLSO	XLSN	XLSD
I2	I2	I2	I2	F5.2	F5.2	F5.2

1 2 4 6 8 13 18 23

MODEL - Model number which describes the traffic situation

NLANES - Total number of lanes in the facility

NLRO - Number of open lanes in the overlay direction in the restricted zone

NLRN - Number of open lanes in the non-overlay direction in the restricted zone

XLSO - Centerline distance over which traffic is slowed in the overlay direction

XLSN - Centerline distance over which traffic is slowed in the non-overlay direction

XLSD - Distance, measured along the detour, around the overlay zone

FPS13 (CFHR) - GUIDE FOR DATA INPUT

EXISTING PAVEMENT AND PROPOSED ACP (Provide for overlay design made only)

ACTLD	PO3	DIP	COSTI	PSVGE1	COST2	PSVGE2	FLU
F10.2	F5.2	F5.2	F5.2	F5.2	F5.2	F5.2	F5.2
1 - 10	15	20	25	30	35	40	45

ACTLD - One-direction accumulated number of equivalent 18-KSA that have passed over pavement since construction of last overlay

PO3 - Initial serviceability of original pavement

DIP - Composite thickness of the existing pavement (inches)

COSTI - In-place cost per compacted cubic yard of proposed ACP

PSVGE1 - Salvage value of proposed ACP at the end of analysis period

COST2 - In-place value of existing pavement (dollars per cubic yard)

PSVGE2 - Salvage value of existing pavement at end of analysis period (percent)

FLU - Level-up required for the first overlay (inches)

MATERIAL PARAMETERS (Do not include for overlay design)

ID	CODE	NAME	COST	STRENGTH	MINTCK	MAXTCK	SALVAGE
I1	A1	5A3, A5	F10.0	F10.0	F10.0	F10.0	F10.0
2	4	6 - 25	35	45	55	65	75

ID - Layer identification (the layer number in which material can be used)

CODE - Code letter of the material

NAME - Name of the type of the material

COST - In-place cost per compacted cubic yard

STRENGTH - Strength coefficient of the material

MINTCK - Minimum layer thickness allowed

MAXTCK - Maximum layer thickness allowed

SALVAGE - Salvage value percentage of the material

STANDARD DEVIATIONS AND VARIANCES

CADT	CT12	CAX	P1ST	AST	PLEST	EST	CVD	CVDO	
F5.2	F5.2	F5.2	F5.2	F5.2	F5.4	F10.4	F5.3	F5.3	

1 5 10 15 20 25 30 40 45 50

CADT - Design ADT coefficient of variation (percent)

CT12 - Percent trucks coefficient of variation (percent)

CAX - Axles per truck coefficient of variation (percent)

P1ST - Serviceability index of initial structure standard deviation

AST - District temperature constant standard deviation

PLEST - Variance of axle load/equivalency parameter

EST - Performance model lack-of-fit variance

CVD - Deflection model lack-of-fit coefficient of variation for new construction

CVDO - Deflection model lack-of-fit coefficient of variation for overlay

FPS13 (CFHR) - DATA GUIDE

EXISTING PAVEMENT DATA (provide only for overlay mode)

Data taken at .1-mile intervals, up to five SCI replicates per card

SI	SCI(1)	SCI(2)	SCI(3)	SCI(4)	SCI(5)
F5.2	F5.2	F5.2	F5.2	F5.2	F5.2
1 5	10	15	20	25	30

(As many cards of these as sections)

Blank card terminates existing pavement data.

THE AUTHORS

Michael I. Darter is an Assistant Professor of Civil Engineering at the University of Illinois. During the time this work was performed he was a research engineer at the Center for Highway Research at The University of Texas at Austin. His engineering experience includes seven years work with the Utah Highway Department and three years with the Center for Highway Research. He has also served as an assistant research professor at the University of Utah. His recent research work has been in applying and implementing probabilistic design concepts to pavement design systems for the Texas Highway Department. He is the author of several technical papers and is a member of several professional societies.

W. Ronald Hudson is a Professor of Civil Engineering and Director of Research, Council for Advanced Transportation Studies at The University of Texas at Austin. He has had a wide variety of experience as a research engineer with the Texas Highway Department and the Center for Highway Research at The University of Texas at Austin and was Assistant Chief of the Rigid Pavement Research Branch of the AASHO Road Test. He is the author of numerous publications and was the recipient of the 1967 ASCE J. James R. Croes Medal. He is presently concerned with research in the areas of (1) analysis and design of pavement management systems, (2) measurement of pavement roughness performance, (3) slab analysis and design, and (4) tensile strength of stabilized subbase materials.