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NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **332**

FRAMEWORK FOR DEVELOPMENT OF PERFORMANCE-RELATED SPECIFICATIONS FOR HOT-MIX ASPHALTIC CONCRETE

MATERIALS	ACT	INF
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Research Supv		
QC Supv		
Geotech		
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Proj Dev		
Pavement		✓ R
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REPLY TO: Robert J. Reilly
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TO: CHIEF ADMINISTRATIVE OFFICERS
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FROM: Thomas B. Deen
Executive Director

SUBJECT: National Cooperative Highway Research Program Report 332
"Framework for Development of Performance-Related Specifications for Hot-Mix Asphaltic Concrete"
Final Report on Project 10-26A of the FY '84 Program.

I am enclosing one copy of the final report resulting from research conducted by the Pennsylvania State University, University Park, Pennsylvania. In accordance with the selective distribution system of the Transportation Research Board, all persons who have selected the transportation modes and subject areas listed below will receive copies of this document.

The NCHRP staff has provided a foreword that succinctly summarizes the scope of the work and indicates the personnel who will find the results of particular interest. This will aid in the distribution of the report within your department and in practical application of the research findings on performance-related specifications for hot-mix asphaltic concrete.

Modes	Areas
Highway Transportation	Pavement Design and Performance
Air Transportation	Bituminous Materials and Mixes

Enclosure

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **332**

FRAMEWORK FOR DEVELOPMENT OF PERFORMANCE-RELATED SPECIFICATIONS FOR HOT-MIX ASPHALTIC CONCRETE

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The Pennsylvania Transportation Institute
The Pennsylvania State University
University Park, Pennsylvania

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST

Pavement Design and Performance
Bituminous Materials and Mixes
(Highway Transportation, Air Transportation)

TRANSPORTATION RESEARCH BOARD
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WASHINGTON, D.C.
DECEMBER 1990

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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

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

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

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

NCHRP REPORT 332

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation officials, or the Federal Highway Administration, U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

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FOREWORD

By Staff
Transportation Research Board

Many highway agencies are considering moving from statistically based, end-result specifications to performance-related specifications. When end-result specifications are used, randomly selected samples are tested, and payments to the contractor depend on the product meeting the stated specification limits. When performance-related specifications are used, price adjustments resulting from product variability under control of the contractor are assessed in proportion to the positive or negative effect of that variability on the performance of the pavement. This report will be of special interest to engineers involved in designing pavements or specifying their materials and construction procedures. The report contains concepts for performance-related specifications that focus on asphaltic concrete, but which may, in a broad sense, be applicable to any highway material.

There is increased interest among pavement engineers in performance-related specifications. This report presents a conceptual framework for developing performance-related specifications for hot-mix asphaltic concrete. The framework is based on the assumption that the potential performance of the pavement, as built by the contractor, can be related to appropriately selected materials and construction (M&C) variables. Further, to fully develop performance-related specifications based on this framework, there must be established a sequential relationship between the M&C variables and the fundamental mixture variables, such as stiffness; between those variables of the asphalt mixture and the fundamental mixture response variables, such as tensile strain; between the response variables and the pavement distress indicators, such as cracking or rutting; and eventually between the distress indicators and life cycle costs. The research was conducted under NCHRP Project 10-26A by the Pennsylvania Transportation Institute, Pennsylvania State University, under the direction of Dr. David A. Anderson, Principal Investigator.

Full implementation of performance-related specifications will depend on the results of recently completed and ongoing research efforts (conducted by the NCHRP and others) to improve design, materials, and construction of the nation's highway pavements. This report lays the groundwork for the development of these specifications. Some of the other research efforts are:

- Work under FHWA Contract DTFH61-C-00025, "Performance-Related Specifications for Asphalt Concrete—Phase II," to continue development toward measuring the effect of variations in mixture composition on fundamental mixture properties, such as the effect of deficient asphalt cement on mixture stiffness.

- Parallel efforts reported in Report No. FHWA-RD-89-211, "Development of Performance-Related Specifications for Portland Cement Concrete Pavement Construction," which demonstrated a system designed to consider three key factors: PCC strength, slab thickness, and initial serviceability, in assessing the as-constructed pavement delivered by a contractor for calculating an appropriate reward (bonus/incentive) or penalty (disincentive).
- The accomplishments on NCHRP Project 9-6(1), "Asphalt-Aggregate Mixture Analysis System (AAMAS)," which provides a method for analysis and design of asphalt concrete mixtures based on performance-related tests and provides a system for measuring the fundamental mixture response variables required in the conceptual framework.
- The Strategic Highway Research Program (SHRP) contracts in the Asphalt area, the final products of which are anticipated to be performance-related specifications for asphalt binders and asphalt concrete mixtures.
- Efforts on NCHRP Project 1-26, "Calibrated Mechanistic Analysis Procedures for Pavements," which eventually should result in pavements being designed by mechanistic or mechanistic/empirical procedures.
- Data from the SHRP Long Term Pavement Performance area to support and calibrate results of the above research efforts.

As these research efforts evolve, the results will provide pavement engineers with new concepts to design, specify, and construct pavements for improved performance.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was conducted under NCHRP Project 10-26A by The Pennsylvania Transportation Institute, The Pennsylvania State University. David A. Anderson and David R. Luhr served as principal investigators and authors of the report. The other author of the report was Charles E. Antle.

This work was a collaborative effort that included the valuable efforts of a number of key support personnel: Zahur Siddiqui, Emmanuel Fernando, Joseph Tarris, and Thomas Chizewick. Each of these members of the research team provided special expertise that was essential to the development of the conceptual framework and the overall completion of the study.

FRAMEWORK FOR DEVELOPMENT OF PERFORMANCE-RELATED SPECIFICATIONS FOR HOT-MIX ASPHALTIC CONCRETE

SUMMARY

The research presented in this report was carried out under NCHRP Project 10-26A. This project's two main objectives were to: (1) develop a general conceptual framework for statistically based performance-related specifications that could be applied in general to highway materials and their associated construction processes, and (2) to demonstrate the validity of the conceptual framework. The principal result from this study was the definition of a conceptual framework that can be used to develop performance-related specifications for highway materials and construction. Although the framework is general in nature and can potentially be applied to a wide variety of highway materials and types of construction, the applications in this study were limited to hot-mix asphaltic concrete.

Development and Philosophy of Conceptual Framework

By definition, a performance-related specification requires that the payment provided the contractor be related to the anticipated performance of the as-constructed pavement. In the framework developed in this study, the payment is determined by comparing the life-cycle cost of the as-constructed pavement to that of the target or design pavement, where life-cycle cost includes the cost of the initial construction as well as anticipated user and maintenance costs. In order to execute this comparison, the condition and maintenance costs for the target and as-constructed pavement must be predicted on a yearly basis so that an average annual cost can be calculated for each of these pavements. Cost differentials between the target and as-constructed pavement are then used to determine any payment reduction. Provision is made for bonus payments as well as restricted payments, and when the quality of the work falls below a minimum level the construction is rejected.

Elements Required in a Performance-Related Specification

The primary component of a performance-related specification is the collection of prediction models that are used to predict the life-cycle cost of the target and as-constructed pavements. The models, test methods, and databases required for these prediction models have not been sufficiently developed to the point that they can be reliably used in a performance-related specification. Work planned, or currently underway, is expected to rectify this situation so that the necessary models should be ready for full implementation within the next 3 to 4 years.

In order to develop valid relationships between the materials and construction variables and the fundamental mixture/pavement response variables, and to predict field

performance, both laboratory and field performance databases will be required. The laboratory databases are needed to develop relationships between the materials and construction variables and the fundamental mixture response variables. Databases in the literature are weighted heavily toward measurements on mixtures that center about the target or mean. More robust databases that include data from nonconforming mixtures and pavement construction are needed.

Performance data from the field are also needed to develop the databases required to predict field performance from fundamental mixture and pavement response variables. Historical and observational databases, for which data are available, are deficient for the task at hand. These databases are often incomplete, lacking sufficient data in one or more of the following categories: traffic, environmental, construction, materials, maintenance, or user cost data. The best opportunity for obtaining such data is the construction and close evaluation of the special pavement sections material within the Strategic Highway Research Program's (SHRP) Long-Term Pavement Performance (LTPP) project.

Careful experiment design will be required in the generation of each of the above databases to ensure that the database is robust and statistically sound. Although SHRP will likely develop regional databases (dry-cold, wet-cold, and so forth), additional validation testing will have to be conducted by the individual states.

A paper demonstration of an actual prototype of a performance-related specification was conducted and was shown to be reliable and implementable. The demonstration was accomplished through a sensitivity analysis that is an indispensable step in the development of any performance-related specification. In the demonstration, traffic was the predominating variable affecting pavement performance; the model behaved realistically with respect to the materials and construction variables, indicating fair and reasonable pay adjustments.

The conceptual framework does not include aggregate or asphalt cement as materials variables but, instead, assumes that they are source accepted. Further development, especially concern for legal considerations, must be completed before aggregate and asphalt cement can be added to the framework as materials variables.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

BACKGROUND

Highway agencies have traditionally used method-type specifications for specifying and accepting highway pavement materials and construction. With this type of specification, the methods that are used in constructing a particular section of pavement are stated by the user agency. If the contractor adheres to the methods prescribed by the agency and the adherence is verified by the inspector, 100 percent payment to the contractor is assured. A major deficiency of the method-type specification is that penalties for contractor nonconformance are often arbitrary and based solely on the judgment of the inspector. Statistical concepts are seldom employed in a typical method-type specification.

In the past 20 years, a number of states have moved toward end-result specifications in which the contractor is responsible for quality control and is free to choose the construction methods. Thus, with the adoption of end-result specifications, the burden of choosing the proper construction methods and the responsibility for quality control have shifted from the highway agency to the contractor.

Whereas the end-result specifications adopted in the last 20 years are generally judged as being desirable by both contractors and highway agencies, payment schedules are almost universally based on the past ability of contractors to perform. This is in contrast to an end-result, performance-related specification (PRS), where the contractor's payment is adjusted in relation to any loss in pavement life or performance that may result from contractor nonconformance.

The objective of NCHRP Project 10-26A, "Performance-Related Specifications for Hot-Mix Asphalt Concrete," was to develop a conceptual framework which can serve as a guide for the development of a PRS. The framework was to be broad in concept so that it could be applied to pavement materials in general, but the demonstration of the framework was to be applied to hot-mix asphaltic concrete pavements. At the outset of this effort it was recognized that the project resources were insufficient to develop working specifications which would be ready for implementation when the project was completed. Further, some of the key elements of such specifications were not available and will only be available with the completion of the Strategic Highway Research Program (SHRP), the National Cooperative Highway Research Program (NCHRP) Project 9-6(1), "Development of Asphalt Aggregate Analysis System" (AAMAS), and other studies being completed by the Federal Highway Administration (FHWA) and NCHRP. Therefore, at the request of the project panel the resources of the project were directed toward the development of a conceptual framework and the identification of those elements of the framework that require further development.

PROBLEM STATEMENT

Recent years have witnessed a trend in the highway industry toward the adoption of statistically based, end-result specifications. The payment schedules in those specifications are typically based on the historical performance of the construction industry, and not on the loss in pavement performance that results from contractor nonconformance. Ideally, the price adjustment resulting from any nonconformance by the contractor should reflect the increased cost that will be incurred by the highway agency over the life of the pavement.

Although all of the elements that are needed to develop fully functional and reliable PRSs are not available at this time, a conceptual framework for such specifications is needed to guide the work that will be performed as part of the SHRP Asphalt Program and other FHWA and NCHRP research programs.

OBJECTIVES AND SCOPE

The overall objectives of this study were to: (1) develop a general conceptual framework for statistically based PRSs that can be applied in general to highway materials and their associated construction processes, and (2) demonstrate the validity of the conceptual framework.

The study was confined to the use of existing relationships between materials and construction (M&C) variables and pavement performance. The development of new algorithms for predicting mixture response or pavement performance was outside the scope of the study. Specific items addressed by the research team were, as follows:

- The development of a conceptual framework that can be applied to hot-mix asphalt materials and construction.
- The identification of pavement condition indicators that are related to M&C variables.
- A review of existing performance-related databases to identify databases that are of potential use in developing PRSs.
- An evaluation of currently available performance models that are of potential value in developing a PRS payment schedule.
- The identification of M&C variables that can be controlled by the contractor and that are related to pavement performance.
- The development of a conceptual framework for a performance-related acceptance and payment schedule.
- The identification of alternative strategies for demonstrating the validity of the conceptual framework.

OVERVIEW OF THE CONCEPTUAL FRAMEWORK

Before proceeding to the details of the research plan, a brief review of the assumptions used by the researchers in the development of the framework is appropriate. First, the researchers assumed that the objective of any specification must be clearly stated, both to the user agency and to the contractor. This is essential to ensure that the pavement is built in accordance with the materials and construction specifications, and that payments to the contractor are in accordance with the expected performance of the as-constructed pavement. To meet this objective, the M&C variables that are related to performance, and over which the contractor has control, must be identified and separated from the materials, construction, design, and environmental variables over which the contractor has no control.

To successfully develop a PRS for hot-mix asphalt, it must be possible to relate the M&C variables to pavement performance by some mathematical algorithm. Ideally, the same algorithm(s) should be used for the specification and for designing the pavement. With the M&C variables identified and the performance algorithm well-defined, the anticipated performance of the as-constructed pavement and the design (target) pavement life may be predicted and compared. Life-cycle cost, expressed as equivalent uniform annual cost or some other appropriate economic factor, may then be used to judge the relative costs of the target and as-constructed pavements. The difference between these costs determines the payment schedule and any price adjustment assigned to the contractor. The adjustment may be either positive or negative. Limits are placed on any bonus payment as well as on the level of nonconformance tolerated before the work is rejected without payment to the contractor.

The key features of the conceptual framework that were adopted by the research team included: (1) a payment schedule that is related to the difference between the projected performance of the target and as-constructed pavements; (2) the use, for payment and quality assurance purposes, of only those M&C variables that are performance-related and that can be controlled by the contractor; (3) use of fundamental mixture response variables to predict the performance of the target and as-designed pavements; (4) the incorporation of the pavement design algorithms into the schedule used to pay the contractor; (5) the use of a predicted equivalent uniform annual cost, or some other economic factor, to express life-cycle cost as a basis for determining any payment adjustment; and (6) the use of stochastic variables and statistical concepts and methodologies wherever appropriate.

The primary difference between the proposed performance-related conceptual framework and the framework of current end-result specifications is the dependency of the payment schedule, in the performance-related specification, on anticipated pavement performance.

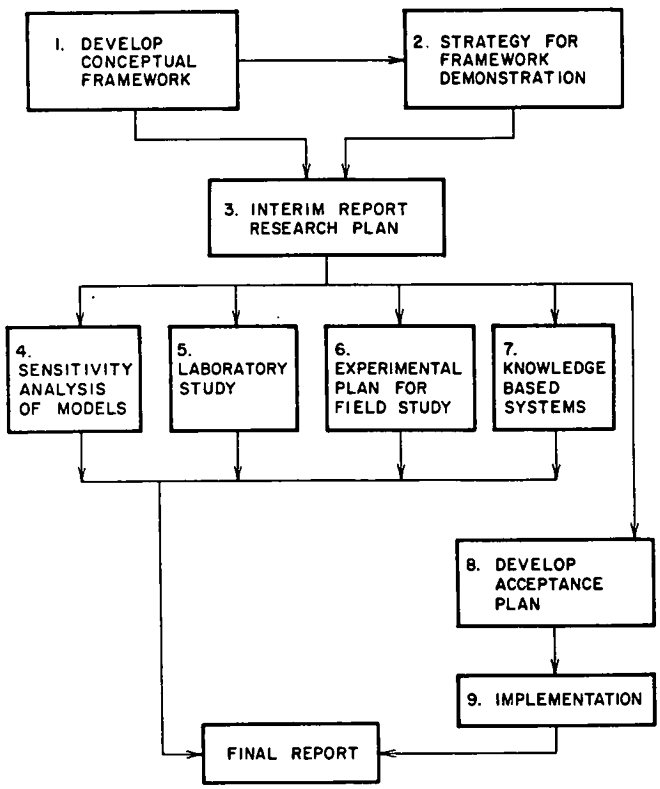


Figure 1. Flow diagram illustrating project tasks.

RESEARCH APPROACH

To meet the objectives of this project, a research plan was adopted which consisted of nine tasks, as shown in Figure 1. These tasks and the type of work performed are briefly described as follows:

1. *Development of a Conceptual Framework*—The initial work was focused on the development of a conceptual framework for developing PRSs that can be applied to highway materials and pavement construction. The framework that was developed is general in nature so that it can be applied to highway materials, in general. However, hot-mix asphalt pavement was used in the examples and demonstrations.

2. *Identification of Alternative Strategies for Framework Demonstration*—In this task, a number of alternative strategies for demonstrating the conceptual framework were identified. These strategies were presented to the panel, and a preferred strategy was selected for implementation during the final phase of the project.

3. *Interim Report and Research Plan*—A report detailing the conceptual framework and alternative strategies for demonstrating the framework was submitted to the panel in the form of an interim report. This report also included a research plan for the remainder of the project, which included tasks 4 through 9.

4. *Sensitivity Analysis*—In this task, sensitivity analyses were conducted of various models that relate M&C variables to fundamental mixture response, and fundamental mixture response variables (FMRV) to fundamental pavement response. The purpose of this task was to demonstrate techniques that can be used by other researchers to evaluate the sensitivities of various models (algorithms) that may eventually be incorporated into the specifications framework.

5. *Laboratory Study*—A laboratory study was conducted to demonstrate the technique that should be used to develop models that relate nonconformance of mixture and construction variables to variations in FMRV. For this purpose, a two-level partial factorial study was conducted. The study variables were asphalt and aggregate type, air voids, asphalt content, and percent asphalt passing the No. 200 sieve.

6. *Development of an Experimental Plan for a Field Study*—The purpose of this task was to develop a field study that can be used to develop or verify relationships between M&C variable nonconformance and pavement performance. A review of observational and experimental databases was also included as part of this task.

7. *Use of Knowledge Base for Expert Systems in the Development of Performance-Related Specifications*—The objective of this task was to investigate the utility of expert systems in the development of PRSs. This task consisted primarily of a review of the potential use of expert systems; they were not used during the development of the framework.

8. *Acceptance Plans for Performance-Related Specifications*—A framework for developing a statistically based, performance-related acceptance and payment plan was developed in this task. This plan consists of a formal procedure that can be used to evaluate the acceptability of a given lot of material and to allocate payment in proportion to the anticipated performance of the pavement.

9. *Implementation*—This task was essentially a paper demonstration of the conceptual framework. Several M&C variables were chosen and realistic data for these variables were assumed. A performance model was evoked, and the annualized cost of the target and as-constructed pavement were computed. These results were used, in conjunction with the payment schedule, to determine the contractor's payment for a hypothetical project.

The results of these tasks form the basis for the findings and conclusions presented in the remainder of this report.

CHAPTER TWO

FINDINGS

The findings that resulted from the completion of the research plan are described in this chapter. These findings are supplemented by Appendixes B through G, where several topics are presented in greater detail. For the convenience of the reader, a glossary of terms is presented in Appendix A.

DEVELOPMENT OF THE CONCEPTUAL FRAMEWORK

The primary objective of this study was to develop a conceptual framework for performance-related specifications (PRSs), wherein the payment to the contractor is related to the anticipated performance of the as-constructed pavement. The conceptual framework was designed so that it could be applied to the development of PRSs for highway pavements and materials in general; however, the primary focus and illustrative examples in this study are for hot-mix asphaltic concrete.

Factors Considered in the Framework

The framework within which PRSs are developed must accommodate all the factors that affect pavement performance. These factors may be grouped as design, controlled, and undefined factors. Design factors include environmental, traffic, and other site-dependent factors such as quality of existing subgrade support and drainage characteristics. For the most part, these factors are inputs to the design and must be assumed constant for a particular project. Whereas for specification purposes the design factors would most likely be considered constant for a given construction project, their treatment as stochastic variables for design purposes is not precluded.

Controlled factors are those that can be controlled by the engineers and contractors that specify the design, choose the materials, and execute the construction. Materials and construction (M&C) variables are controlled factors and, therefore, are of primary interest in this study. Examples are the percent asphalt cement, air voids in the compacted mat, and mat thickness.

The undefined factors are the source of uncertainties in measured properties and predicted results. Uncertainties generated by these factors may include the uncertainty of traffic predictions or unknown variations in subgrade support or drainage not accounted for in current or future performance models. Other undefined factors may include those that have not been identified as being significant in affecting pavement performance. These unidentified factors contribute to the overall uncertainty or "noise" in the performance models.

Types of Variables Within the Conceptual Framework

The nature of the variables included within the above factors must be well understood by those developing a PRS. At least five types of variables that affect pavement performance must be considered: (1) pavement design variables for which constant values are assumed prior to or during the pavement design process, (2) pavement design variables determined iteratively as a part of the pavement design process, (3) variables that specify the recipe values for the hot-mix asphalt mixture, (4) quality control and acceptance variables for the component materials, and (5) construction quality control and acceptance variables.

The first type of variable includes those variables whose values must be assumed prior to or during the design process. These variables include environmental factors such as drainage properties and number of freeze-thaw cycles, as well as traffic loading factors such as average daily traffic and percent trucks. Other environmental factors are related to the structure upon which the pavement is to be built, including factors such as the subgrade modulus, support of the existing pavement, or subbase characteristics. Once again, these factors are outside the control of the contractor and the designer of the pavement. Whereas for the purpose of an acceptance plan the values of these variables are fixed for a given project, they may be considered as stochastic variables during the pavement design process. When the design process is completed, these values will remain fixed and will be used in predicting the performance of the target and as-constructed pavements.

Values for the second type of variable are determined or assigned iteratively during the pavement design process. Pavement thickness is an example of this type of variable. Once the pavement thickness has been determined, its design or target value is held constant during the bidding, construction, and acceptance process. Once again, this type of variable may be considered as stochastic in nature. Future development of pavement technology and PRSs may allow this type of variable to be varied in the bidding process (i.e., the contractor bids alternative designs with different thicknesses), but, currently, these variables will likely be held at fixed design or target values. Pavement roughness and the thickness of other pavement layers are other variables that might be included in this classification.

The third type of variable is referred to as recipe variables because they are dependent on the particular job-mix formula (or recipe) used by the contractor. In most instances, the recipe is not known until after the contract has been awarded and, in many instances, after the contract is underway. These recipe variables include the percent asphalt content and percent passing the No. 200 sieve. Until the source of aggregates has been established and a job-mix has been submitted to the user agency, the recipe is unknown. This type of variable is generally not a fundamental response variable of either the mixture or the pavement, but instead is a variable that reflects the recipe under which the contractor is operating.

The fourth type of variable specifies the properties and uniformity of the component materials used in the production of the hot-mix asphalt concrete. Examples of this type of variable are the penetration, viscosity, and thin-film oven test properties of the asphalt, and the Los Angeles abrasion, absorptivity, and soundness of the aggregates. Variables or properties such as these, and other more fundamental properties which will be developed as part of the SHRP and other ongoing research, are undoubtedly performance-related variables and should, in the future, be incorporated in some manner into PRSs. However, the inclusion of variables such as these is not considered within the scope of the current demonstration, although they can be accommodated in the conceptual framework.

The fifth type of variable consists of those construction variables that are measures of the quality and uniformity of the contractor's construction. These variables include the thickness (also a design variable) and roughness of the completed pavement. These variables have little to do with the recipe but are important in the acceptance of the job and are directly related to the performance of the completed pavement.

A detailed evaluation of the variables is important in the development of PRSs. As will be shown in the next section, the variables included in a specification that is based on pavement performance must be chosen with care to ensure that they are both performance-related and controllable by the contractor.

General Assumptions and Basis of Framework

Several ground rules were established for the development of the conceptual framework. First, it was decided that the time to economic failure predicted for the as-constructed pavement would be compared to the time to economic failure for the target construction, and the life-cycle costs associated with the difference in the number of load applications to failure would be used as a payment criterion. (Note: The word target is used to indicate the desired construction. The design and target pavements are synonymous.) It was assumed that performance models are (or will be) available that can be used to calculate pavement performance as a function of the number of load applications. Further, it was assumed that these performance models are based on measured or predicted fundamental mixture response variables (FMRV), such as mixture strength or stiffness, and calculated fundamental pavement response variables (FPRV), such as stress and strain within the pavement structure. A general review of pavement performance models that can be used to calculate pavement performance is presented in Appendix B.

Specifically, the following assumptions were made in the development of the conceptual framework:

- The basic element in the framework is the various relationships between the M&C variables and pavement performance. The framework is not built upon any specific relationship but can accommodate different relationships such as those in the new AASHTO Design Guide (*1*) or any other valid relationships between M&C variables and pavement performance.
- The same performance relationships should be used to design the target pavement and to assess the costs associated with any materials or construction nonconformance in the as-constructed pavement.
- The framework must include a statistically based acceptance plan and payment schedule. Process quality control is considered the responsibility of the contractor and is an inherent part of the specification. Furthermore, the framework places the responsibility for quality control with the contractor rather than with the user agency. Acceptance testing may be performed by the user agency or the contractor, as desired by the user agency.
- Pavement performance can be quantified in terms of functional performance or structural performance. Functional performance is measured by evaluating pavement condition indicators such as roughness, skid resistance, and cracking. Structural performance is measured by nondestructive testing (deflections) and properties such as backcalculated moduli.
- For the purpose of calculating payment factors, performance is used to calculate life-cycle cost, expressed in terms of equivalent uniform annual cost. This provides a common denominator for equating life-cycle costs with the penalties assessed for M&C specification noncompliance. Life-cycle costs include the costs of construction and maintenance, and user costs.
- To be fair and equitable, the specifications must incorporate, as M&C variables, only those variables over which the contractor has control.
- The conceptual framework should be of a modular design so that the key elements can be modified as new performance relationships or other algorithms are developed in future studies.

GENERALIZED CONCEPTUAL FRAMEWORK

The generalized conceptual framework that was developed during this study is shown schematically in Figure 2. This framework is consistent with the general principles outlined in the research proposal and later detailed by Irick (*2*). The conceptual framework is built upon the hypothesis that the performance of the as-constructed pavement can be predicted from pavement performance models if the properties of the hot-mix asphalt and the pavement are appropriately measured. This process is illustrated in Figure 2, which contains several different levels described as follows:

- Level A variables characterize the compositional characteristics of the pavement and the materials in the pavement. These may be thought of as the recipe for constructing the pavement. They include the thickness of the layers, the gradation of the mixture, and the asphalt content. These are the materials and construction (M&C) variables.
- Level B variables are selected as part of the design process and become fixed for the particular construction project under consideration. They may, however, be considered as stochastic variables during the design process. These pavement design variables (PDVs) include the thickness of the subbase, modulus of the subbase, drainage factors, traffic levels, and so on.
- Level C variables are measured pavement response variables (MPRV). They cannot be measured until the pavement has been constructed. They are not used in the conceptual model as developed in this study, although the framework could be modified to accommodate their use.
- Level D variables are mixture properties that have been backcalculated from the Level C variables. They are not used in the conceptual framework developed in this study, and it would be difficult to modify the framework to accommodate their use.
- Level E variables are fundamental mixture response variables that have been measured in the laboratory. They may be measured as part of the mix design process, or on the optimum mixture after the mixture design process is completed. They may also be measured on laboratory-compacted field mix or on field cores. These variables may be used to establish construction target values or values for acceptance control. If necessary, values for the FMRV may also be determined empirically from relationships between M&C variables and the FMRV.
- Level F variables are fundamental pavement response variables (FPRV) expressed in units such as strain or stress. They are to be differentiated from the Level C variables in that the

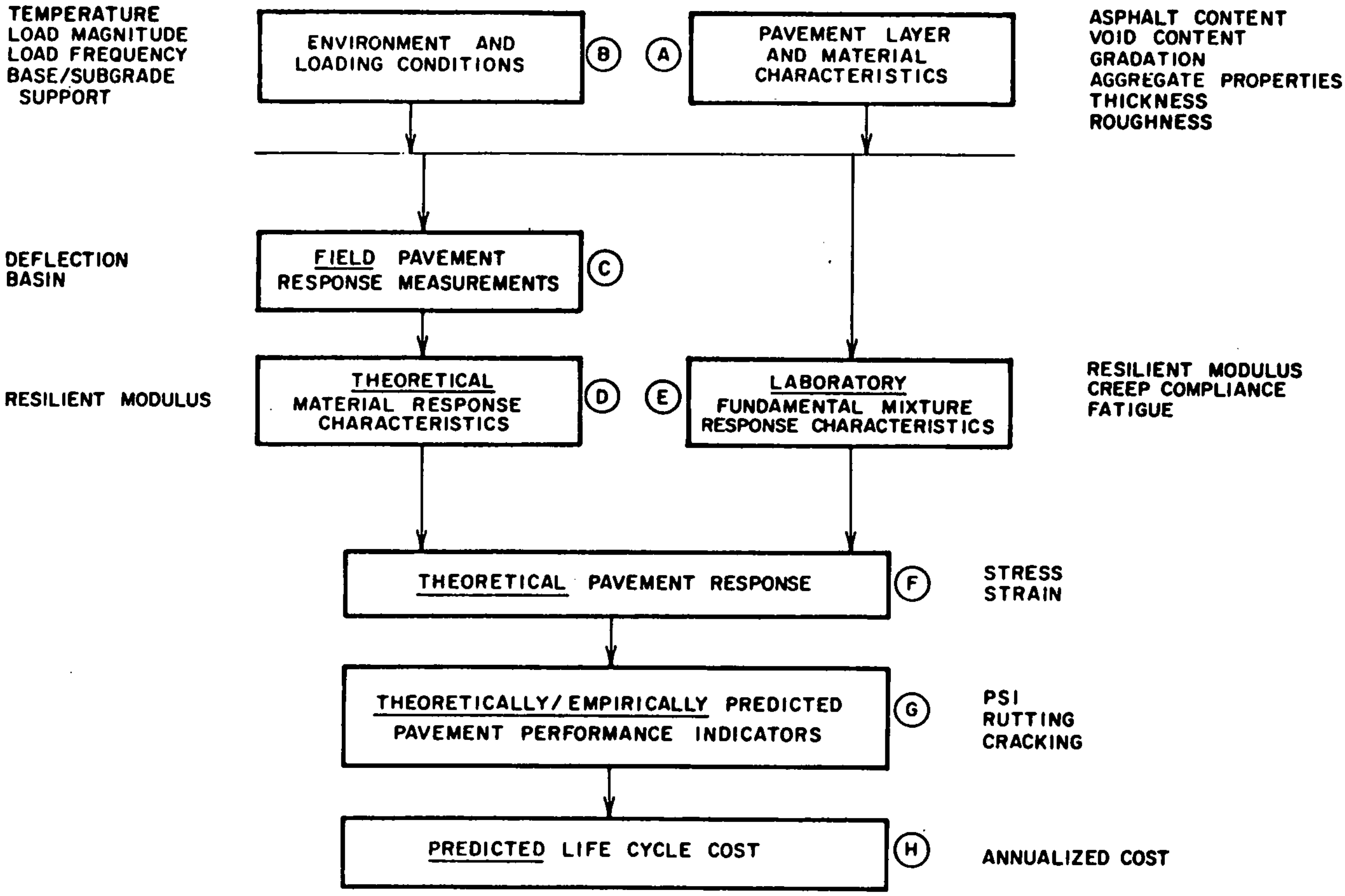


Figure 2. Generalized conceptual framework.

Level C variables describe the measured response of the pavement system, whereas the Level F variables describe the calculated response of any point within the system.

- Level G variables are performance indicators that can be predicted, preferably, by some mechanistic model using the Level F variables. These variables, such as the current serviceability index (PSI), describe the condition of the pavement at any time, *t*, as a result of the combined action of the environment and the traffic.
- Level H variables describe the life-cycle cost of the pavement. Life-cycle cost may be expressed as the average annual cost or some other indicator of cost.

Two approaches to a PRS specification for hot-mix asphaltic concrete are implied in Figure 2. The specification may be based on environmental and loading conditions and in situ field measurements (levels B and C), as shown in the left-hand path in Figure 2. This path, B-C-D-F-G-H, implies that only the as-constructed structural response characteristics of the pavement are of significance. For example, this would eliminate percent air voids as a specification acceptance criterion. Elimination of a construction variable such as air voids would be counter to the experience of most materials engineers who recognize the important influence of air voids on performance.

As an alternative, a PRS may be based on the right-hand path in Figure 2, which includes environmental and loading conditions, pavement layer and material properties, and fundamental mixture response variables (levels B, A, and E) for the as-constructed pavement. Because of the absence of measured materials properties from the left-hand path, the research team has selected the right-hand path, B-A-E-F-G-H, for the conceptual framework.

Obviously, some of the algorithms in Figure 2 are not fully developed. This is particularly true with regard to models that relate material properties to fundamental pavement response and to pavement performance. The Strategic Highway Research Program (SHRP) and, in particular, the special pavement sections (SPS) as envisioned in the Long-Term Pavement Performance Program (LTPP), offer an excellent opportunity to develop the databases needed to validate many of the missing links in the conceptual framework. A discussion of performance models is presented in Appendix B, and the use of sensitivity analysis in assessing the validity of these models is addressed in Appendix C.

APPLICATION OF THE CONCEPTUAL FRAMEWORK TO HOT-MIX ASPHALT

A flow diagram illustrating a PRS for hot-mix asphaltic concrete is shown in Figure 3. The key elements in the specification are:

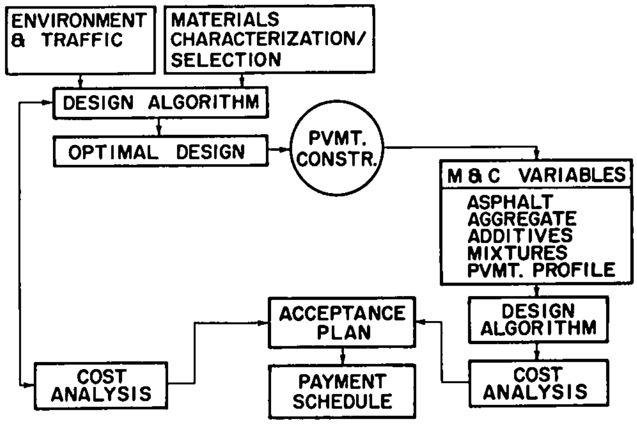


Figure 3. Generalized framework for a performance-based specification for hot-mix asphaltic concrete.

- The target design values, which include the pavement design (i.e., thickness, percent compaction, and allowable roughness) as well as the target values for the mixture (i.e., percent asphalt cement, gradation, and Marshall stability). These are the target M&C variables.
- A characterization of the M&C variables for the as-constructed pavement. These are the measured values of the as-constructed M&C variables.
- The algorithms that are used to determine life-cycle cost.
- Predicted life-cycle cost for the target and as-constructed pavements.
- An acceptance plan and payment schedule.

As shown in Figure 3, the payment to the contractor is determined by the acceptance plan in which the life-cycle costs for the target and as-constructed pavements are compared. However, before the acceptance plan can be developed, a number of considerations must be addressed relative to how, and at which point in the design-bidding-construction process, the target values are selected.

Role of the Pavement Design Process

The pavement design process is an integral part of a PRS because it is used to develop the pavement design which, during the bidding, construction, and acceptance process, becomes the target design. Ideally, the pavement design should be based on mechanistic principles so that the pavement design algorithms can be used within the specification for predicting service life.

Once a pavement design has been completed for a particular project, the design factors are fixed. There is, however, a degree of uncertainty associated with each design factor, which will be reflected in the reliability of the design. Concepts of reliability have been introduced in the recently revised AASHTO Interim Design Guide (*1*) and can be readily accommodated within the framework, as can a series of alternative designs, each with an associated cost and reliability. The effect of variability in the design variables is addressed in Appendix D.

The environment, traffic, and materials-related variables are inputs to the design algorithm and are used with the performance relationships to determine different pavement design and rehabilitation strategies. The optimum strategy for a given set of conditions is selected from an economic analysis. The implementation of this strategy is initially carried out during the construction of the pavement structure for the initial performance period. At this stage, departures from the optimal pavement design can be expected, and it therefore becomes necessary to monitor the levels of certain performance-related M&C variables so as to evaluate the extent to which the as-constructed pavement conforms to the established optimum design. In the evaluation of the consequences of departures from the optimum design, the algorithm that was used during the design process to select the optimum strategy is also used to determine the optimum rehabilitation strategy for the as-constructed pavement structure. This optimum rehabilitation strategy may be different from that selected for the initial pavement design, depending on the magnitude of the deviations from the target values of the performance-related M&C variables, and on whether certain deviations tend to produce offsetting effects on the predicted pavement performance. In any case, the life-cycle cost associated with the optimum rehabilitation strategy for the as-constructed pavement is evaluated and compared with the corresponding life-cycle cost for the initial pavement design to determine appropriate changes in payment to the contractor. By evaluating, during the preconstruction process, the predicted consequences of contractor noncompliance, it is possible to establish an acceptance plan and a payment schedule for the completed construction work.

Performance-Related Specifications for Mixture Components

Currently, in most agencies, the asphalt cement, aggregates, and additives are source-accepted, meaning that as long as they meet some minimum criteria they are acceptable. A more desirable situation would be to specify and accept the mixture components on the basis of their contribution to the performance of the pavement, implying a PRS-type specification for each mixture component. The conceptual framework developed in this study will accommodate performance-related specifications for the component materials, but the development and demonstration of PRSs for mixture component materials was outside the scope of this study. One of the objectives of the SHRP research program is to develop such specifications.

Applying PRSs to the mixture components poses some difficult legal problems when more than one contractor is involved in a project. A typical situation occurs when asphalt and aggregate are supplied to a hot-mix producer who, in turn, supplies hot-mix to a paving contractor who, in turn, is a subcontractor to the prime contractor. In such a situation it becomes difficult to determine which contractor or subcontractor should be assessed the penalty. This becomes even more complicated when a given contract is supplied with asphalt cement from more than one source. Although addressing this problem is outside the scope of the current study, this aspect of the PRS conceptual framework must be investigated before PRSs can be developed for the component materials.

Selection of Values for Target Design

The PRS illustrated in Figure 3 requires that a pavement design be completed for each construction project and that the design include life-cycle costs. In order to conduct a pavement design, mixture properties must be either known or assumed. Depending on the design algorithm, these properties may include the traditional mixture recipe properties (e.g., percent asphalt cement), empirical mixture properties (e.g., Marshall stability), or FMRV properties such as resilient modulus. However, these mixture properties are generally unknown when the project is bid and cannot be measured until the job mix is determined, which may be some time after the construction contract has been awarded. Consequently, the pavement design is typically based on M&C and FMRV values assumed by the designer and the persons preparing the bidding documents rather than on laboratory-determined values. The values assumed for the design may represent average values for the agency or may be determined by some other means. Using the resilient modulus, M_r, as an example, the target value for pavement design and bidding purposes may be assumed on the basis of the historical characteristics of mixes produced within a state or within a region of a state. Associated with the historical value would, of course, be a mean and a standard deviation. The M&C variables and FMRV are stochastic variables, and both a mean and a measure of dispersion are necessary in order to properly characterize them.

The selection of a historical mean and standard deviation for M&C and FMRV values is certainly justified for the preliminary design phase. These assumed design values may be carried throughout the course of the project; this is appropriate for certain construction variables such as the pavement thickness, but may be inappropriate for materials or mixture variables such as asphalt content or resilient modulus. Design target values for the mixture will most likely become those identified in the contractor's bid or those of the job mix once the mix design is submitted to (or completed by) the user agency and is accepted and contractually adopted by the contractor.

At the present time the research team recommends that realistic target values be chosen for the M&C variables during the design process and that the target values for the construction variables (thickness, roughness, air voids) be continued, unchanged, throughout the course of the project. The same would be true if FMRV are used in the design process. Given current construction practice and the present state of the art in the characterization of the FMRV, the research team recommends that the target values for the materials variables be measured or predicted from the job mix. As the AAMAS, SHRP, and other research results become available, it may be possible for target FMRV mixture values to be established as part of the contractor's bid. This would be the preferred procedure if it can be accomplished. With values for the FMRV from the AAMAS included as part of the contractor's bid and the specification, the conceptual framework will offer an incentive to the contractor to optimize the performance-related quality of the job mix, something that current end-result specifications do not accomplish.

The framework is compatible with contractor-prepared mixture designs, a situation that is preferable to agency-prepared designs. If the job mix is used to establish target values for the mixture, the astute contractor will want to have the mix design properties as well defined as possible. In this manner, the mixture can be optimized for performance, offering the contractor the maximum advantage in preparing the bid. In this scenario, it will be to the advantage of the bidder and the agency to have, as M&C or FMRV acceptance variables, mixture properties that are strongly related to the potential performance of the pavement.

Determination of Values for the Fundamental Mixture Response Variables

In the conceptual framework, the FMRV for the hot-mix asphalt are used to calculate the fundamental responses of the pavement system. As noted earlier, the FMRV include modulus or stiffness, creep parameters, fatigue parameters, and any other properties needed to define mixture behavior. These FMRV values are, in turn, used to predict levels of the pavement distress indicators which can then be used to predict the performance and life-cycle cost of the pavement.

Several alternatives are available by which FMRV data can be obtained. The preferred approach is to measure them directly using the AAMAS mixture characterization procedures (*3*). This is shown in Figure 4, where the AAMAS characterization is completed on the optimum or target mix once the mix design has been completed. AAMAS-type FMRV measurements may also be specified on cores or laboratory-compacted hot-mix specimens to characterize the as-constructed pavement.

Because of cost and time considerations, it is highly doubtful that the AAMAS characterization scheme can be fully implemented for all contracts. To accomplish this, a full AAMAS testing program must be completed for each design mix and for acceptance samples. If target FMRV values obtained from the job mix submitted by the contractor are used as specification criteria, an AAMAS testing schedule must be conducted on the job mix. Whereas this may be appropriate for high level uses, it is doubtful, again because of cost and time considerations, that such a procedure would be appropriate for many paving jobs. The ultimate implementation scenario would be to use

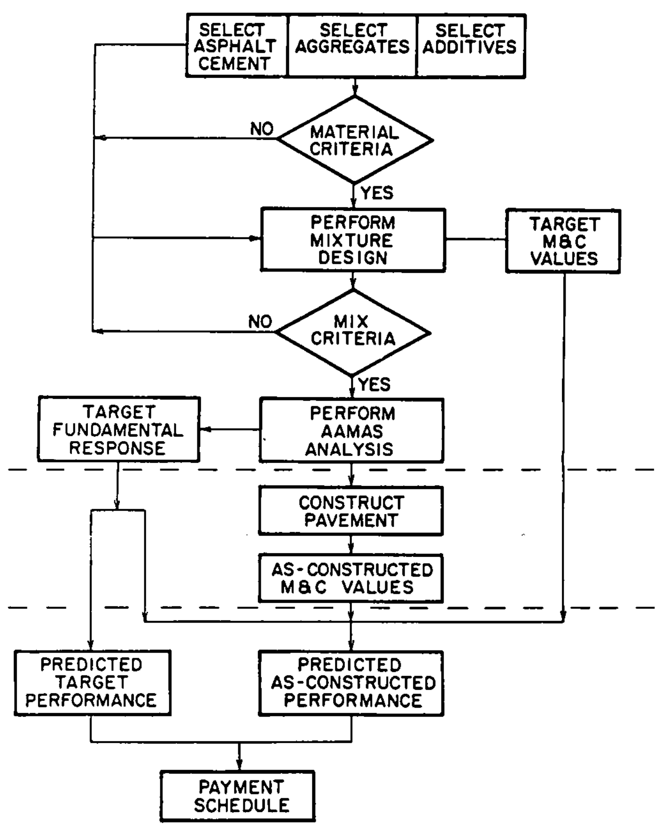


Figure 4. Flow diagram illustrating inclusion of AAMAS in acceptance plan.

AAMAS-determined properties for the bidding process, requiring that the contractor characterize the proposed mix prior to the bidding process. As mentioned earlier, this is not practical, not only from a cost and time consideration, but also from the logistics of many situations where the source of aggregates is not known until after the contract is awarded.

An alternative, but least desirable, approach would be to predict values of the FMRV from prediction equations, such as the Asphalt Institute equation, that relate the complex modulus $|E^*|$ to the percent asphalt content, air voids, and percent passing the No. 200 sieve in the mix (*4*). This relationship may be applied to the design mix to obtain target FMRV values or to field cores to obtain FMRV values for the as-constructed pavement. This is the approach that will be used in the framework demonstration presented later in this report and is the approach that must be recommended until AAMAS-type characterization procedures are fully developed. However, the framework will allow the use of FMRV properties predicted or calculated by any manner and at any time during the design, bidding, construction, and payment process period.

A third approach to the determination of values for the FMRV would be to apply AAMAS-type characterization procedures to the design mix as part of the mix design process and to predict the effect on the values that might result from nonconformance the M&C variables. The functional form of the prediction algorithm would then be:

$$FMRV_c = [FMRV_T] \cdot K\,[f(M\&C_T - M\&C_c)] \qquad (1)$$

where $FMRV_c$ = as-constructed value for a fundamental mixture response variable, $FMRV_T$ = target value for a fundamental mixture response variable, K = adjustment factor, $M\&C_T$ = target values for M&C variables, and $M\&C_c$ = as-constructed values for M&C variables.

In other words, the values for the fundamental mixture response variables would be measured directly for the design mix but predicted for the as-constructed mix using the differences between the target materials and construction values and the as-constructed materials and construction values.

This approach would require that values for the target FMRV be measured only once for a given mix, affording considerable economy when a given mix is used for multiple jobs. Unfortunately, the adjustment factors, *K*, that are needed to implement this approach are not reported in the literature and are not available.

The framework could be readily modified to accommodate the results of in-situ structural testing, such as falling weight deflectometer (FWD) measurements. Such measurements would be used to backcalculate FMRV or FPRV for the as-constructed pavement. This approach is not considered to be practical given the current state of the art, and will not be considered further in this study.

In summary, a number of scenarios can be used to calculate values for the target and as-constructed FMRV: (1) Values for the FMRV for the design mix and for the as-constructed mix may be measured directly using procedures such as those provided by AAMAS. (2) Values for the FMRV for the design mix and for the as-constructed mix may be predicted using prediction equations that contain the M&C variables as independent variables. (3) Values for the design mix may be measured directly using AAMAS procedures, but the as-constructed values are obtained by correcting the design values with a "correction" equation that is based on the differences between the target and as-constructed M&C variables.

From the standpoint of rigor and reliability of the FMRV values, the first scenario is the most desirable, although it may not be acceptable from the standpoint of cost. The second scenario is the least expensive, but produces the least reliable values. The third scenario may offer the most cost-effective compromise if the appropriate algorithms can be developed. Regardless of which scenario is adopted, the direct measurement of M&C variables will most likely remain as an integral part of performance-related specifications.

DEVELOPMENT OF AN ACCEPTANCE PLAN FOR A PERFORMANCE-RELATED HOT-MIX SPECIFICATION

Determentation of Pavement Cost

The primary difference between end-result specifications and performance-related specifications is the payment schedule. The PRS includes a payment schedule that is related to the anticipated performance of the pavement.

To assess the ultimate performance of the pavement, it is necessary to select some common denominator that will allow both the functional and structural aspects of pavement performance to be considered. This evaluation is best accomplished in terms of the life-cycle cost of the pavement. This concept has several advantages:

- The road user pays user costs directly and also pays (indirectly through taxes) the agency costs for construction and maintenance. The common-denominator approach provided by life-cycle cost determination allows the pavement engineer to consider all costs associated with the pavement and to minimize the cost to the user (taxpayer).
- Life-cycle cost information can provide feedback to the pavement design process so that the cost of predicted pavement performance is considered during the structural design, selection of materials, and determination of construction specifications.
- Because the contractor is paid (or penalized) in terms of dollars, the use of life-cycle costs provides a direct link to performance-based payment schedules.
- By reducing the pavement engineering process to one objective (i.e., minimizing life-cycle costs), the procedures for determining the optimum pavement strategy are simpler than if more than one objective is to be considered (multiobjective optimization).

The further development of the conceptual framework and the demonstration of the acceptance plan and payment schedule that follows are based on the concept of a payment schedule keyed to the life-cycle cost of the design and as-constructed pavements.

Pavement Economics

One of the most difficult components of performance-related specifications is the economic quantification of predicted pavement performance. This quantification is necessary because the payment to the contractor (in dollars) must somehow be affected by the expected pavement performance resulting from the contractor's work (in dollars).

Life-cycle costs are used in the conceptual framework as a common-denominator tool in evaluating pavement performance. Life-cycle costs include costs associated with initial construction (or reconstruction), routine maintenance, rehabilitation, user operation, user delay, and salvage value. Future costs are discounted according to a specified interest rate so that cost comparisons can be made on the basis of value at a particular point in time. Costs are considered over a designated analysis period, which can vary in length depending on the specific conditions being analyzed.

By employing life-cycle cost determinations for the as-constructed pavement and the target pavement it will be possible to calculate the increase in life-cycle cost that results from any nonconformance. Comparisons will be made on the basis of the unit price bid by the contractor (measured in dollars per yd^2 in-place). It will not be possible to compare or use the entire contract price in life-cycle cost determination because it includes other contract items, such as fencing, painting, mulching, and earthwork.

The process of wear begins when construction ends and pavement maintenance is essential. The impact that maintenance costs have on total life-cycle costs will depend on the maintenance policy of the agency, including the type of maintenance and when it is performed. Preventive maintenance should result in lower life-cycle costs, while higher life-cycle costs would be associated with deferred maintenance. The type and frequency of maintenance will also depend on the quality of the constructed pavement.

In addition to routine maintenance, rehabilitation may be required to upgrade the geometry (widening) or to correct the surface or structural deficiencies (overlays). These deficiencies may result from inadequate design, inferior construction, or unexpected growth in traffic. Once again, the effect on life-cycle costs will depend on the type of rehabilitation carried out.

Cost is the principal variable used to analyze pavement economics, although some use the term "benefits" as well as "costs." The use of the term "benefits" automatically results in the question, "As compared to what?" because "benefits" refers to the difference between two alternatives. The use of the term "cost" by itself is preferred by the research team, because the transportation service provided by a highway can be evaluated in terms of the cost of that service, rather than through the additional step of comparing it to the cost of service provided by some alternative. This is advantageous in project-level pavement management when, sometimes, dozens of alternative pavement design and rehabilitation strategies are compared. The advantage of a benefit-cost ratio is that it provides information concerning how efficiently the benefit is being provided. However, a similar variable can be expressed using cost only, such as cost per equivalent single axle load (ESAL)-mile, which indicates how efficiently service is being provided. As a result, the term "total cost" is preferred over "benefit/cost" for quantifying pavement economics. The total cost can be expressed as annual cost per mile or as cost per ESAL-mile.

Determination of Cost Responsibility

The problems associated with determining the contractor's cost responsibility can be illustrated through the examples given below. Naturally, for design purposes, total life-cycle cost is considered over a long time frame, possibly spanning several performance periods. However, the effect of one contractor's conformance to specifications can only reasonably be associated with one performance period, because the succeeding performance periods would be a function of how well each contractor conformed to specifications in each period.

The following four analysis examples are not intended for design purposes, but are for evaluating cost responsibility.

- *Example 1*—Total cost is considered for only the first performance period (excluding rehabilitation costs at the end of the performance period). This is shown in Figure 5, where the contractor's cost responsibility is ($\$X_c - \$X_o$). The difficulty with this procedure is that the contractor's influence on future rehabilitation requirements is not considered.
- *Example 2*—Total cost is considered for the first performance period and the next rehabilitation cost. This is shown in Figure 6, where the contractor would be responsible for ($\$X_c + \$R_c - \$X_o - \$R_o$). In this method, the contractor would be penalized for causing the rehabilitation to occur earlier and for any increased cost of rehabilitation R_c over rehabilitation R_o. This procedure is similar to the one developed by Brent Rauhut Engineering, Inc. (BRE, Inc.), for FHWA, in which a decision tree process is used to determine the required rehabilitation at the end of the first performance period (*5*). However, this procedure does not consider the effect of the first rehabilitation on future agency and user costs. This omission may result in the contractor's being overpenalized, because user costs may be reduced because the first rehabilitation occurred earlier. Previous research has shown that this can happen, particularly with high-volume roadways (*6*).
- *Example 3*—Total cost is considered for the optimum strategy that results from the contractor's performance. If failure occurs in 4 years instead of 8 years as a result of the contractor's performance, a new optimum strategy must be developed, as shown in Figure 7. The contractor's cost responsibility is (Σ $\$X_c$

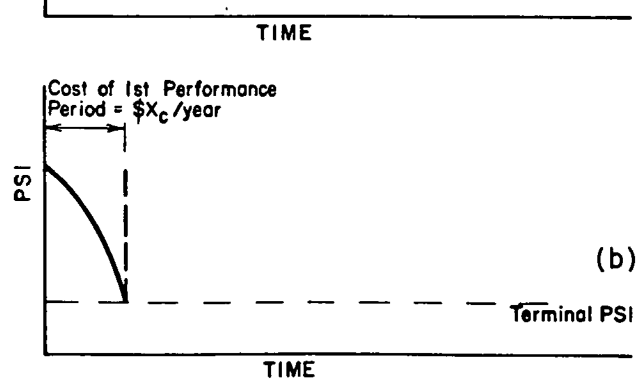


Figure 5. Example of considering cost responsibility based on first performance period only. (a) Optimum design and rehabilitation strategy, based on all contractors being at target specifications; (b) predicted performance based on contractor's work.

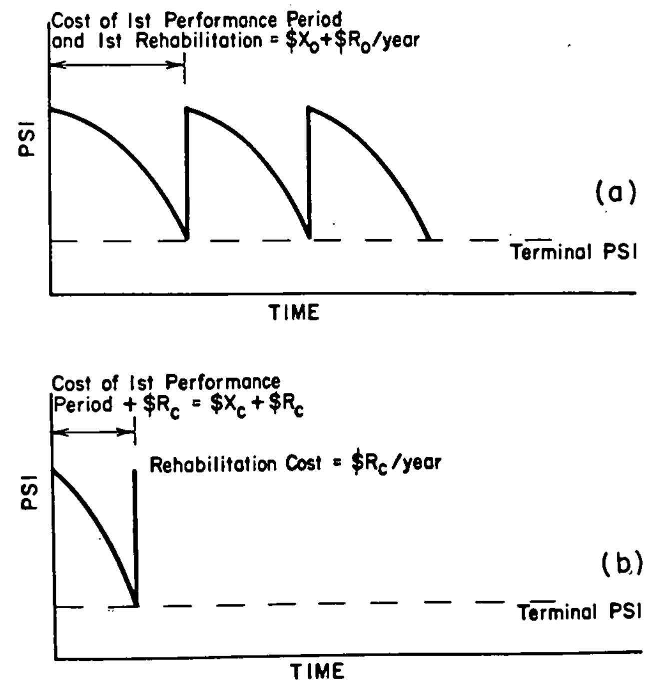


Figure 6. Example of considering cost responsibility based on first performance period and first rehabilitation. (a) Optimum design and rehabilitation strategy, based on all contractors being at target specifications; (b) predicted performance based on contractor's work.

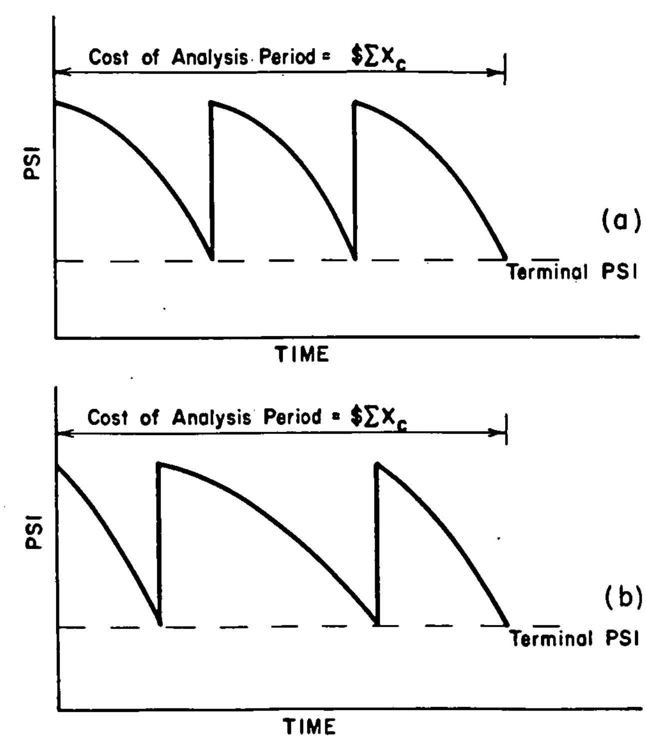


Figure 7. Example of considering cost responsibility based on entire analysis period. (a) Optimum design and rehabilitation strategy, based on all contractors being at target specifications; (b) predicted performance for first period and new optimum strategy for future rehabilitations.

$- \Sigma \$X_o$), which is probably lower than that determined in example 2. A problem with example 3 is the computational effort that would be required to determine optimum pavement strategies for a wide range of possible contractor performances. However, the methodology in this example is probably the most complete and rational.

- *Example 4*—Total cost is considered in two parts: (1) the first performance period, plus (2) the capitalized cost of all future performance periods. Total cost for the first performance period is the concept used in example 1. However, in this example it is combined with the "capitalized cost" of future performance periods. This means that future performance periods are identical and that they occur over an infinite period of time. This assumption simplifies the concept illustrated in example 3, where a variety of future performance periods (and costs) were considered.

Concepts Related to Payment Schedule

A conceptual framework has been developed by the research team for a payment schedule that can be used with PRSs. The conceptual framework is based on comparing the predicted performance of the as-constructed pavement to the predicted performance of the target pavement. Performance is quantified in terms of the performance period costs associated with a certain pavement history.

The procedure selected by the research team for the framework demonstration in this project is shown in Figure 5 (example 1), where only the costs in the first performance period are included. The conceptual framework that was developed will accept any of the methods shown previously. However, the difficulties encountered when future performance periods (and future contractors) are included make it impractical at this time to use more than one performance period for demonstration purposes.

In order to develop a rational approach for the economic evaluation, the project team has developed a procedure based on the economic life of the as-constructed pavement, as compared to the economic life of the target (design) pavement. The term economic life refers to the time in an analysis when the annualized costs are minimum. This concept is frequently used in replacement analysis in industrial engineering applications and is shown in Figure 8. The annualized total cost is calculated as follows:

$$A_n = [\text{Total cumulative cost at year } n] \times \left[\frac{1 - r}{1 - r^n}\right] \quad (2)$$

where A_n = annualized total cost at year n, \$/yd²/yr; $r = 1/(1 + i)$; and i = real discount (interest) rate.

The consideration of the as-constructed cost curve versus the curve for the target pavement is shown in Figure 9. In this example, the as-constructed pavement was not built to design standards, and the nonconformance resulted in higher maintenance and user costs than those of the target pavement. The as-constructed pavement also requires rehabilitation after 8 years, as compared to 10 years for the target pavement.

In summary, the conceptual framework for the payment schedule is based on several basic criteria: (1) The contractor with the lowest bid price is awarded the contract. This bid price for the asphaltic concrete paving is then used in the economic life calculations. It must be remembered that the contract is

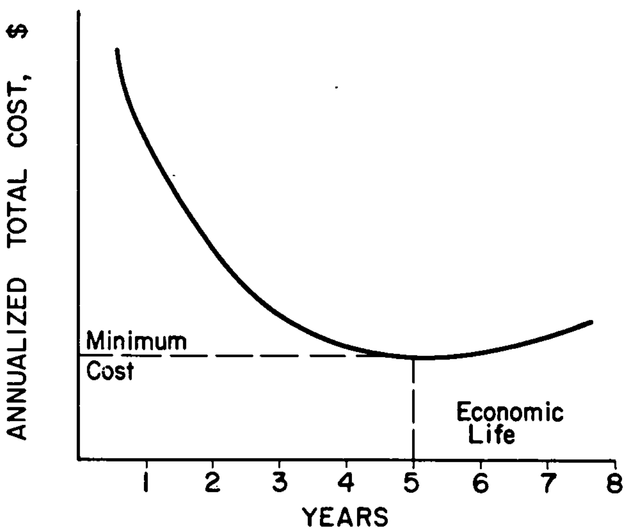
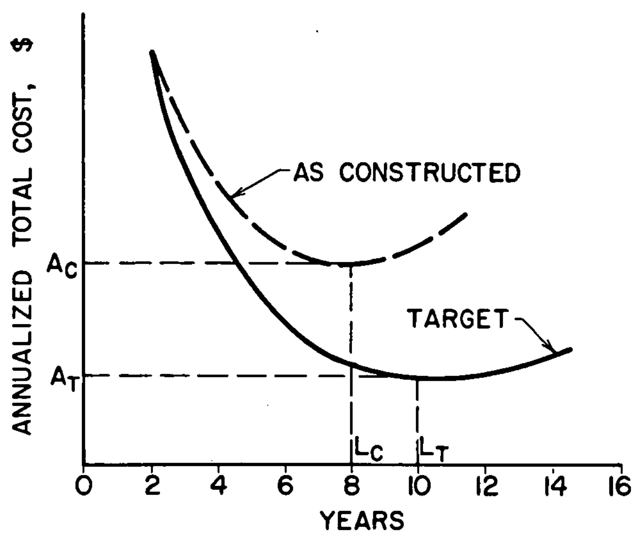


Figure 8. Concept of economic life in replacement analysis.



A_c = annual cost at end of economic life (as-constructed pavement)

L_c = economic life of as-constructed pavement

A_T = annual cost at end of economic life (target pavement)

L_T = economic life of target pavement

Figure 9. Economic life of as-constructed versus target pavement.

awarded on the basis of the overall bid, but this study is focusing on a single pay item, the bituminous paving. (2) The target design, which is used for bidding purposes, is specified by the agency; if the contractor meets this target, payment will be at exactly the bid price. (3) If the contractor is either above or below the target, this will affect the predicted pavement performance, and a cost evaluation will be made of the as-constructed pavement using the total of construction (bid) costs, agency maintenance costs, and user costs. (4) The contractor's bid price payment will be adjusted by the amount of cost difference over the period defined as the economic life of the as-constructed pavement; this adjustment could be either positive (bonus) or negative (penalty). (5) The adjustment in the payment to the contractor will likely have some upper limit (e.g., within some percentage of bid), so that extreme bonuses and penalties will not occur. Provisions will also be made such that the entire item may be rejected and, in this instance, no payment will be made.

Hypothetical Applications of the Payment Schedule

As illustrated in Figures 10 through 13, the as-constructed annual cost, A_c, may be greater than or less than the target annual cost, A_T. When A_c is greater than A_T, a negative price adjustment (penalty) is assessed to the contractor. The life of the as-constructed pavement, L_c, (i.e., the time in years when the annual cost attains a minimum) may be less than or greater than the anticipated life of the target pavement, L_T. These relative values of A_c versus A_T and L_c versus L_T generate the four scenarios presented in Figures 10 through 13. Hypothetical data for each of these scenarios have been used to generate the numbers given in Table 1.

The payment for each of the examples presented in Table 1 is calculated from the following formula:

$$\text{PAYMENT} = \text{BID} - (A_c - A_T)\{[(1 + i)^{L_c} - 1]/[i(1 + i)^{L_c}]\} \qquad (3)$$

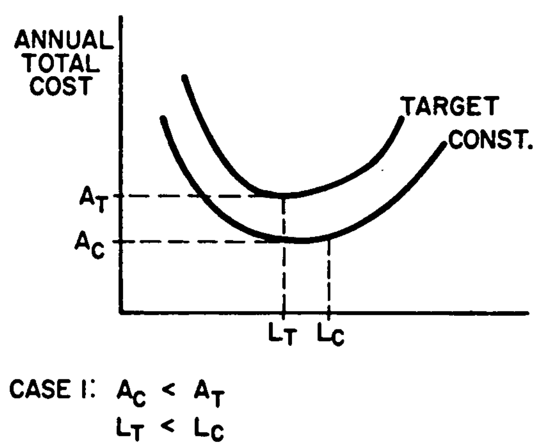


Figure 10. Annual cost versus pavement life, case 1.

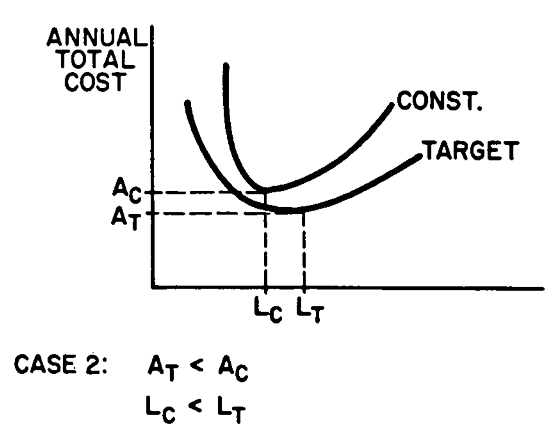


Figure 11. Annual cost versus pavement life, case 2.

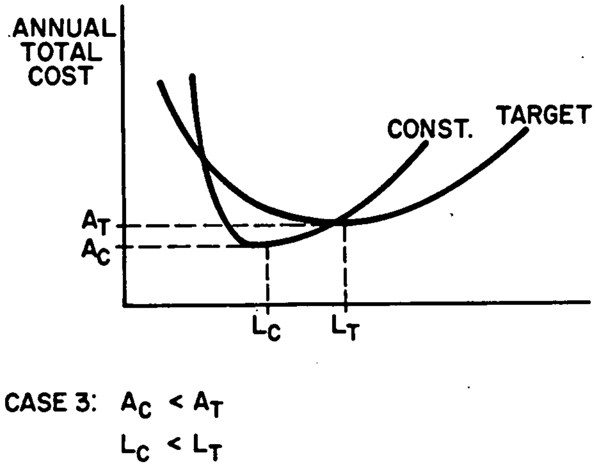


Figure 12. Annual cost versus pavement life, case 3.

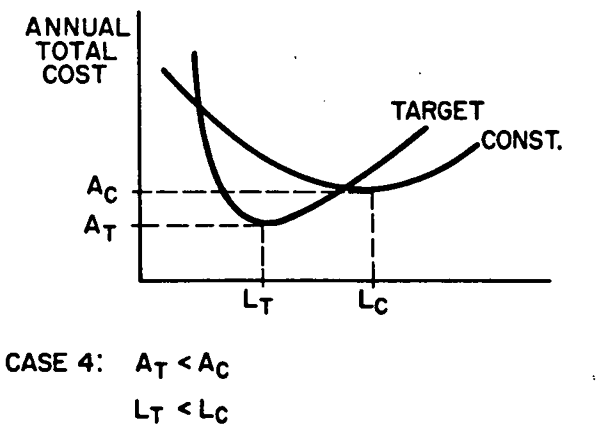


Figure 13. Annual cost versus pavement life, case 4.

where A_c = annualized total cost at economic life of as-constructed pavement, A_T = annualized total cost at economic life of target pavement, L_c = economic life of as-constructed pavement, and i = interest rate.

The hypothetical examples shown in Figures 10 through 13 are summarized in Table 1 and represent extreme values in order to illustrate the need for the criteria stated above, especially the fifth criterion, which limits payment adjustments. Clearly, case 1 results in an excessive positive payment adjustment (bonus) and an upper limit, for example, 10 percent of the bid, is needed. Similarly, the payment in case 2 may reflect such inferior construction that the work should be rejected with no payment allowed.

The use of annual cost as the basis for developing the payment schedule has several advantages: (1) The payment schedule is based on the total costs (construction, agency, and user) estimated from the predicted pavement performance. (2) Failure levels of roughness, rutting, and cracking do not have to be arbitrarily defined. Instead, the economic life determines which combinations of different distress modes will cause failure. For example, suppose the failure level for PSI is 2.0, for rutting is 1.0 in., and for cracking is 25 percent. If the PSI was 2.1, rutting was 0.9 in., and cracking was 24 percent, the road would not have failed according to the arbitrary limits. However, the economic life will indicate failure when the annual rise in user and maintenance costs overshadows the decline in amortized construction costs. This will be true regardless of whether or not any individual distress mode indicates failure. (3) A design period (e.g., 20 years) does not have to be arbitrarily established. Instead, the economic life will indicate when rehabilitation should occur.

Table 1. Four different scenarios to demonstrate a comparison of annualized total cost with economic life of pavement. Note: Payment is calculated using Eq. 2, where bid price = \$10.00/yd^2 and i = interest rate at 5%.

Case No. 1: As-constructed cost less than target cost and as-constructed life greater than target life (Figure 10).

A_T = \$2.25/yd^2 L_T = 10 yr
A_C = \$1.50/yd^2 L_C = 15 yr

Payment = \$10.00 + \$7.78 = \$17.78/yd^2

Case No. 2: As-constructed cost greater than target cost and as-constructed life less than target life (Figure 11).

A_T = \$1.50/yd^2 L_T = 15 yr
A_C = \$2.25/yd^2 L_C = 10 yr

Payment = \$10.00 - \$5.79 = \$4.21/yd^2

Case No. 3: As-constructed cost less than target cost and as-constructed life less than target life (Figure 12).

A_T = \$2.25/yd^2 L_T = 15 yr
A_C = \$2.00/yd^2 L_C = 10 yr

Payment = \$10.00 + \$1.93 = \$11.93/yd^2

Case No. 4: As-constructed cost greater than target cost and as-constructed life greater than target life (Figure 13).

A_T = \$2.00/yd^2 L_T = 10 yr
A_C = \$2.25/yd^2 L_C = 15 yr

Payment = \$10.00 - \$2.59 = \$7.41/yd^2

NOTE: Payment is calculated using Equation 2, where:

Bid Price = \$10.00/yd^2
i = interest rate at 5%

Demonstration of the Conceptual Framework and Payment Schedule

In order to demonstrate the PRS conceptual framework and the proposed framework for the payment schedule, a computer program was developed that relates M&C variables to annual cost. The flow of the program follows path A-B-E-F-G-H as described in Figure 3. A flow diagram for the program, which has been named PERSPEC, is presented in Figure 14. A complete listing and a more detailed description of the program are included in Appendix E.

The program uses relatively straightforward pavement analysis and design algorithms and is not meant to be an example of sophisticated pavement analysis. It is, however, a demonstration of the conceptual framework and the development of a payment schedule based on performance-related criteria. Any of the modules in the program (pavement analysis, performance prediction, etc.) can be replaced with more elaborate algorithms as they become available. As improved models are developed, especially as part of the SHRP A-005 research project, they should be incorporated into the PERSPEC program.

The basic procedures used by the PERSPEC program are summarized as follows. The program:

1. Allows the user to input M&C values from which FMRVs are calculated. In its current form, the Witczak equation is used to calculate the complex modulus of the asphalt layer (see Eq. 4 as presented later in this chapter)(*4*). M&C variables are: (a) percent aggregate passing No. 200 sieve, (b) percent air voids, and (c) percent asphalt by weight of mix.
2. Accepts other input for: (a) thickness of surface layer, in.; (b) thickness and elastic modulus of base layer, in.; (c) 18-kip ESAL's, initial design value; (d) percent annual traffic growth; (e) total traffic; (f) bid price, $; and (g) interest rate.
3. Calculates subgrade strain using program ELSYM5.
4. Calculates annual performance as a function of traffic with: (a) PSI as a function of subgrade compressive strain; (b) rutting as a function of subgrade compressive strain; (c) cracking as a function of asphalt tensile strain.
5. Uses a straight-line relationship between performance and cost, assuming no cost immediately after construction and: (a) $1.75/yd^2 maintenance cost at rutting failure level; (b) $0.50/yd^2 maintenance cost at roughness failure level; (c) $1.00/yd^2 maintenance cost at cracking failure level; (d) $0.10/veh/mile user cost at rutting failure level; (e) $0.05/veh/mile user cost at roughness failure level; and (f) $0.01/veh/mile user cost at cracking failure level.
6. Calculates the total costs on an annualized basis, and sets the minimum equal to the economic life. These values were selected for the purpose of illustrating the program and may be changed as desired by the user of the program.

To demonstrate the use of the PERSPEC program, three runs were made using the hypothetical data in Table 2. One of the runs represents a job with the construction (M&C) variables as the target values. The other runs represent the same job but with the values for the M&C variables above and below the target values. Using a bid price of $10/yd^2, the results given in Table 3 were obtained (refer to Appendix E for detailed output).

Figures 15 through 17 show graphically the output from the PERSPEC program for the three conditions and illustrate how the individual costs of initial construction, user cost, and maintenance cost combine to create the total cost curve for each of the three examples. It is important to emphasize that these figures are only examples using hypothetical data. The examples do, however, illustrate the validity of the concept and give assurance that reasonable payment values may be obtained with the PRS conceptual framework and payment schedule developed as part of this project.

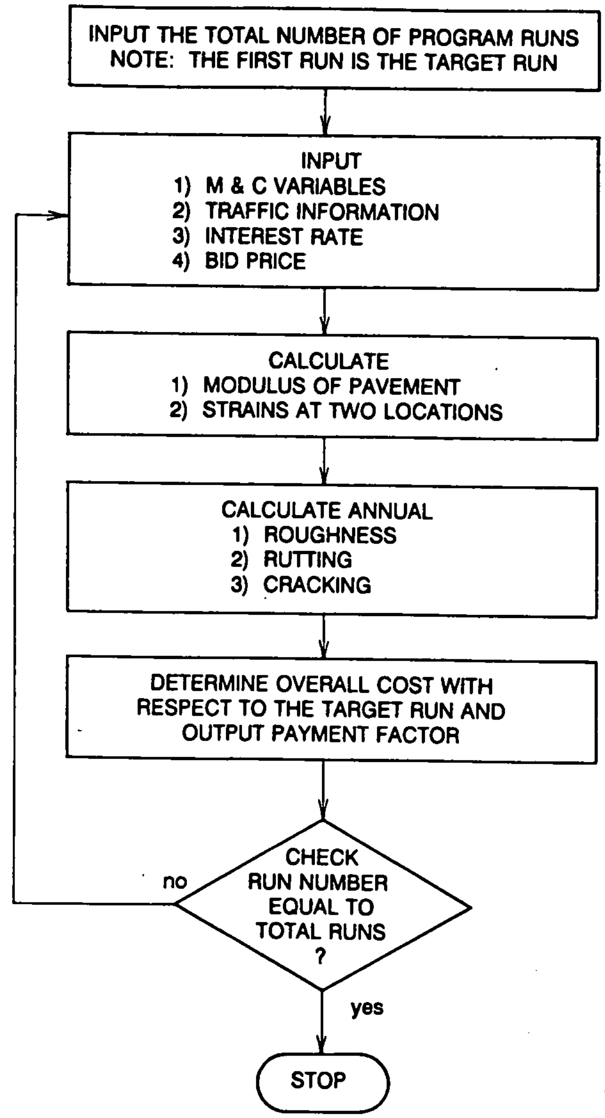


Figure 14. Flow diagram for PERSPEC computer program.

Table 2. Three sample runs of the PERSPEC program.

Run No.	Condition	Percent Passing No. 200 Sieve	Percent Air Voids	Percent AC
1	target	7	5.5	6.5
2	below target	4	3	5
3	above target	10	8	8

Table 3. Payment values as determined by PERSPEC.

Run No.	Condition	Annual Cost ($1/yd^2)	Pavement Life (yr)	Payment ($1/yd^2)
1	Target	3.21	4	10.00*
2	Below Target	3.96	4	7.36
3	Above Target	2.33	6	11.00**

* Bid price

** Payment calculated by PERSPEC algorithm was $14.51/yd^2. An upper limit of the bid price plus 10% was used to limit the payment to $11.00/yd^2.

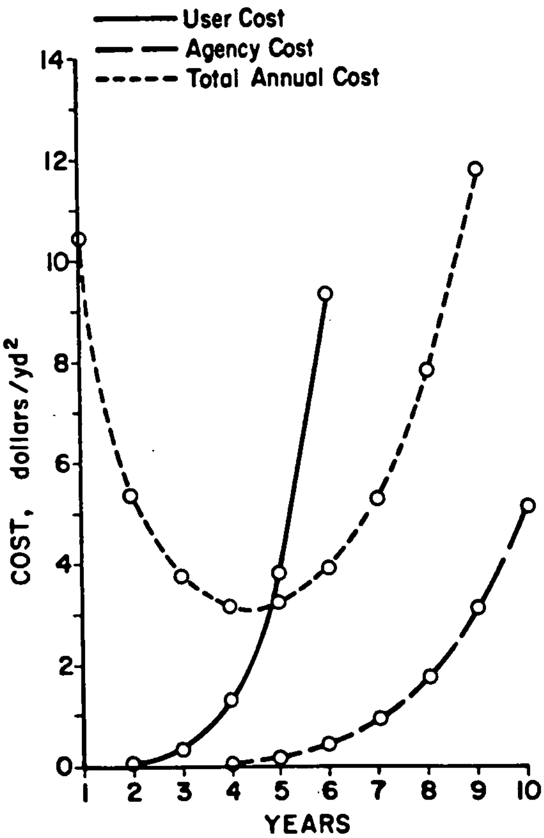


Figure 15. Predicted cost with M&C variables at target levels.

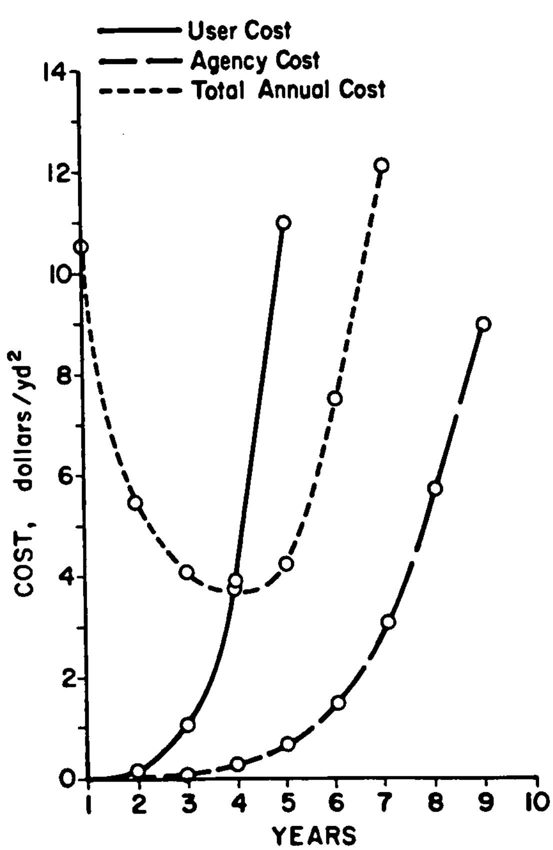


Figure 16. Predicted cost with M&C variables below target levels.

DEVELOPMENT OF ALGORITHMS THAT RELATE MATERIALS AND CONSTRUCTION AND FUNDAMENTAL MIXTURE RESPONSE VARIABLES FROM LABORATORY STUDIES

Algorithms that can be used to predict values for the FMRV from M&C variables are an important element of the conceptual framework for performance-related specifications. Such algorithms exist in the literature; however, they do not, in any systematic way, account for the effects of deviations in the values of the M&C variables from the values for design (target) mixes. A review of selected models that relate M&C variables to FMRV is given in Appendix B. The models that were found in the literature do not specifically account for the effect of departures in the M&C values from optimum or design values.

The hypothetical example in Figure 18 is shown to emphasize the difference between the effect of asphalt content on mixture modulus when the different levels of asphalt content are the result of different mixture designs versus nonconformance with the target level. The first effect, resulting from different target mixtures, is demonstrated by using the equation developed by Witczak and others, which is used in a number of design procedures to predict the complex modulus, $|E^*|$, from M&C variables. This model predicts the compressive complex modulus, $|E^*|$, from the M&C variables, percent minus No. 200, percent air voids, and percent asphalt cement in the mix:

$$\begin{aligned}\log(|E^*|) = {} & 5.553833 + 0.028829 \frac{P_{200}}{f^{0.17033}} \\ & - 0.03476 V_v + 0.070377 \eta + \\ & 0.000005\ t_p\ (1.3 + 0.49825 \log f)\ P_{ac}^{\ 0.5} - \\ & 0.0189\ t_p^{\ (1.3 + 0.49825 \log f)} \frac{P_{ac}^{\ 0.5}}{f^{1.1}} + \\ & 0.931757 \frac{1}{f^{0.02774}}\end{aligned} \qquad (4)$$

where $|E^*|$ = dynamic modulus, lb/in.^{2}; P_{200} = percent aggregate passing No. 200 sieve; f = loading frequency, Hz; V_v = percent air voids; η = asphalt viscosity at 70 °F, 10^6 poises; P_{ac} = percent asphalt by weight of mix; and t_p = temperature, °F.

The second effect, nonconformance for a given mix wherein the as-constructed asphalt content deviates from the design or target mix, is also shown in Figure 18.

Clearly, the relationship between the M&C variables and FMRV (air voids versus modulus) is considerably different for the two sources of variation in asphalt content. The latter scenario, which accounts for contractor nonconformance, must be included in the database that is used to develop relationships between M&C variables nonconformance and the FMRV.

Based on the foregoing discussion, if the algorithms that relate M&C variables to FMRV are to be used to predict the effect of M&C nonconformance on the FMRV, as required in a PRS, the database used to develop the models must contain a carefully selected range of nonconforming mixes. The purpose of the laboratory study was to demonstrate how such a database can be created and how algorithms can be developed from such a database.

A carefully designed experiment plan is essential for generating the database that is necessary for developing the algorithms (mathematical models) that relate the M&C variables and FMRV. In the process of creating the experiment plan, close cooperation between the materials engineer and the statistician is required to:

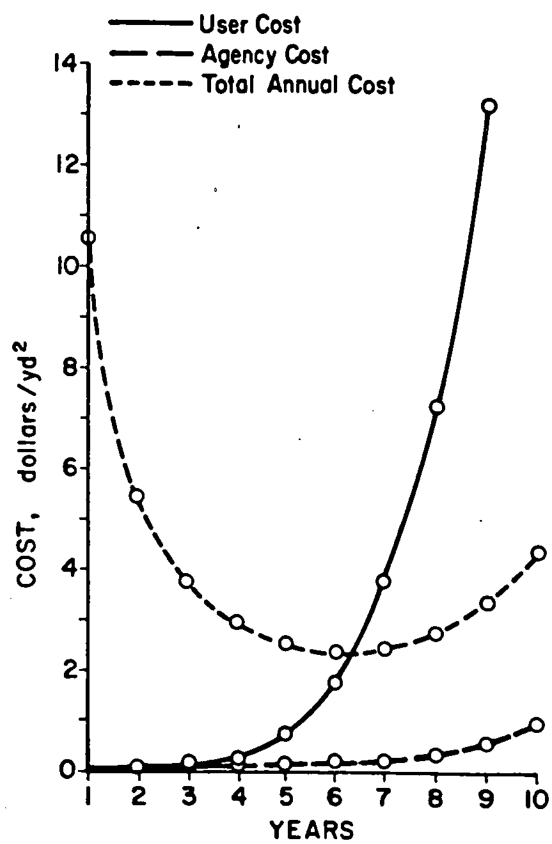


Figure 17. Predicted cost with M&C variables in excess of target levels.

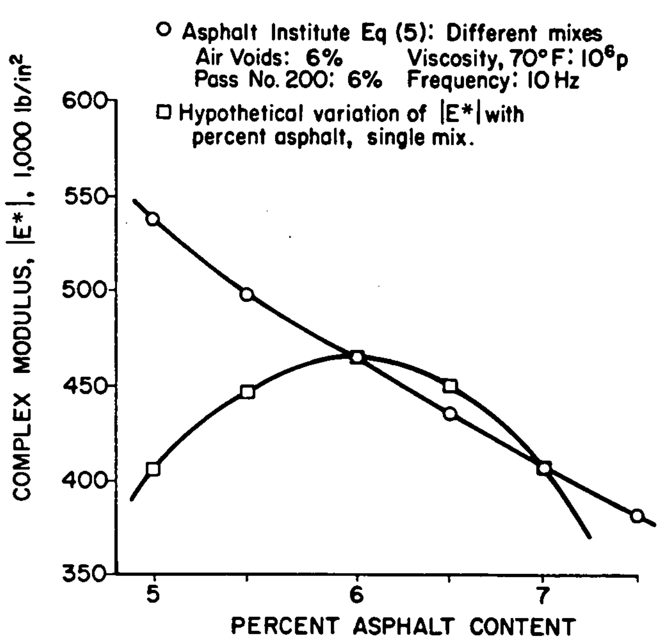


Figure 18. Variation in $|E^*|$ with percent asphalt; single mix at various levels versus different mixes.

- Identify the FMRV that are related to pavement performance and, therefore, should or should not be included as the dependent variables in the algorithms.
- Select the M&C variables that, based on engineering judgment, are most likely to affect the FMRV that are of interest.
- Develop a plan for preliminary experiments that are needed to determine experimental error variance.
- Develop plans for the primary experiments, after conducting the preliminary experiments.
- Analyze the data to generate the required relationships between the M&C variables and FMRV.

DEVELOPMENT OF THE LABORATORY TESTING PROGRAM

As part of the project a laboratory study was conducted to demonstrate a procedure for developing a database that can be used to develop new or validate existing relationships between the M&C variables and the FMRV when the database contains nonconforming ("out of specification") mixes. The first step in the study was to select the FMRV of interest. A detailed description of the experiment plan for the laboratory testing is presented in Appendix F.

Selection of Study Variables

Diametral complex modulus, tensile strength, and creep and fatigue properties were selected as the FMRV. Complex modulus was chosen because it is used in several commonly used design procedures (*7, 8*); tensile strength, because it is used to predict thermal cracking (*9, 10*); creep properties, because they are used in the VESYS prediction equation (*11, 12*); and fatigue properties, because they are used in the VESYS equation and, also because fatigue is a primary cause of distress in hot-mix asphalt pavements. Although a number of testing configurations are available for measuring the selected properties, the diametral test was chosen because it was being considered for adoption in a companion NCHRP study (*3*) and because all four FMRV selected for the laboratory study can potentially be measured on specimens with this specimen geometry.

Standard 4-in. diameter by 2.5-in. thick Marshall-type specimens were used in all of the testing. Air void levels in the test specimens were controlled by varying the number of blows of the Marshall hammer. Results of more recent work indicate that gyratory or kneading-type compaction would have been a preferred compaction method (*3*); however, those results were not available when this study was initiated. In the future, gyratory or kneading compaction should be adopted for the development of any database that will be used to develop relationships between M&C variables and FMRV.

The complex modulus was measured by applying a 10-Hz diametral load to the test specimens. Data for the calculation of $|E^*|$ were obtained after several hundred cycles of loading once the response to the applied load had equilibrated. $|E^*|$ was calculated by dividing the peak-peak diametral stress by the peak-peak diametral strain as follows (*13*):

$$|E^*| = \frac{P}{\delta * t}[0.2692 - \nu(-0.9974)] \qquad (5)$$

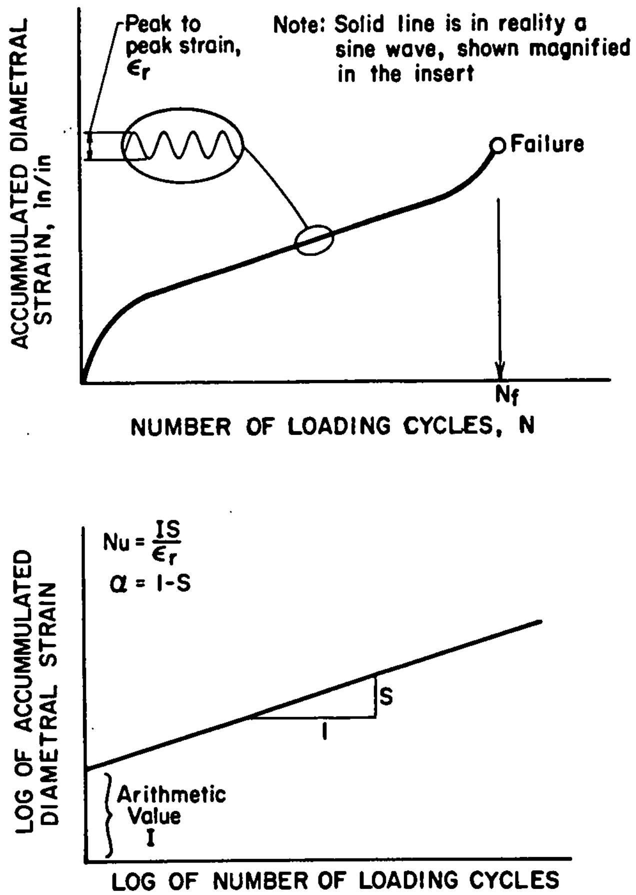


Figure 19. Schematic illustrating fatigue life test data and data reduction.

where $|E^*|$ = complex modulus, lb/in.^{2}; P = peak-to-peak load amplitude, lb; δ = diametral deflection, in.; t = thickness of the specimen, in.; and ν = Poisson's ratio. $|E^*|$ is used to denote the complex diametral modulus and is used herein interchangeably with the compressive dynamic modulus, also called $|E^*|$.

The diametral load was continued until failure, as shown schematically in Figure 19. The technique developed by Kennedy and Anagnos (*14*) was used to determine the creep parameters, alpha and nu, as used in the VESYS performance model. The methods used to calculate $|E^*|$, alpha, nu, and the number of cycles to failure, N_f, are shown schematically in Figure 19.

The fatigue parameters, K_2' and N_2, are defined as the intercept and slope, respectively, of an assumed linear relationship between the logarithm of the applied tensile stress and the logarithm of the fatigue life defined by Eq. 6 (*14*):

$$N_f = K_2' \left(\frac{1}{\Delta\delta}\right)^{N_2} \tag{6}$$

where N_f = fatigue life, $\Delta\delta$ = stress difference; K_2' = the antilog of the intercept value of the logarithmic relationship between fatigue life and stress difference, and N_2 = slope of the logarithmic relation between fatigue life and stress difference. Kennedy (*14*) has reported that using the stress difference instead of the applied tensile stress will shift the fatigue relationships determined from diametral testing so that they are in general agreement with the relationships determined by other testing procedures.

Tensile strength was determined in the diametral mode with a platen speed of 2.0 in./min. The tensile strength was recorded as the maximum tensile stress calculated using the following equation:

$$\sigma_t = \frac{P_f}{t}(0.156) \tag{7}$$

where σ_t = tensile strength, lb/in.^{2}; P_f = load at failure (maximum load), lb; and t = thickness of the specimen, in.

The choice of the independent M&C variables was limited to two materials-related variables, asphalt content and percent passing the No. 200 sieve, and one construction variable, percent air voids. These were chosen because they are known to affect the chosen FMRV and they can be readily controlled by the contractor. It is also well known that the FMRV are affected by the source of the aggregate and the properties of the asphalt cement. Therefore, a crushed dolomite and an uncrushed rounded river gravel and two AC-20 asphalt cements were also included in the study. The crushed dolomite is quarried in State College, Pennsylvania, and is used locally in high quality mixes (*15*). The river gravel, obtained from a source near Pittsburgh, Pennsylvania, contained less than 20 percent crushed faces. The gravel contained very little minus No. 200 material; therefore, the dust of fracture from the crushed dolomite was used as a mineral filler for the river gravel. Properties of the asphalt cement are given in Table 4. Additional details regarding the development of the laboratory testing program are given in Appendixes F and G.

Testing Program and Specimen Preparation

The Marshall mix design procedure was used to establish the optimum asphalt content for the four mixes—two aggregate sources and two asphalt sources. In order to develop useful relationships between the FMRVs and M&C variables in the performance model, it is essential to include in the laboratory experiment a range of the levels of the M&C variables that corresponds to what is expected in the field. Construction records and existing hot-mix acceptance data available from highway agencies are a valuable source of information for setting the range of nonconformance that should be included in the laboratory study. Data obtained from PennDOT were used in the selection of the range of the M&C variables used in the laboratory study.

The range for the M&C variables must be selected carefully. Should the selected range be too narrow, predictions of the FMRV where the M&C variables lie outside the range would not be appropriate. It is important to note that in order to develop the complete relationship, treatment levels lying to both sides of the target values of the dust content, air voids, and asphalt content should be selected. However, for this limited demonstration study, only two of the possible three treatment levels for each of the three M&C variables were selected. As noted later, this severely limited the ability to consider other than linear effects. The study performed was a half-replication of the complete factorial experiment, i.e., only 16 of the 32 possible treatment combinations were examined. In order to provide a comparison with mixes that contained all the variables

Table 4. Summary of laboratory test results.

Mix No.	Aggregate Type(1)	Asphalt Type(2)	Passing No. 200 (%)	Asphalt Content (%)	Air Voids (%)	|E*|, 1,000 lb/ft² Unaged Actual	|E*|, 1,000 lb/ft² Unaged Predicted	|E*|, 1,000 lb/ft² Aged Actual	Tensile Strength (lb/ft²) Unaged	Tensile Strength (lb/ft²) Aged
1	D	L-AI	5.4	5.8	3.3	463	480	512	222	250
2	D	L-AI	5.4	5.0	10.0	372	321	674	158	187
3	D	L-AI	7.5	5.0	4.5	588	564	718	228	264
4	D	H-AI	7.5	5.8	1.9	361	553	592	176	216
5	G	H-AI	5.9	6.6	3.0	210	430	395	137	165
6	D	L-AI	7.5	5.8	5.3	411	436	500	144	165
7	D	H-AI	5.4	5.0	5.3	427	456	481	178	239
8	G	L-AI	8.3	6.6	1.3	332	563	428	137	172
9	D	H-AI	5.4	5.8	7.8	291	316	392	120	148
10	G	L-AI	5.9	5.7	5.2	436	434	541	171	227
11	G	L-AI	5.9	6.6	6.9	332	326	408	107	140
12	G	H-AI	8.3	5.7	2.9	325	559	483	165	196
13	G	H-AI	8.3	6.6	4.0	245	445	300	104	138
14	G	H-AI	5.9	5.7	10.0	260	285	379	99	123
15	G	L-AI	8.3	5.7	7.2	319	413	512	127	168
16	D	H-AI	7.5	5.0	8.4	360	386	439	144	172
17	D	H-AI	5.4	5.8	3.6	411	437	460	173	204
18	G	L-AI	5.9	6.6	2.4	326	468	454	170	189

(1) D - Dolomite, G - Gravel
(2) L-AI - Low Aging Index, H-AI - High Aging Index

at the target levels, two additional mixes were included bringing the total number of mixes to 18. A more complete description of the experiment design and the rationale used to develop it is given in Appendix F.

After an examination of typical PennDOT acceptance data, the treatment levels were selected as follows: percent asphalt cement: target and target minus 15 percent of target asphalt content percentage; percent air voids: target and target plus 4 percent air voids; percent dust: target percentage and target plus 40 percent of target percentage passing No. 200 sieve.

To determine the required number of Marshall blows that would result in the plus 4 percent air voids, a plot of measured air voids versus number of blows was generated. This plot was then used to determine the number of blows required to achieve the desired percent air voids in the test specimens. Once the specimens had been compacted, they were allowed to cure at room temperature for 4 days to allow for the early development of steric hardening. A flow diagram showing the manner in which the specimens were tested is shown in Figure 20. Further details regarding the testing procedures and the complete set of test results are presented in Appendix G.

MODEL DEVELOPMENT

Models that relate nonconformance in the M&C variables to the performance-related FMRV may be generated in several ways. Existing models that relate M&C variables to FMRV may be used directly without modification, or, more appropriately, they may be validated using a new database developed especially for validation purposes. Such databases should be generated us-

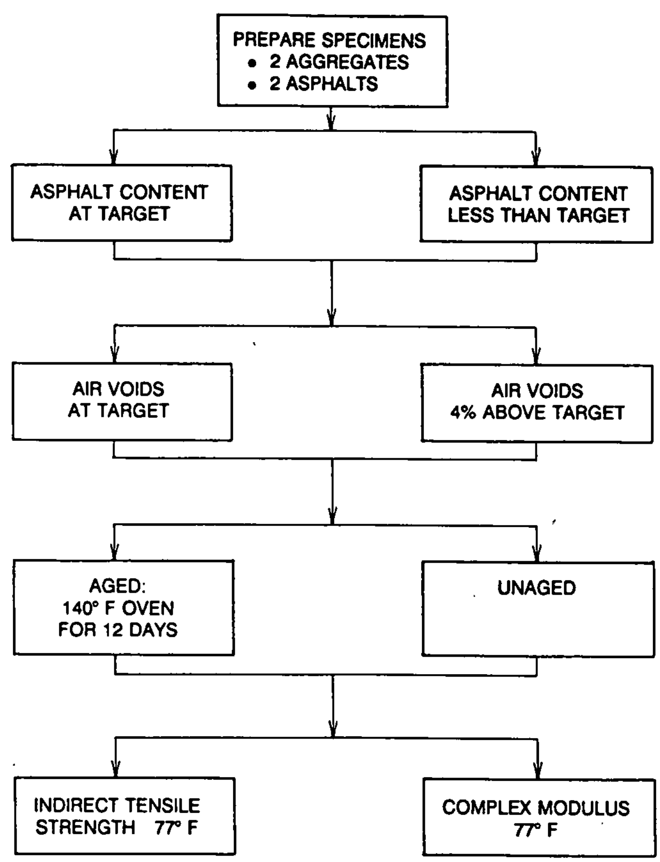


Figure 20. Flow diagram for laboratory testing program.

ing a well-designed experiment plan and a sufficient range in the nonconforming values for the M&C variables (Appendix F). Alternatively, such databases may be used to develop new models that are specifically designed to account for nonconformance. Both approaches are discussed below.

Validation of Existing Models

A brief review of existing models that relate M&C variables and FMRV is presented in Appendix B. Of particular note is the model developed at the Asphalt Institute that relates the complex modulus, $|E^*|$, to M&C variables (*4, 16*). This model, Eq. 4, as discussed earlier, contains the following variables: P_{200} = percent aggregate passing No. 200 sieve; f = loading frequency, Hz; V_v = percent air voids; η = asphalt viscosity at test temperature, 10^6 poises; P_{ac} = percent asphalt by weight of mix; and t_p = test temperature °F.

The laboratory experiment was designed to include P_{200}, V_v, P_{ac}, and η as M&C variables. The test frequency and test temperature were held constant at 10 Hz and 77 °F, respectively. The test specimens were aged by storing them in a 140 °F forced-draft oven for 12 days. Further details of the test conditions and protocol are presented in Appendix F.

In its original form the complex modulus, $|E^*|$, was measured using 4-in. diameter by 8-in. high cylinders loaded in compression. In the current study, diametral specimens, 2.5 in. thick by 4 in. in diameter, loaded diametrically, were used to measure $|E^*|$. The average measured complex modulus and the modulus predicted by Eq. 4 are given in Table 4. A review of the data in Table 4 reveals that Eq. 4 appears to provide reasonable estimates of the modulus except for those mixtures using the gravel aggregate. This is shown graphically in Figure 21 where the predicted values of $|E^*|$ are plotted versus measured values of $|E^*|$, producing an R^2 (see Appendix A) of 0.31. The gravel mixtures yield measured values that are generally less than the predicted values. This is not unexpected, given the rounded nature of the gravel aggregate. Based on these results, any further development of Eq. 4 should include variables that describe the properties of the aggregate. Such properties might include surface roughness, angularity, flakiness index, or a measure of interparticle friction. In addition, the original model was developed using compressive loading, whereas the current study used indirect tension. This could also account for the poor predictions afforded by the model (Eq. 4). The gravel mixture may be more sensitive to the type of loading than the crushed dolomite mix. These results demonstrate some of the difficulties that can be encountered when a model developed for one purpose is applied to a different situation.

A further discussion of the statistical analysis of the data can be found in Appendix G where the results of various regression analyses are shown. These analyses indicate that when the nonconforming mixes were removed from the database, the modulus predictions were greatly improved. When the gravel mixes were removed from the database, the predictions were further improved.

Development of New Models

Data from the complex modulus, $|E^*|$, tensile strength, creep coefficients alpha and nu, and fatigue parameters, k_2' and N_2, were available for developing models that relate M&C variables to FMRV. Considerable difficulty was encountered with the measurement of alpha and nu and, as a consequence, the fatigue parameters k_2' and N_2. Therefore, these were not used in the model development and, instead, the demonstration focused on the complex modulus and tensile strength data.

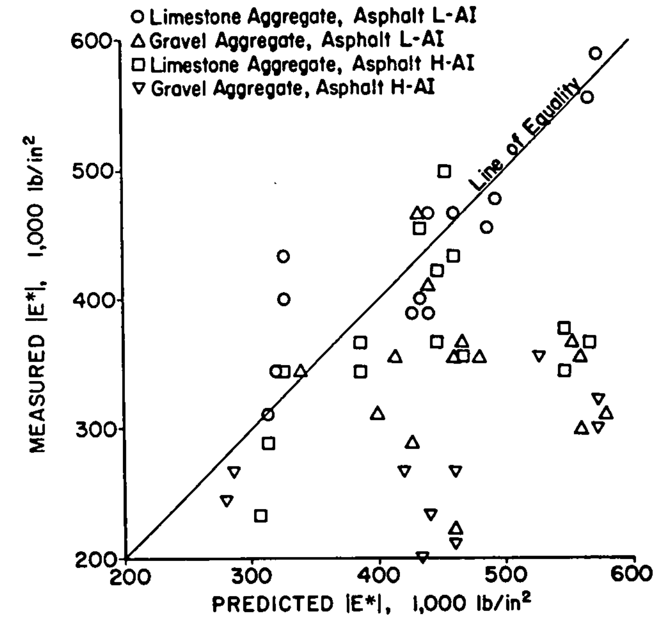


Figure 21. Measured versus predicted value of $|E^*|$.

Regression was used to develop relationships between $|E^*|$ and tensile strength and the M&C variables: P_{200}, V_v, η, and P_{ac} (*17*). Aggregate type, asphalt source, and aging were also included as indicator variables when appropriate. Because the experiment was designed using a 2^5 partial factorial design, it was difficult to extract other than linear effects from the data. As discussed elsewhere in this chapter, a number of the factors should have been included at three or more levels so that nonlinear effects could have been studied.

Using a simple linear model, an R^2 value of 0.47 was obtained when the complex modulus was predicted from the entire data set. For this model, the coefficient of variation was 22 percent. This simple linear model actually gave a better prediction than the Asphalt Institute equation (Eq. 4), which yielded an R^2 value of 0.31.

Significantly better correlation was obtained when the tensile strength data were used as the response variable. Using a model that incorporated only P_{200}, asphalt type, V_v, and aging as predictor variables, an R^2 value of 0.87 was obtained with a coefficient of variation of 11 percent. This result is very encouraging when no aggregate- or asphalt-specific variables (other than an indicator variable) were used in the model. The improved R^2 values, as compared to those obtained for the models that predict $|E^*|$, are most likely the result of the smaller testing error associated with the indirect tensile strength test than with the $|E^*|$ test procedure. The reported problems with the complex modulus test device may be attributed to the particular test device, although the repeatability of the procedure itself is also suspect.

The larger R^2 values reported for the indirect tensile strength test for the complex modulus may not necessarily point in favor of the indirect tensile strength. Instead, it may simply indicate that the tensile strength is less sensitive to the M&C variables and that, in order to properly predict $|E^*|$, more aggregate and asphalt variables are needed or a more robust database is needed.

The modeling described above includes the target (design) and nonconforming ("out of specification") data in a single database. Equation 1 was presented as a generic model that specifically accounts for nonconformance or nonspecification mixtures. Attempts to apply this model to the data set for either $|E^*|$ or tensile strength were not successful, primarily because of a lack of a sufficient number of levels in the response variables. In retrospect, a different experiment should have been designed if this approach was to be effectively pursued.

APPLICATION OF SENSITIVITY ANALYSIS

In order for performance-related specifications to be adopted and successfully implemented, the models that predict pavement performance must be well understood and must provide reasonable results. An excellent methodology for measuring and evaluating the effects of independent variables upon their associated response variables is a sensitivity analysis. This process involves selecting ranges of values for the independent variables and using the model to determine the values of the dependent response variable. The results indicate the "sensitivity" of the response variable to changes in the individual independent variables. Usually, a sensitivity analysis is performed using statistical methods of experimental design, such as a factorial examination of all independent variables. This involves selecting two or three levels of values for each independent variable, and determining the response value for all combinations of all independent variables. This "full factorial" approach results in 2^n or 3^n determinations of the dependent variable for two or three levels of n independent variables, respectively. A full factorial permits the use of statistical calculations and analysis of variance techniques in evaluating the effect of the independent variables as well as the interactive effect of combinations of independent variables.

Pavement performance modeling can be divided into two principal segments, identified as A-E (material property to material response) and E-F-G (material response to pavement response to pavement performance). This pavement performance framework (shown in Figure 2) is closely associated with mechanistic/empirical modeling, with the A-E relationship determined empirically through laboratory experiments, the E-F relationship modeled with mechanistic programs such as BISAR, and the F-G relationship modeled empirically through field experiments such as the AASHO Road Test. In order to demonstrate how a sensitivity analysis can be applied to pavement performance modeling, analyses were performed on the two individual segments identified above, A-E and E-F-G.

A-E (Material Property to Material Response) Sensitivity Analysis

For the A-E sensitivity analysis Eq. 5, the relationship used in the laboratory demonstration (described in the previous section) was analyzed. In this relationship, the material response characteristic being estimated is the complex modulus $|E^*|$ of the mix. The following M&C variables were used in the sensitivity analysis: percent passing the No. 200 sieve, percent air voids, and percent asphalt cement.

The variables that were held constant include frequency (10 Hz), temperature (77 °F), and viscosity (107 mp at 70 °F).

Figures 22 through 24 illustrate the effect on the complex modulus of changing the values of the independent variables from the high to low levels. Figure 22 shows the sensitivity of each independent variable when the values of the other variables are held at their low levels. Figures 23 and 24 represent the same information with the other variables held at their middle and high levels, respectively. These figures provide graphic comparisons of the relative effect of the independent variables under different conditions. They indicate that, regardless of the levels of the other independent variables, and for the range of values used in the analysis, the percent passing the No. 200 sieve has the smallest effect on the complex modulus, the percent air voids has a larger effect than the percent passing the No. 200 sieve, and the percent asphalt cement has the greatest effect.

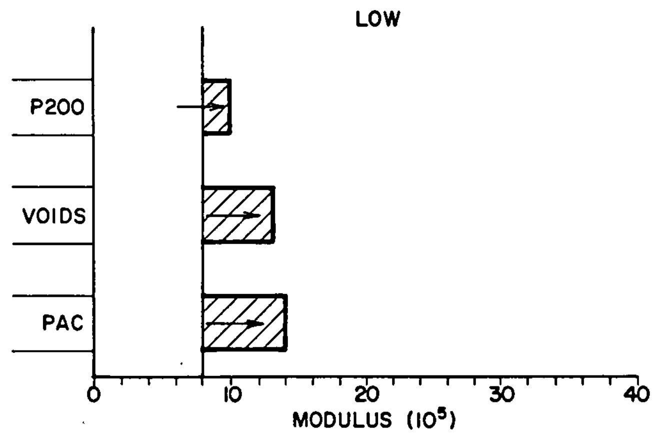


Figure 22. Sensitivity of response variables when independent variables are at their low levels.

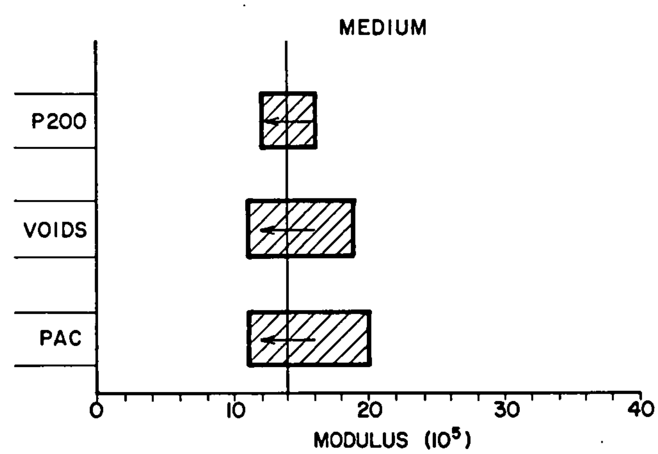


Figure 23. Sensitivity of response variables when independent variables are at their middle level.

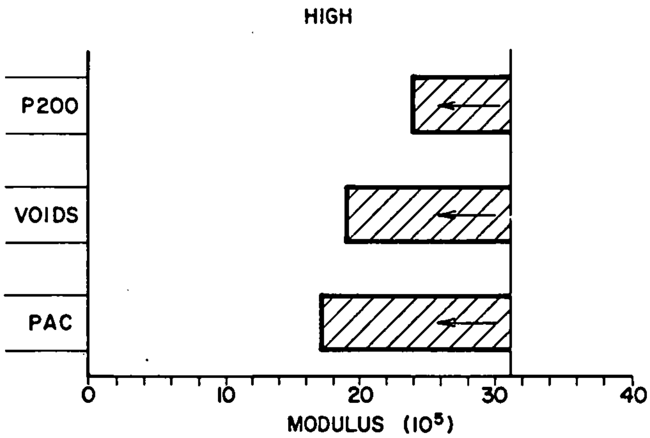


Figure 24. Sensitivity of response variables when independent variables are at their high levels.

E-F-G (Material Response to Pavement Response to Pavement Performance) Sensitivity Analysis

The second sensitivity analysis was applied to the portion of the pavement performance prediction framework that relates material response characteristics (FMRV) to theoretical pavement response (usually an elastic layer structural pavement response calculation), and then empirically correlates the pavement response to observed field performance (levels E-F-G in Figure 2). Because the trend of pavement performance models is toward more comprehensive characterization of the FMRV properties (consideration of viscoelastic and nonlinear stress-dependent material behavior), it was desirable in the sensitivity analysis to select a performance model that included more fundamental material properties. It was also desirable to have a performance model that included roughness as a response variable, inasmuch as user costs are typically determined on the basis of pavement roughness.

Table 5. Levels of factors in the sensitivity analysis.

Factor	Levels	Units
1. Initial present serviceability index (PSI_i)	3.6; 3.9; 4.2	-
2. Asphalt concrete modulus (E_{ac})	3,000; 450,000; 600,000	psi
3. Asphalt concrete thickness (T_1)	3; 5; 7	inches
4. Granular Base thickness (T_2)	4; 7; 10	inches
5. Granular Base k_1	3000; 6000; 9000	-
6. Granular Base k_2	0.20; 0.50; 0.80	-
7. Subgrade m_1	10,000; 20,000; 30,000	-
8. Subgrade m_2	-1.00; -0.60; -0.20	-

For the reasons given above, a performance model was analyzed that considered the nonlinear stiffness behavior of the unbound pavement layers, and used pavement roughness as the response variable (*18*). The variables and their values that were used in the analysis are given in Table 5. As a result of the more complicated nature of the sensitivity analysis for this segment, the detailed description is presented in Appendix C. The appendix includes figures that illustrate the sensitivity of the response variable as the independent variables are changed from low to high values.

Other Considerations

In a sensitivity analysis, the objective is to measure the effects of certain independent variables on the response-dependent variable. However, when predicting pavement performance there are several sequential steps that affect the final pavement performance estimate. This indicates that not only do individual modules have an effect on the final result, but also the prediction error that is involved in the early steps of the framework, because this error is carried through the later steps.

The information obtained from evaluating the error terms is important, since it may indicate that certain modules have more prediction error than the sensitivity of the independent variables within the module. This evaluation can then be used in combination with the results of the sensitivity analysis to determine the proper levels of data precision, and how accurately the results from the model can be interpreted. A detailed example evaluation of this type is presented in Appendix D, where the variability in the design factors is considered for a pavement design model.

Use of Stochastic Variables in Sensitivity Analyses

The M&C variables are stochastic in nature and should be treated as such in a sensitivity analysis. The two rather simplistic sensitivity analyses given above are useful for testing the reasonableness of prediction models; however, in the implementation of a performance-related specification, the M&C variables must be considered as stochastic variables.

Figure 25 illustrates the selected procedure for evaluating the effects of departures from design on predicted pavement performance, and for assessing payments associated with various levels of conformance to design specifications. In the procedure, design (job-mix formula) values for M&C variables such as binder content, air voids content, and percent passing the No. 200 sieve are input to an algorithm that relates M&C variables to fundamental response variables such as asphaltic concrete mix modulus, creep compliance, and fatigue behavior. M&C variables are treated as stochastic variables in recognition of the fact that, in actual practice, these variables do not take on deterministic values but instead are defined by statistical distributions characterized by parameters such as means and variances. Estimates for these statistical parameters are determined by the contractor from a knowledge of the contractor's process variability.

In this simulation the M&C variables are assumed to be normally distributed, characterized by certain means and variances. From each distribution, *n* different uniform random values are sampled. For each of the *n* different combinations generated, a payment factor is generated using the procedure shown in Figure

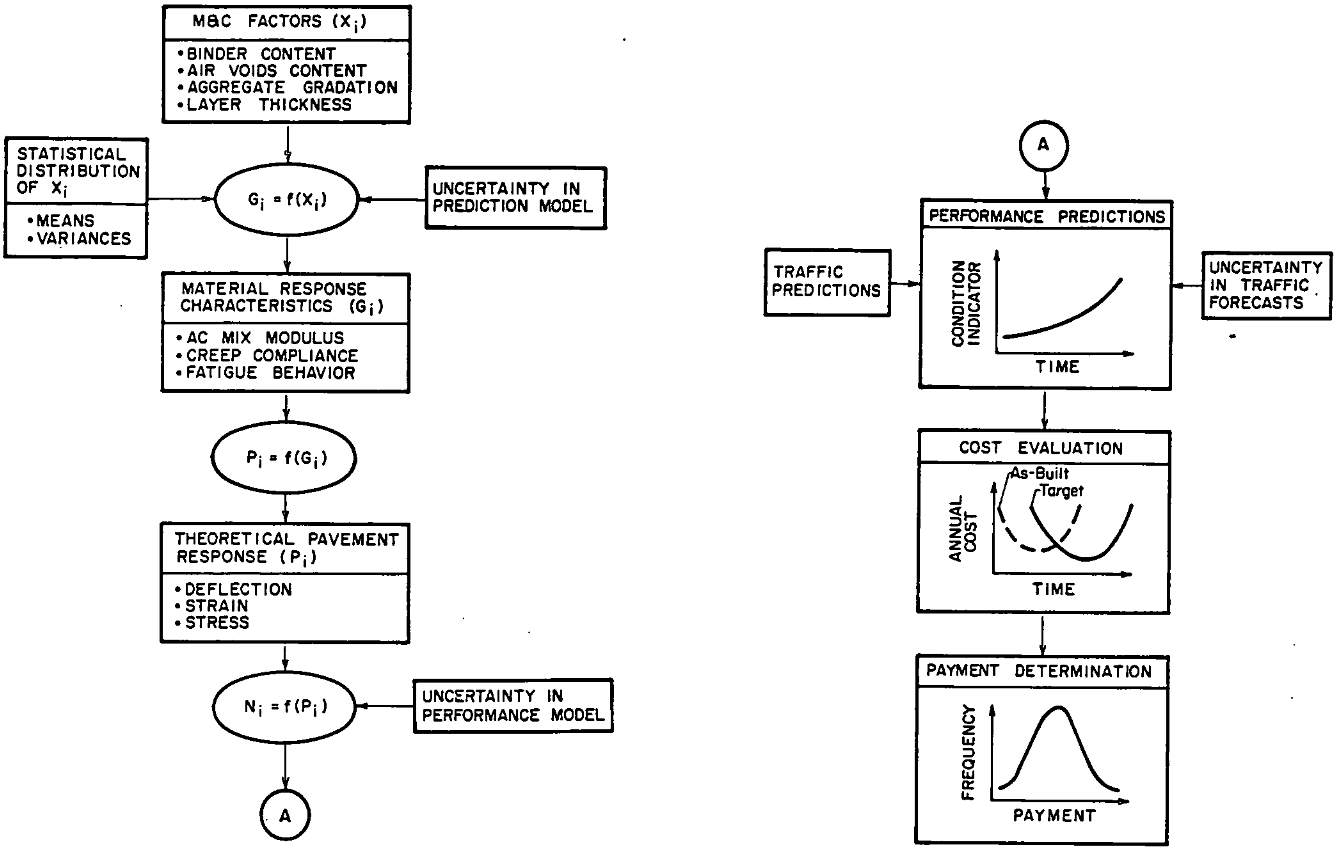


Figure 25. Simulation procedure for framework demonstration.

25. Consequently, a distribution of payments is obtained from which statistical parameters such as the mean and variance can be determined.

For each combination of sampled values for asphalt content, air voids content, and percent passing the No. 200 sieve, the asphalt concrete mix modulus is calculated using the selected A-E relationship. Therefore, there are *n* different values for the asphaltic concrete mix modulus. Each mix modulus value is then paired with a sampled value for asphaltic concrete thickness, and multilayer linear elastic theory is used to calculate the pavement structural response variables required for the performance predictions. The performance models recommended for the demonstration require the application of linear elastic layer theory for the calculation of pavement response.

Because the model for predicting $|E^*|$ and the performance models were developed using regression analysis with observed data, there are uncertainties associated with the predictions. These uncertainties are connected with the lack of fit and pure errors associated with the regression equations. It would be interesting to evaluate how the performance predictions and the distribution of payment factors are influenced by these uncertainties. However, for the purpose of estimating a payment factor associated with a given level of conformance to specifications, it is not necessary to consider the uncertainties in the prediction models as long as the prediction models produce realistic estimates of the dependent variable. In practice, the models used in the algorithm for performance-related specifications will be clearly stipulated in the contract documents. Consequently, as long as the contractor understands that payments will be evaluated using specified models or procedures, the payments generated should be legally binding. For the framework demonstration, therefore, the predictions from the A-E equation and the performance models were used directly in the development of a payment schedule. The uncertainties associated with the various regression equations were not considered in the development of this payment schedule.

A CRITICAL ANALYSIS OF OBSERVATIONAL AND EXPERIMENTAL DATABASES VERSUS REQUIREMENTS FOR IMPLEMENTING PERFORMANCE-RELATED SPECIFICATIONS

Databases can usually be classified as either observational or experimental, according to whether the data were obtained simply by observing things of interest in ongoing projects (observational) or as the result of an experiment especially designed to produce the database (experimental). Data obtained from the AASHO Road Test are a good example of an experimental database where the data were obtained under closely controlled experimental conditions. An assembly of the skid resistance, Mays meter, and falling weight deflectometer data in a pavement management system constitutes an observational database. Both types of database can be useful; each has limitations and advantages as described next.

Observational Databases

The field performance of the many highways constructed in the last 20 years in the United States would seem to provide a very rich database from which to evaluate the importance of the many factors known to influence the performance of highways. These data should be available from state highway agencies and, on a per-observation basis, should be relatively inexpensive to obtain. In many states, these types of databases have been assembled in a pavement management system. Another source of observational data is the quality assurance data that many states have computerized.

Advantages and Disadvantages of Observational Databases. The limitations as well as the usefulness of observational data must be carefully evaluated before assuming unreasonable expectations from the study of such databases. Many of the data limitations are the result of the fact that the "good" roads tend to have "good" values for all of the factors important to their field performance and the "bad" roads tend to have "bad" values for several of these factors. This leads to a database that is useful for predicting the performance of highways constructed in the same manner as the ones in the database, but is of only marginal value for evaluating the importance of the various factors known to influence field performance. The mathematician refers to this as the problem of multicolinearity in the database.

The problem of multicolinearity occurs to some extent in all observational databases. This can be illustrated most conveniently in a very simple setting in which there are, supposedly, two important factors, A and B, which may influence the value of some response of interest, y. If one has a database that might result from always using combinations with either high A and high B values or low A and low B values, then no matter how many of these data points may be observed (planned or otherwise), one will never know whether it is A or B or their interaction that is producing the observed effects in response y. The predictions of responses at the two points (and perhaps on the line between them) may be quite good, but the predictions will be very unreliable when one moves away from the line connecting them. In this sense, the ability to predict the points in the database may give a very false idea of how well one can predict observations for other values of the factors. For this example it is clear that observations with high A and low B and also low A and high B are needed. In other words, one needs to at least fill in the other corners of the design, something that by its very nature is not likely to occur in an observational database. It will be noted later, following discussion of experimental databases, that observational databases may play a very important role in the model building and evaluation process, though not to the exclusion of the requirement for well-designed experimental databases.

There are advantages and disadvantages to using an observational database. The principal advantages may be summarized as follows: (1) It may be relatively inexpensive to obtain the data. (2) In the process of collecting data, it is not necessary to interfere with the construction process, or the interference can be kept to a minimum. (3) As long as future projects are constructed with approximately the same materials and procedures as in the past, their performance can be predicted fairly well.

The disadvantages of an observational database are: (1) If the database is not robust, it may be impossible to establish meaningful relationships between the dependent (e.g., FMRV) and independent (e.g., M&C) variables. (2) Prediction equations will not be useful outside the usually small region in which the variables were observed, and the effect of M&C nonconformance will be poorly characterized. (3) Prediction equations developed from an observational database may be highly reliable for predicting the performance of projects constructed about the same as those in the past; but, the equations should be used cautiously to suggest methods for improving the performance of future projects constructed outside the range of observations.

Table 6. Hypothetical observational database.

y	x_1	x_2	x_3
14.5	-1.0	-1.0	-1.0
15.2	-1.1	-1.0	-1.0
11.5	-0.9	-1.0	-1.0
14.0	-1.0	-1.1	-1.0
11.1	-1.0	-0.9	-1.0
-1.00	0.0	0.0	0.0
0.583	-0.1	-0.1	0.0
-0.344	0.1	0.1	0.0
-0.039	0.0	0.0	0.0
1.034	0.0	0.0	0.0
25.87	1.0	1.0	1.0
30.0	1.1	1.0	1.0
26.6	0.9	1.0	1.0
28.3	1.0	1.1	1.0
26.1	1.0	0.9	1.0

An Example of Limitations of Observational Data. An example will be used to demonstrate the difficulties in using observational data to study the effects of the important factors on a response of interest. First, assume that the following hypothetical model represents the true effect of x_1, x_2, and x_3 on y:

$$y = x_1 + 2x_2 + 3x_1x_2 + 4x_3 + 5x_1x_3 + 6x_2x_3 + 2x_1^2 + 2x_2^2 + 2x_3^2 + \epsilon, \quad \epsilon \sim N(0, \sigma_2) \tag{8}$$

Data generated with this model (see Table 6) have a high level of multicolinearity, or in other words, the x's are highly related to each other. For example, if x_1, x_2, and x_3 are regarded as coded values of the asphalt content and the thickness of two layers of the roadway, at the low levels (near -1 values), one would have some poor roads; at or near the 0 level, some medium quality roads; and some very good roads with coded values around 1. Note that the database in the example has all low x's, all medium (near 0) x's, or all high x's. These represent approximately what one should expect to find in an observational database.

Using the data in Table 6, the researchers attempted, using regression analyses, to estimate the parameters in a model of the same form as Eq. 8, the equation used to generate the data. Regressing the data in Table 6 produced the following fitted model:

$$y = -0.191 - 2.13x_1 + 0.64x_2 - 49.1x_1x_2 + 8.5x_3 - 51.9x_1x_3 + 62.0x_2x_3 + 59.3x_1^2, \quad R^2 = 0.996 \tag{9}$$

The fitted model would appear to provide an excellent description of the data set: the value of R^2, 0.996, would be regarded as extremely good. However, note that the estimated parameters given in Eq. 9 bear little resemblance, even in algebraic sign, to the coefficients in the true model Eq. 8, and clearly do not properly describe the effects of the x's on the response.

The fitted equation predicts quite well the expected responses for values of x that are close to those in the data set. However, if one asks for an estimated response at $x_1 = 1$, $x_2 = 0$, and $x_3 = 0$, the fitted equation would yield $y = 57$, which could be compared to a value of 3 (the value given by the true model). Thus, if this experiment had been applied at the point (1, 0, 0), values of approximately 3 would have been observed, but the fitted equation would have produced expected values approximately 57. The coded data point (1, 0, 0) would correspond to a high value of x_1 (e.g., asphalt content) and medium values of x_2 and x_3 (e.g., the thickness of the two layers). It might seem reasonable to use the fitted equation to estimate the response at this point, but in fact there were no experimental points near this point and the multicolinearity in the data set has produced a fitted equation that is of very limited use. To avoid these limitations, an experiment must be planned and conducted that is balanced over the full range of x values. As demonstrated above, such a collection of data points is very unlikely to occur even in a very large observational database. Thus, observational databases must be used with extreme caution when generating the types of models needed to develop performance-related specifications.

Experimental Databases

For many years, the value of well-designed 2^n or 3^n factorial experiments in industrial and university research laboratories has been widely recognized. Whenever the costs of such experiments is high, a case-specific fractional factorial or central composite design may be selected (*19, 20*). Generally the choice of the design depends on the need for estimating certain coefficients in the response model, which in turn depends on the nature of the assumed effects in the region of interest. If all effects are additive (and linear), then very small (but planned) experiments can be used to estimate the effects and predict future responses for the entire range of interest. This sort of assumption will generally hold in only a very small region and is a very risky one to make. Indeed, nonlinear effects and interactions among the factors are to be expected and the experiments must be designed so that these can be evaluated.

Development of a response surface is too much to expect from an experiment in which the factors are considered at only two levels. Although in some cases the response variables of interest may vary in a monotonically increasing or decreasing manner, often the true response may be highly nonlinear as shown in Figure 25, where hypothetical modulus data are shown as a function of air voids. If a 2^n design is chosen with air voids at Levels A and E, no effect will be shown; if air voids are shown at Levels B and E, a decreasing effect is shown; and if Levels A and D are chosen, an increasing effect is shown. Clearly, many

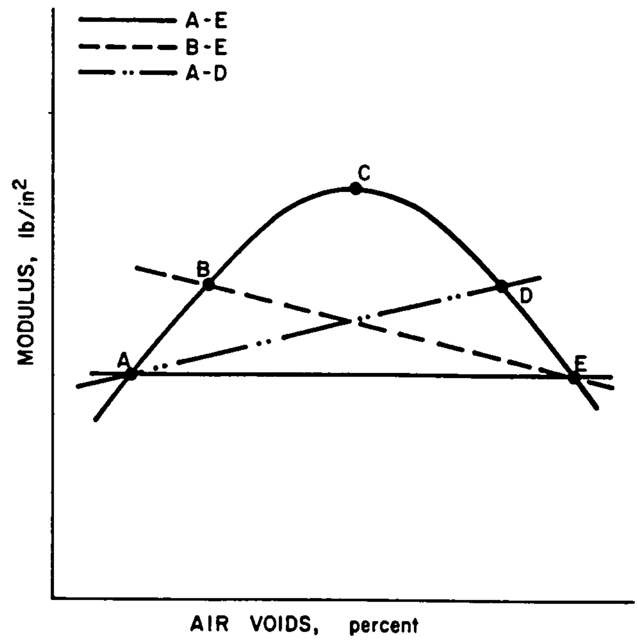


Figure 26. Hypothetical graph of modulus vs. air voids.

of the variables of interest must be studied at three or more levels, necessitating 3^n factorials. The prior experience of the researcher conducting the experiment, or the published literature, may be used as a guide for selecting the needed number of levels of each variable.

To properly study nonlinear relationships such as the one shown in Figure 26, one must have at least three values of each variable, and these must be in a well-chosen arrangement if a reliable estimated response surface is to be produced. In this case, the independent variable, air voids, must be chosen so that the range of values extends above and below the value that produces the maximum modulus. This may be accomplished by employing a fractional 3^n experiment or a central composite design.

In the case of field experiments, it may not be possible to develop the experiments in a sequential manner. In this case, the best historical data sets (observational and experimental), and the judgment of knowledgeable workers, must be used to decide which factors are to be studied, which ones must be included at three or more levels, and which interactions may be assumed to be small, if not zero. With this background, it is likely a 2^j3^k factorial experiment will be chosen.

When a field experiment has been chosen as described above, it will be fairly clear, even before conducting the experiment, just how well the objectives of the experiment will be met. In this regard, the choice of experiments should be made on the basis of their costs and their ability to achieve their objectives. When the long-range benefits from well-planned experiments are clearly understood, it is very likely that their cost will no longer be considered prohibitive.

The advantages of a well-designed experiment plan and the resulting experimental database, for which the range in the variables can be specified to include all regions of interest (both target values and ranges of nonconformance), are: (1) The importance of the various independent (e.g., M&C) variables and their interactions on the response of interest can be evaluated. (2) The prediction equation will be useful over a large region because the independent control variables are varied in a well-balanced manner. (3) The prediction equation will suggest the optimal set of M&C variables for use in the development of the acceptance plan.

The disadvantages of an experimental database are: (1) It will undoubtedly be very expensive to develop. (2) Some poorly performing sections must be constructed.

It should be understood that the ultimate purpose of the experimentation is to understand how the responses of interest depend on the controllable factors, so that wise choices can be made as to the levels of these factors. The framework for doing this most efficiently has been described in rather general terms.

General Comments on the Databases Needed to Develop Performance-Related Specifications

The database used in the development of a PRS must allow researchers to develop or verify models that relate M&C variables to pavement performance, either directly or through the various levels, as shown in Figure 2. As noted earlier, the prediction equations developed from an observation-type database should not be used to predict performance outside the region in which the database was developed. Thus, an experimental database, such as will be possible with the SHRP SPS pavement sections, is preferred in the development of PRSs. Such sections must include pavement sections that are deliberately constructed in nonconformance with the target construction (i.e., out of specification). Reliance on observational—particularly historical-type—databases will severely hamper specification development.

Broadly speaking, the adequacy of a database is measured by two criteria: quality and quantity. The attributes of a good database are (1) accurate measurements of unbiased, random samples; (2) replication of the experimental units; and (3) an adequate range and balance of the data elements.

To obtain meaningful results and to make valid inferences, analyses must be based on a statistically adequate quantity of data. Quantity refers to sample size, which, in turn, depends on the number of factors and interactions involved in the analysis. To develop a PRS, the database must contain an adequate number of observations to study the effects of all performance-related factors and at least the two-factor interactions.

The database should include all of the data elements required to establish or validate both Level A-E and Level E-G models. Because the objective is to develop a PRS, the database should include—in addition to design and M&C data—all relevant information pertaining to location or environment, traffic history, pavement condition (cracking, rutting), and maintenance activities and frequency.

One final point must be emphasized concerning the range of data in any database used to develop a PRS. The objective of such a specification is to adjust the payment to the contractor in a manner proportional to any reduced serviceability of the pavement. Therefore, the database must be sufficient to quantitatively relate nonconformance to serviceability, and some of the data elements must contain a range of data showing construction at both target and nontarget values. Furthermore, there must be a certain balance in the high and low values of the factors, so that their effects are not hopelessly confounded. This balance is often lacking in observational databases.

It can be argued that the effects of nonconformance could be estimated from normal construction variability. Nonconformance may, by chance, occur in the database; however, it is more likely that the overwhelming majority of the asphalt contents would be at or near the target value, providing an insufficient range in nonconformance to properly assess (with statistical validity) its importance in predicting performance. This requirement of intentionally and systematically selected nonconforming sections is typically missing from existing observational databases. The requirement for nonconforming construction and its performance history is unique to the databases needed for the development of a PRS, although it is an inherent part of the reliability function in the AASHTO Interim Guide. The construction of nonconforming sections will likely produce premature failures, favoring the use of full-scale testing.

REVIEW OF EXISTING DATABASES

One of the major tasks in this project was to review existing pavement performance and materials databases to identify those that are of potential value in developing the algorithms needed to relate M&C variables to pavement performance. To facilitate this review, and in keeping with the foregoing discussion of database needs, the databases that were reviewed were classified as either historical or observational. As a first step in the overall database review process, the research team reviewed the work that was done as part of the project that preceded the current research, NCHRP 10-26, "Data Bases for Performance-Related Specifications for Highway Construction."

NCHRP Project 10-26 Findings

NCHRP Project 10-26, "Data Bases for Performance-Related Specifications for Highway Construction," the predecessor to this project, was conducted with the specific purpose of identifying and evaluating existing databases that can be used to develop performance-based specifications for highway construction (*21*). Six of the databases evaluated in that study were observational.

An observational database for portland cement concrete (PCC) was established as part of an FHWA study, "Correlation of Quality Control Criteria and Performance of PCC Pavements" (*22*). These data were collected to study the relationships between quality control indicators (M&C variables) and PCC pavement performance measures. Data included environment, traffic, design, and construction information from historical records, and pavement performance records from detailed pavement condition surveys. The study sampled more than 100 PCC pavement projects in five states. M&C variables were not screened; instead, all M&C variables were collected and added to the regression equations. This approach did not lead to a robust model, and the predictive ability of the performance model was poor. The number of M&C variables observed (sample size) in each project varied between 0 and 50. This imbalance in the database design made statistical analysis difficult and limited the utility of the models developed (*21*). Because this database was developed for portland cement concrete and because of the difficulties in using historical observational databases, it would be of no value for the current research project.

One recommendation from the AASHO Road Test was that each state should establish its own satellite test roads. Databases established by the different states as part of these satellite studies were reviewed as part of NCHRP Project 10-26. The quality and quantity of the data were evaluated from answers to questionnaires sent to all 50 states. Only 19 states developed any satellite test road data (*21*). While this database is completely computerized, it was concluded that the data do not include sufficient elements for M&C variables to be related to performance.

Many states have established construction/performance databases and these were also reviewed as part of the NCHRP 10-26 project. Although these databases represent very significant and pioneering efforts on the part of many states, they are, for one or more reasons, not suited to the development of performance-related specifications. Examples of the findings from the NCHRP 10-26 project are given in the paragraphs that follow. For example, construction information in the Georgia database is stored in voluminous manual files (*21*). Apparently, because of its limited accessibility, it was not possible for the NCHRP 10-26 contractor to evaluate the quality or quantity of this database.

Minnesota has collected data for flexible pavements and stored the information in manual files. The quality of maintenance data ranged from very good to very poor (*21*). In the State of Washington, pavement performance data are stored in a computerized database. Traffic data, before and after construction, are also collected. However, limited environmental data are available. M&C data are difficult to retrieve and assemble because they are kept in manual files (*21*) which must be correlated manually to specific roadway sections.

Concerning the above databases, NCHRP Project 10-26 (*21*) found that construction quality control data in the majority of the databases were stored in manual records and were difficult to retrieve and process. Also traffic data were generally estimated from limited traffic counts, loadometer (weigh-in-motion) data were missing or inadequate to validate performance models, and maintenance records were either unavailable or could not be coordinated with other aspects of pavement performance.

In addition, the available databases were collected from normally designed pavements rather than from a range of weak-to-strong pavements. Furthermore, as a result of confounding, the paired variables could not be studied separately in the databases. These findings support the conclusion from NCHRP Project 10-26 that existing historical observational databases were deficient from an analytical viewpoint and cannot be used to develop a PRS for highway construction. In general terms, this research team agrees with the results given in the NCHRP 10-26 final report. Although the databases may be useful in limited ways, such as for validating some A-E relationships or establishing sampling and testing components of variance for M&C variables or performance indicators, they are insufficient for the overall objectives of the 10-26 program. This led the research team for this project to evaluate a number of additional databases.

Observational Databases Evaluated as Part of the Study Quality Assurance in Highway Construction

In the 1960s, the Bureau of Public Roads (BPR) launched a research program, Quality Assurance in Highway Construction, to develop and incorporate a statistical approach to quality assurance in highway construction. Several states followed guidelines developed by the BPR for determining variations in the quality of accepted construction. Research results, in the form of interpretative summaries, were published in six reports in 1969. One report summarized the results of studies for measuring variations in accepted bituminous concrete construction (*23*).

Random samples, collected independently of process control samples, were tested for several quality characteristics: aggregate gradation, asphalt content, compaction temperature, density, and pavement thickness. In addition, stability and flow parameters were measured by states that used the Marshall method of mix design.

The random sampling procedures adopted during the studies made it possible to estimate the quality of construction. In addition, the causes of variation were isolated and quantified. However, only M&C data were analyzed. Environmental, maintenance, and performance data were unavailable. Consequently, the data from these studies cannot be used to correlate pavement performance measures with construction and design data. Although these data may be of limited value in determining the variability that can be expected during the construction process, they are dated (1960s) and may not apply to current construction practice.

Highway Condition and Quality of Highway Construction (HC&QHC) Database

In 1976, FHWA began a program to monitor the construction quality of selected highway projects and to periodically evaluate the condition of pavements surveyed for construction quality. The program has three objectives (*24*): (1) to provide an assessment of the degree of compliance with specifications for construction activities; (2) to provide information on the condition of recently completed highways; (3) to identify potential problem areas in the quality of highway construction in order to formulate strategies to improve the quality and performance of pavements and bridge decks.

Three construction types—rigid pavements, flexible pavements, and bridge deck construction—are included in the program, in which only one type of construction is surveyed each year. Consequently, pavement condition surveys are conducted for any given project every third year; the surveys continue until 9 years have passed or until significant maintenance or reconstruction work has been required.

Projects slated for quality of construction surveys are selected from the lists of scheduled Interstate and Federal-aid primary projects provided by participating state highway agencies. Several maintenance and construction variables, such as bitumen content, layer thickness, and percent passing the No. 200 sieve, are monitored. A statistic called "construction quality level" is calculated for each M&C variable. This statistic measures the degree of compliance to specifications during construction. In addition, pavement condition data for various types of distress are collected according to the procedures contained in the "Highway Pavement Distress Identification Manual" (*25*). Severity levels for various distress types are established by using a qualitative scale (e.g., none, low, medium, and high severity).

A possible application of the HC&QHC data is in the development of relationships between M&C variables and pavement performance. However, because the available data are limited to projects scheduled for construction during a given year, there is no provision for establishing special test sections, such as sections that incorporate nonconformance in selected M&C variables; nor is there an experimental plan to guide the selection of projects to be surveyed. It would be difficult, therefore, to evaluate the importance of the various M&C variables on the observed pavement performance of test sections. In addition, because performance evaluations are conducted only every 3 years, and projects are dropped from the HC&QHC survey after 9 years, data for establishing performance trends are limited.

FHWA Demonstration Project No. 2

FHWA Demonstration Project No. 2, conducted from 1969 to 1974, was a demonstration of statistically based acceptance plans. FHWA personnel visited various state highway agencies to show the advantages of statistically based acceptance plans in the measurement and evaluation of product quality. The feasibility of using more rapid test methods for quality control was also demonstrated, together with the use of control charts for establishing trends in the data.

As part of the demonstration project, bitumen content, aggregate gradation, and mixing temperatures were measured on active construction projects. However, no performance evaluations were made on the construction projects for which data on quality control were collected, so the data collected are not useful in developing performance-related specifications.

FHWA Research Database

FHWA has initiated a study for establishing a centralized, computer-oriented system for locating pavement information (Task 197). The task involves pooling available databases from FHWA and state highway agencies, and referencing the geographical locations of highway projects included in available databases. The system is expected to facilitate the location and retrieval of data collected on pavements subjected to a particular set of conditions, for example, wet, hard, and freeze-thaw environments. Specific types of information that would be stored in the system are limited to information available in existing databases, inasmuch as the thrust of the project is the use of available databases and not the collection of pavement data. Data on materials, construction, and pavement performance should be in the system, but because the project is at an early stage, only limited data are currently available.

EXPERIMENTAL DATABASES EVALUATED AS PART OF THIS STUDY

The research team identified several additional databases that contain information from experimental pavement projects. The contents of these databases and their usefulness to this study are discussed below.

Louisiana DOTD Database. Louisiana has an ongoing research project for evaluating the influence of construction on performance. Since June 1978, all construction quality control test data have been stored in a computerized system. In addition, traffic, maintenance, and performance data have been collected and stored. At the time of completion of NCHRP Project 10-26, the pavement sections under study showed no significant distress (*21*); therefore, this database was not examined. However, discussions with Louisiana Department of Transportation and Development (DOTD) personnel led the research team to evaluate the potential value of this database.

The Louisiana DOTD conducted a research project to verify the adequacy of AASHTO flexible design predictions through direct, controlled comparisons of the behavior and performance of 18 different experimental pavement sections (*26*). The structural sections of the Louisiana Experimental Base Project were constructed with different base types and surface thicknesses, representing various design lives. A comprehensive program for characterizing the stabilized materials used in construction was conducted with test specimens prepared in the laboratory and cores taken from the test sections. The response characteristics evaluated included the modulus of elasticity, tensile strength, and fatigue behavior of the stabilized materials. Test section performance was also monitored periodically.

The data generated as part of this program for materials characterization are potentially useful for developing relationships between basic asphalt mixture properties, such as bitumen content and air voids, and material response characteristics. In addition, the data could be used to verify existing prediction models for fundamental material response characteristics, which are necessary inputs in many rational performance models.

Texas Transportation Institute Study. The Texas Transportation Institute has conducted a study (Research Project 287) to evaluate, among other things, the relationship between laboratory asphalt properties and field pavement performance (*27*). In the study, three experimental test sections were constructed using two asphalt grades from five different sources. A laboratory testing program was established through which properties of the asphalt cement and the asphaltic concrete mix were characterized at periodic intervals during the course of the study.

An evaluation of the findings from the project was conducted to determine if the results can be used in NCHRP Project 10-26A for relating M&C variables to field pavement performance. It was concluded that the data available on the three experimental test sections are insufficient for establishing valid relationships between M&C variables and field pavement performance. Only one mixture design was specified for the construction of the test sections, and only asphalt grade and asphalt source were systematically varied in the experiment. In addition, the available data are too limited to be useful in the evaluation of performance trends.

AASHO Road Test Flexible Pavement Performance Database

The AASHO Road Test, conducted from 1958 to 1960, had, as one of its principal objectives, the determination of significant relationships between pavement performance and certain characteristics of pavement design and applied loadings (*28*). During their evaluation of the AASHO Road Test database, NCHRP Project 10-26 investigators found that much of the data was either missing or inaccessible (*21*). However, flexible pavement performance data collected for the AASHO Road Test have been organized into a database by the Pennsylvania Transportation Institute (PTI). Other research agencies have assembled similar databases consisting of selected data from the AASHO Road Test. The contents and utility of the PTI data bank are discussed in this section.

The AASHO Road Test is the largest full-scale highway research experiment ever to have been conducted in the United

States. The AASHO study continues to be a valuable source of data for the development and verification of pavement performance prediction relationships. For each flexible pavement test section included in the main factorial experiment of the AASHO Road Test, data on roughness, cracking, patching, and rutting were collected at 2-week intervals. These data, together with information on the cumulative number of axle load applications, have been organized into computer data files. For the purposes of NCHRP Project 10-26A, these data files are potentially useful for pavement performance modeling and for verification of existing performance prediction equations. However, it must be cautioned that the construction methods and traffic loads may not be representative of current conditions; this may limit the usefulness of the AASHO Road Test data for the development of performance-related specifications.

One reason for the continued usefulness of the AASHO Road Test data is that the data were collected as part of a carefully planned experimental design. In any type of experiment, the importance of a well-planned statistical design, tailored to the objectives of the particular study, cannot be overemphasized. A primary objective of the AASHO Road Test was the determination of the relationship between pavement performance and certain design variables. The principal factors considered were axle load, axle configuration, and the thicknesses of the various layer components (i.e., surface, base, and subbase). These factors were included in the main factorial experiment of the AASHO Road Test.

The test sections for the Road Test were built in six main loops, five of which were subjected to axle load applications. Within each loop, the sections constituted a complete factorial experiment on two separate lanes. Each lane of a trafficked loop was subjected to only one axle load and axle configuration. In this way, the effects of various axle loads and axle configurations on pavement performance could be evaluated. Within each loop, the test sections were placed at random to reduce the risk of confounding the effects of the various layer thicknesses with the effects of uncontrolled variables that vary systematically. Certain sections were replicated within each loop so that an estimate of error, for evaluating the significance of the effects of the various controlled factors, could be calculated.

In addition to being a valuable source of pavement performance data, the AASHO Road Test illustrates the application of certain principles of experimental design; specifically, randomization, replication, and the use of factorial experiments to efficiently evaluate the effects of various independent variables on the dependent variable(s) of interest. Consequently, the experimental design for the AASHO Road Test can also serve as a model for the establishment of similar designs for other pavement-related research experiments such as those envisioned for SHRP LTPP and other laboratory and field experiments that will be proposed in support of the development of PRSs.

EVALUATION OF KNOWLEDGE-BASED (EXPERT) SYSTEMS FOR USE IN DEVELOPMENT OF PERFORMANCE-RELATED SPECIFICATIONS

Knowledge-based (expert) systems are interactive computer programs which employ a collection of judgment, experience, rules of thumb, intuition, and other experience in a particular field, coupled with inferential methods of applying this knowledge to provide expert advice on the performance of a variety of tasks (*29*). The primary purpose of knowledge-based systems, which have evolved from artificial intelligence research, is to address ill-structured problems for which a numerical algorithmic solution is unavailable or impractical (*30*). The first attempts to commercialize this technology by several large corporations in the early 1970s failed (*31*). Costly development requirements, the complex nature of the programs, and the computer hardware available at the time contributed to these failures. Advances in the theory of artificial intelligence and in computer hardware, especially the development of microcomputer technology, have made the application of knowledge-based systems technology more feasible today. Successful knowledge-based systems have been developed for the medical profession to assist in the diagnosis of bacterial infections, to assist geologists in the search for profitable ore deposits, and to assist in the configuration of computer systems (*29, 32, 33*).

Table 7. Primary differences between expert system programming and conventional programming.

Expert System Programming	Conventional Programming
Representation and use of knowledge	Representation and use of data
Knowledge and control separated	Data and control integrated
Heuristic (inferential) process	Algorithmic (repetitive) process
Effective manipulation of large knowledge bases	Effective manipulation of large databases
Midrun explanation desirable and achievable	Midrun explanation impossible
Oriented toward symbolic processing	Oriented toward numerical processing

The primary differences between knowledge-based systems programming and conventional computer programming are summarized in Table 7 (*33, 34, 35*). Simply stated, knowledge-based systems process knowledge whereas conventional computer programming processes data. Hayes-Roth, Waterman, and Lenat note that applications are suitable for knowledge-based systems when the knowledge is subjective, poorly coded, and partly judgmental (*36*). They continue that, when knowledge is firm, fixed, and formalized, algorithmic computer programs are more appropriate.

The use of knowledge-based systems for predicting pavement performance is one application of knowledge-based systems, because a great deal of the current knowledge available to predict pavement performance may be described as subjective and judgmental (*29, 30*). With few current databases that contain good experimental data, the application of knowledge-based systems is a reasonable alternative approach for developing models to predict future pavement performance.

Knowledge-based systems can be relatively simple decision trees, where, given certain conditions (e.g., traffic, climate, and road condition), the system is asked whether or not the conditions justify a rehabilitation before next year. Or, these systems can be more complicated and incorporate Markov decision processes, where instead of responding to whether or not the road needs rehabilitation, the questions are answered in terms of probability estimates (e.g., a 50 percent chance that an overlay is needed, a 25 percent chance that routine maintenance is needed, and a 25 percent chance that no maintenance is needed).

As discussed earlier in this report, only a limited number of models relate M&C variables to pavement performance, and there are some types of pavement condition indicators for which no suitable models exist (e.g., raveling). Further, there are no models that account for the effects of nonconformance on expected pavement life.

It is anticipated that, over time, particularly as a result of SHRP's LTPP effort, models will be developed that will improve the understanding of the relationships between design, environmental, and M&C variables, and pavement life. However, in the interim, knowledge-based systems can be an effective method of developing performance models for designing pavements and establishing optimal maintenance strategies. The implementation of knowledge-based systems for development of pavement performance models would likely be most beneficial in the following three areas:

1. Development of performance relationships that are currently not modeled satisfactorily by other means. This could be a model to predict raveling, as mentioned above, or one to predict reflection cracking if it is felt that existing theoretical models are not adequate.
2. Development of models that consider the interdependent relationships between pavement distress mechanisms. For example, cracking can allow water into the pavement and accelerate the effects of rutting. Or, if ruts develop, they could cause stress concentrations at certain locations that could result in cracking.
3. Development of models that consider the effect of maintenance on future pavement performance. Very little is known about this effect; however, it may be one of the more appropriate applications of knowledge-based systems. In this example, the knowledge of individuals with expertise in what type of maintenance should be applied, under what conditions, and at what time, could be reasonably incorporated into a knowledge-based program.

The three areas discussed illustrate some of the more probable applications of knowledge-based systems in the design and maintenance areas. Considerable progress has been made in these applications and French researchers have announced that they will soon implement a system for pavement management purposes (*37*).

Application of knowledge-based systems to PRSs serves a purpose that is different from the design or maintenance management application. The development of PRSs must quantify the effect of nonconforming M&C variables on the performance of the pavement, whereas the pavement designer or manager is interested in the expected life of the pavement when different designs or strategies are employed. The developer of a performance-related payment schedule is interested in the loss in pavement life, and the associated costs, when the pavement is constructed at other than the target values. In other words, the designer or maintenance engineer is interested in the expected life of a properly constructed pavement, whereas the developer of a performance-related payment schedule is interested in the loss in pavement life that results when the pavement is improperly constructed. More specifically, a quantitative relationship between degree of nonconformance and decreased life (or increased cost) is needed.

On the basis of the above discussion, the most promising application of knowledge-based systems in the specifications area may be in the development of a payment schedule. Properly designed, the knowledge-based system can account for interactions between variables, as for example between the effect of low asphalt content and poor compaction on rutting and fatigue, or raveling. Consequently, the authors recommend that in-depth studies be conducted to develop within the framework for performance-related specifications a knowledge-based process to address the more complicated aspects of how M&C variables affect pavement performance and the resulting payment schedule.

CHAPTER THREE

INTERPRETATION, APPRAISAL, APPLICATION

The conceptual framework for a performance-related, statistically based specification has been developed and demonstrated with hypothetical data and a limited number of variables. Based on the evaluation of the results, the framework appears to be valid and the scheme for payment appears to be sound and workable.

CONCEPTUAL FRAMEWORK

The conceptual framework developed in this study is based on the assumption that there are selected M&C variables that can be related to pavement performance, and that these variables can be controlled by the designer or contractor. These two assumptions are very important; to reiterate, the materials and construction variables that are included in a performance-related specification must be: related to performance, controllable by the contractor, and independent variables that do not lead to a double jeopardy condition.

If the selected M&C variables cannot be controlled by the contractor, or if the variables are dependent on each other such that a condition of double jeopardy is created, the specifications will not be legally defensible and will be declared invalid.

The primary assumption upon which the framework is built is that, by proper selection, the M&C variables can be used to predict reasonable life-cycle costs for the target and the as-constructed pavement. By comparing these life-cycle costs, which include both user and maintenance costs, a payment adjustment can be established that is equivalent to the increase in life-cycle cost that results from contractor nonconformance to the materials and construction specifications. Two assumptions are then required: (1) properly chosen, the M&C variables can be used for a given pavement with a given climate, set of site conditions, and traffic levels to predict a reasonable and defendable life-cycle cost; and (2) nonconformance in the M&C variables will result in increased life-cycle costs, and these increases can be predicted with properly developed prediction models.

Intermediate between the M&C variables and the calculated life-cycle cost are a number of steps that require the identification and prediction of certain response variables. These variables must be predicted for both the as-constructed and target pavement. A number of levels have been defined as follows (see Figure 2):

- Level A, materials and construction variables (M&C).
- Level E, fundamental mixture response variables (FMRV).
- Level F, fundamental pavement response variables (FPRV).
- Level G, theoretically predicted pavement condition indicators.

Finally, a procedure must be adopted for calculating life-cycle costs. In the past, pavements and materials technologies were not sufficiently advanced to permit the development of reliable predictive models. With the completion of several NCHRP and FHWA projects, and especially the SHRP program, the required prediction models and test methods should become available.

PROCEDURAL CHOICES

The overall procedure described above—measuring the M&C variables for the target and as-constructed pavements, predicting their respective performances, and then comparing their life-cycle costs—may appear deceptively simple to the casual reader.

A number of important questions must be answered if workable performance-related specifications are to be developed. These questions include the following, Which materials and construction variables shall be measured, and how shall they be measured? Who shall conduct the mixture design, the contractor or the owner agency? How shall the fundamental mixture response variables be used for acceptance and payment purposes? Should the individual mixture components, asphalt cement, aggregates, and additives be included in the specification; or, should only the properties of the hot-mix itself be specified with a performance-related specification?

Prediction of Mixture and Pavement Response Variables

The answers to the foregoing questions can be found, in part, by asking the question: What distress modes must be included in the specification? This question should be the first one answered when a PRS is being developed. This is an important philosophical point: the distress modes that are important to the owner agency should drive the specification. Without first identifying the distress modes that are of concern, the performance indicators and, in turn, the fundamental pavement response variables, the fundamental mixture design variables, and, at the end of the chain, the M&C variables that must be indicated in the specification cannot be selected. Indeed, it is important that the development of the specification be driven by the distress modes and not by the available M&C variables. The purpose of a PRS is to control the acceptance of the pavement materials and construction in a manner such that a certain level of performance, as compared to the target pavement, can be ensured.

The conceptual framework developed in this study is modular in concept. Not all distress modes need to be addressed—rather, only those that are of particular interest to the user agency. For example, moisture damage may not be a problem in an arid portion of Arizona and, therefore, it need not be included in the specification. This is similar to the approach taken in the AAMAS (*3*) NCHRP 9-6 project where, for a given agency, only those test modules of interest need to be considered.

Having chosen the distress modes of concern to the user agency, it then becomes a matter of selecting the appropriate performance prediction models and, subsequently and as a consequence of the chosen mode, the fundamental pavement or mixture response variables. Once the required fundamental mixture response variables have been selected, several additional questions must be addressed as described below.

Measurement of Fundamental Mixture Response Variables

The first of these questions deals with the determination of the fundamental mixture response variables for the target mix: How should they be measured, and at what point should they be measured? Three different scenarios are possible. As discussed in Chapter Two, the approach that should be taken depends on the level of service provided by the highway (i.e., Interstate versus farm-to-market). Presumably the pavement with the higher level of service will require a more reliable design and more accurate estimates of the response variables. Further, the cost of the construction on a low-volume rural roadway may not permit the expense of extended testing of the more fundamental mixture response variables. The three scenarios, in order of increasing cost and reliability, are:

1. Use models that predict the FMRV from M&C variables. Examples are the modulus prediction model by Witczak (*4*) and the fatigue model by Cooper (*38*).
2. Measure the needed FMRV on the optimum or target mixture where the mixture recipe is determined from a standard Marshall or other design method.
3. Develop new mixture design procedures or modify present methods, so that the fundamental mixture design variables are part of the mix design process as will be the case with the AAMAS system under development as part of the SHRP program.

Although these scenarios are alternatives for determining the properties of laboratory mixes, they do not cover the measurement of the as-constructed field properties. Considering the first scenario, two approaches arise for measuring as-constructed material. Models could be developed or current models refined, so that mixes with nontarget recipes (e.g., low binder content, excess fines) are also included in the prediction models. The second approach is to create a prediction model that relates the degree of recipe nonconformance to the difference between the values for the target and nontarget FMRV. Either of the two approaches is valid, and in both approaches cores would be required to measure the values for the M&C variables.

The second and third scenarios warrant field coring or, at the minimum, the testing of plant mix compacted in the laboratory. The same schedule of testing for FMRV would be required for the field specimens as for the mix design specimens and would be compatible with the input required for the performance models. In all cases, construction variables, such as roughness, percent air voids (density or relative specific gravity), and thickness require direct field measurements.

Selection of Target Values

A second question which must be addressed concerns determination of the target values for the M&C variables and the FMRV that are required for the target pavement design. This step must be completed in advance of the bidding of the contract, typically before the sources and properties of the materials and mixtures are known. Therefore, the pavement designer must assume target values for the M&C variables and the FMRV. With fully implemented specifications, the specified material properties may include properties of the asphalt cement, aggregates, and mixtures. As a minimum, however, the target properties of the mixtures must be specified. The selected target properties would most likely be representative of pavements that have historically provided acceptable performance. These values would then be used to design the pavement and to set the levels of the target M&C and fundamental pavement response variables.

Having selected initial values for the target variables, the next question concerns the status of these variables throughout the bidding, construction, and acceptance process. The various possibilities are described below. First, the target materials, construction, and FMRVs may remain constant throughout the design, construction, and acceptance process. In this scenario, the contractor is evaluated against the same target values in the bidding, construction, and acceptance phases of the project. This situation creates a strong incentive for the contractor to improve the quality of the materials and construction beyond that determined by the initial target values. However, the incentive is to bring the quality only to the level of the target values—there is no incentive to improve quality beyond the target values.

Second, the levels of the target variables may change from the initial design to the bidding and construction phases of the contract. In this scenario, each contractor would submit individual target values for the mixture design and fundamental mixture response variables, and each bid will be evaluated on the basis of these values. The contract would be awarded on a best buy basis and the payment would be based on the as-constructed pavement properties versus the target values contained in the bidding document. To the extent that better quality material leads to a longer pavement life, and, hence, reduced life-cycle costs, the contractor should develop the bid in favor of the higher quality materials. This is a desirable goal of a PRS—to provide an incentive to the contractor to improve the quality of the materials and construction.

Although the tendency in the varying target value approach (second scenario) would be to use fundamental mixture response variables for the mixture design and the selection of specification criteria, it is expected that some recipe variables would be retained, such as the percent mineral filler or the percent rock in the mineral aggregate. However, caution must be used to ensure that the mixture is not overspecified to the extent that situations of double jeopardy are avoided. In summary, the varying target values approach is the preferred approach because it provides an incentive to the contractor to improve the quality of the construction.

EVALUATION OF PAYMENT SCHEDULE

The framework developed as part of this project has been evaluated in terms of the original objective of the project. This objective was to develop a rational and fair methodology by which contractor payment may be related to the expected performance of hot-mix asphaltic concrete pavements.

The conceptual framework uses current pavement engineering technology to predict the effects of nonconformance in the M&C variables on future pavement performance. The procedures consider individual distress modes (e.g., cracking, roughness) without the use of general or combined distress condition indicators (e.g., Pavement Condition Index). The framework is modular, so that improvements can be made to the pavement performance algorithms as advances are made in engineering technology.

The payment schedule is related directly to the estimated agency and user costs resulting from the predicted pavement performance. No transfer functions are used. The contractor is penalized, or paid a bonus, depending on the degree with which the estimated as-constructed pavement-related costs are different from the estimated target pavement-related costs. Agency and user costs are estimated considering the real interest (discount) rate, life-cycle cost categories of initial construction, maintenance, and rehabilitation, and the effect of future performance periods.

The framework approach, which is to pay a contractor according to the estimated value of the as-constructed pavement, is fair. The contractor is aware, when preparing his bid, of the penalties or bonuses that will be paid, depending on the outcome of the quality control measurements. In fact, the contractor can make use of the PERSPEC program (Appendix E) to develop alternate bidding strategies where material quality, levels of nonconformance, and materials and construction variability are variables in the bidding strategy. The consequences of nonconformance in terms of expected payment are known to the contractor at the time the contract is bid.

The payment schedule is based on a rational determination of estimated pavement performance and principles of engineering economics. The bid procedure, where the lowest qualified bid is selected, is also fair and consistent with the payment schedule.

The overall concept developed during this project of relating the value of the expected performance to the contractor's payment schedule is sound and robust. Again, as a result of the modular nature of the framework, changes in technology, or even conversion to other applications (such as rigid pavements and airport pavements) can be accomplished using the same methodology.

Evaluation of Specification Application

In the previous section, the discussion was related to the evaluation of the payment schedule methodology. In this section, the application and use of the performance-related specification will be reviewed.

The application of a PRS begins with the initial pavement design and preliminary mix design. The State Highway Agency (SHA) engineer can select a pavement design that meets the agency's objectives, including the consideration of reliability in the 1986 AASHTO Pavement Design Guide or other procedures that may result from future NCHRP or SHRP studies. In this design process, assumptions about material properties and the job-mix formula are made based on historical experience within the geographical region. The SHA will then publish the assumed pavement design and materials information at the time that bids are requested. This design then becomes the "target" pavement.

During the procedure for selecting design and material properties, the SHA will use the performance specification computer program (PERSPEC) to evaluate the economic lives of the various trial designs. The agency will also determine the optimum level of service for future rehabilitations, based on the same economic methods used in the first period. This optimum level of service is used in the performance specification program to determine the capitalized cost of future performance periods as described in Chapter Two.

As an alternative, the designer may choose a method other than PERSPEC and the associated life-cycle costs to select the target pavement design. However, once a target design is selected, the economic life of the target pavement, L_T, would be determined for the target pavement. The economic life of the target pavement is used to determine the payment factors.

The application of the PERSPEC program by the contractor involves primarily the consideration of M&C variables and their effect on the contractor's bid price. The contractor will be able to determine the payment schedule that results from different construction scenarios. In this way, the contractor can evaluate the effect of different job-mix formulas, process control plans, production and construction processes, and equipment on the bid price. With superior equipment and quality control, the contractor can bid a lower price, or expect a greater profit, knowing that the bonus resulting from superior expected performance will make up for the discounted bid price. In the same respect, a contractor with faulty equipment and quality control will have to bid higher, knowing the penalty that will be expected resulting from the poorer (as-constructed) pavement.

Potential Refinements

As with any system, certain refinements and improvements are always possible. Certain refinements can be anticipated as the framework is developed in the future. Some of these anticipated refinements are discussed below.

The current framework considers payment based on the construction lot size (e.g., one lot per mile). The contractor is paid for each lot, depending on the sampling of M&C variables and the payment schedule resulting from the performance specification. However, this process does not consider the effect of how the different lots will be combined into future rehabilitation projects. For example, consider a construction project of 10 miles that is divided into 10 lots (1 per mile) for quality control and payment purposes. If the contractor performs poorly in the first five lots, and well in the second five lots, there may likely be two future rehabilitations that occur at different times, with the first 5 miles being rehabilitated earlier. However, if the contractor performance alternates between poor and good construction for the 10 lots, rehabilitation will be required at some time in the future where some sections may be performing better than the poor ones, but will be combined with the poor sections for rehabilitation for practical reasons of project size.

Considering the project size for the two scenarios described earlier, the contractor should be paid more for the first case, where the bad lots and good lots are grouped together, because they can be rehabilitated in different projects. In the second scenario the contractor would be paid less, because certain sections would be rehabilitated before necessary. This concept could be made part of the performance specification, most likely through the use of a statistical cluster analysis of the constructed lots to determine how the road sections would be combined into future rehabilitation projects.

The pavement performance models currently included in the framework are implemented assuming that the individual distress modes (i.e., cracking, rutting, roughness) are independent. At some time in the future, when pavement technology advances make it appropriate, the pavement performance models should include the effects of the interactions of the different distress modes. For example, a pavement that is thermally cracked is more likely to allow water intrusion that may lead to stripping and/or rutting. Thus, the presence of one distress mode precipitates the occurrence of another.

The current framework does not consider the uncertainty in the different distress prediction models, the prediction of traffic, or other empirical relationships that are used in the procedure. Although this consideration is not necessary for the initial development of a PRS, the evaluation of the uncertainties in the models would allow the process to be used as a roadway management tool. Consideration of prediction error could also lead to benefit/cost evaluations of the benefit associated with improving various modules in the framework. For example, improving traffic prediction accuracy would require an expenditure for traffic studies and equipment, but the cost might be offset by the benefit of a more reliable pavement design.

LABORATORY STUDY

In Chapter Two, the need and framework for developing relationships between the fundamental response variables and the M&C variables was presented and demonstrated by a limited laboratory investigation. The statistical analysis of the laboratory experiment is presented in Appendix G. An existing model that relates the complex modulus to M&C variables was applied to a laboratory database generated as part of this study. The database included both conforming and nonconforming mixtures. The results indicated that models generated for mixes that are typically in conformance with optimum or target designs may not offer reliable predictions for nonconforming mixtures. Thus, just as it will be necessary to construct and monitor "out of specification" sections in the field, it will be necessary to develop M&C and FMRV relationships for "out of specification" mixes.

Careful consideration must be given to the construction of laboratory databases to ensure an adequate range of values for the variables, to insure that replicate measurements are obtained, and to ensure that a range of materials are included in the testing program. Caution must be used to be certain that three or more levels are used for the variables unless it is known a priori that the effects are truly linear in nature. Thus, 3^n or 4^n factorial designs may be necessary.

As studies currently underway proceed, especially the development of an AAMAS system, the materials variables will increasingly encompass fundamental response variables. However, relationships between M&C variables (the construction variables, e.g., as-constructed air voids and pavement roughness and recipe variables, e.g., asphalt content) and fundamental mixture response variables will continue to be a necessary part of performance-related specifications.

The use of statistically designed experiments and regression analysis to develop relationships between M&C variables and FMRV was demonstrated. The models relating the complex modulus and the tensile strength to M&C variables contained no quadratic effects as a limitation of the experiment design, a 2^n fractional factorial. A more appropriate experiment would have included the M&C variables at three or more levels. The laboratory study also demonstrated the need for careful experiment design and clearly showed that considerable testing resources will be needed to generate the databases required to generate the needed models.

APPLICATION OF SENSITIVITY ANALYSIS TO SPECIFICATION DEVELOPMENT

Sensitivity analyses are important to the development of any PRS. Primarily, they are used to evaluate the behavior of various modules within the framework. This information is important to substantiate that the models yield reasonable results, and to determine which variables and conditions have the largest effects on the dependent variable in the model. The relative importance of the different variables is important to the design process and is important to the contractor in determining which factors have the largest effect on the bid price.

When a PRS is implemented within a certain geographic region, sensitivity analyses should be performed on the different framework modules using typical ranges of variables for that region. This will provide information on the important factors for a specific region, and the conditions under which they are sensitive. These results should be evaluated with respect to local experience, and if the results do not agree, it may be necessary to recalibrate the existing models or use alternative models that were specifically developed for the region. The modular specification framework developed for this project will allow for substitution of regional performance models.

FIELD EVALUATION

The requirements for experiments designed to provide databases for developing or verifying relationships between M&C variables and their nonconformance and pavement performance were defined. The special requirements that must be met when these databases are to be used to develop performance-related databases were also defined. Several different strategies can be adopted in the development of the experiment plans, but before these alternative strategies are developed, several considerations are in order. First, performance or pavement performance indicators may not always be linear functions of the M&C variables. For example, the effect of nonconformance with regard to asphalt content may lead to excessive rutting with either too much or too little asphalt in the mixture. If asphalt content versus rutting is studied in a 2^n factorial experiment, then, depending on the levels of asphalt content, the experiment may result in various apparent effects attributable to the asphalt content or there may be no significant effect of asphalt content on rutting. Therefore, unless the effects are known a priori to be linear, the M&C variables must be studied in a 3^n experiment and the levels for the factors must be carefully chosen.

A second consideration requires that the experiment be replicated so that true estimates of the experimental error variance can be obtained, thereby allowing the testing of hypotheses and other statistical analyses. The AASHO Road Test is a good example of the use of replicate sections. Those who have been associated with full-scale pavement section analysis fully understand the difficulties related to variations of the in situ materials, environment, and construction practices. The second requirement for replication involves the need to repeat the experiment within several regions. Construction done in one region is not necessarily representative of the in situ materials and construction and environmental conditions prevalent within other regions.

Experimental plans for generating databases to be used in the development of performance-based specifications should be replicated for each of the four regions established as part of the SHRP initiative: dry/no freeze-thaw, wet/no freeze-thaw, dry/freeze-thaw, and wet/freeze-thaw. The models would thus assume a regional character, much as is described by the current AASHTO interim guides. This allows the models to contain different factors that are important in some but not all of the regions. For example, moisture damage may not be a significant form of distress in a region that is dry and free of freezing and thawing.

Several alternative approaches can be taken in the development of an experimental design. First, the experimental design could be developed simply by listing all M&C variables, developing a 2^n partial factorial, and then relating the performance variables to the M&C variables. For example, one may wish to relate the number of 18-kip ESALs to a predetermined level of serviceability as measured by the M&C variables. This approach requires a very large experimental design with a large number of cells and neglects several important factors. First, the trend in current pavement technology is toward mechanistic or empirical-mechanistic design procedures. The input needed for these models comes from the level-E or basic pavement/material response variables rather than from the M&C variables. Second, the mechanistic models that are currently being developed, or will be developed, as part of the overall SHRP program relate fundamental mixture/pavement characteristics to a specific distress mode. For example, the VESYS program uses creep compliance and fatigue parameters to predict the amount of rutting or roughness that can be expected in a pavement after a specified period of service. This roughness value can then be used to calculate the number of 18-kip ESALs required to produce a certain PSI level.

Because current and future mechanistic models for predicting pavement behavior contain basic material/pavement properties as input and because mechanistic models are distress mode-dependent, it is necessary to consider the distress modes that are being controlled during the experimental design. The use of the A-E and E-G relationships can be used to simplify the experiment design. The selection of the experiment design should be driven by the distress mode being considered, the basic fundamental pavement/material response characteristics that control distress mode, and the M&C variables that best predict the fundamental pavement/material response parameters. On this basis, the development of the experiment design should proceed with, first, the choice of the distress modes that are being considered; then the choice of the fundamental pavement/material properties that best control the distress mode; and finally, the judicious choice of M&C variables that are expected to relate best to the fundamental material/pavement properties.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

On the basis of the findings reported in the previous chapter, it was concluded that the development of performance-related specifications for hot-mix asphaltic concrete is a realistic and implementable goal. Before such a specification can be used as a replacement for current end-result specifications, additional or refined pavement performance prediction models must be developed. Two types of models are needed: those that relate M&C variables to fundamental mixture response (Level A) variables and models that relate either fundamental mixture response variables or fundamental pavement response variables to field performance.

The first type of model can be generated from well-planned laboratory studies based on statistically sound experimental plans. These laboratory studies should include mixtures that are at or close to the target design. In addition, the values of the independent variables (M&C) must be varied to produce a robust database that includes a range of nonconforming mixtures. The currently available prediction models are not valid for specification purposes because the values of the independent variables are centered about the mean.

The second type of needed prediction model relates fundamental mixture or pavement response variables to field performance. The databases required to develop these prediction models do not exist as yet. Available historical databases are often incomplete: one or more factors such as traffic, environmental, material, or construction data are often missing. Most importantly, the existing databases do not adequately represent nonconforming materials and construction, instead favoring projects completed near or at optimal materials and construction. Thus, the researchers have concluded that it is essential to build such a database and that the effort should be coordinated with the Strategic Highway Research Program. In particular, the special pavements sections in the Long Term Pavement Performance Program offer the best opportunity for establishing a national database. It is further recommended that individual states develop their own regional databases that are representative of local or regional conditions.

Areas requiring additional research include:

- New or improved models that relate materials and construction variables to the fundamental response variables. Work sponsored by FHWA and SHRP is currently underway in this area.
- New or improved test procedures for measuring the fundamental response variables are needed. Such tests must be suitable for design as well as acceptance and quality control purposes. NCHRP Projects 9-6 and 1-28 and SHRP Project A-003A will address the need for improved testing procedures.
- New or improved models that relate fundamental mixture response variables to pavement performance are needed. This requirement will also be addressed by the SHRP A-005 contract, Performance Models and Validation of Test Results; however, the models must be modified and calibrated to include the effects of nonconformance so that they may be used in the development of payment schedules.
- Databases are needed to verify the relations between the fundamental mixture response variables and pavement performance. This should be one of the primary objectives of the LTPP program within SHRP. These databases must be developed from statistically sound, planned experiments; observational databases are not acceptable for this purpose. It is also essential that both conforming and nonconforming sections be built. Only in this way can the models be calibrated for the effect of nonconformance materials and construction variables.
- The current framework was developed for hot-mix asphalt and does not include provisions for the acceptance of asphalt cement or other components of the hot-mix asphalt. It is well known that the properties of asphalt cement significantly affect the performance of hot-mix asphalt. Consequently, these properties should be part of the overall specification requirements for hot-mix asphalt.
- Combining acceptance and payment specifications for the asphalt cement with the hot-mix specification may pose significant legal problems with respect to liability. For example, simultaneously imposing performance-related criteria for the asphalt cement and the hot-mix may result in contradictory or redundant criteria. If this occurs, a double jeopardy situation may be created. As a further example, if the properties of the asphalt cement change, even perhaps within the specification range, thereby causing the properties of the mixture to be in nonconformance, which party is responsible? This scenario is even more complicated when the asphalt supplier, hot-mix producer, and paving contractor are all involved in the dispute. In addition, some obvious interactions exist between the fundamental performance-related asphalt properties and the fundamental mixture properties and these must be accounted for in any combined specification. Therefore, the applicability of the conceptual framework to a specification that includes asphalt properties should be investigated. If applicable, the framework should then be extended to include asphalt properties and other mixture components. This step is necessary if the findings from the SHRP Research Project A-002, Binder Characterization and Evaluation, are to be implemented in the form of improved performance-related specifications.
- The research team believes that the framework has been

developed to the point that it can be demonstrated in a parallel pilot study in the field. In such a study, a prototype specification would be written and applied to several construction projects. In this demonstration, the requisite number and types of acceptance tests would be taken, and the acceptance and payment criteria would be determined. The quality control and payment provided by the prototype PRS would then be compared to the standard specification to assess the reasonableness of the prototype PRS. In this parallel pilot study, the current specification would be used in the actual acceptance and payment for the job.

- The pilot study will determine whether any unanticipated flaws exist in the current framework that could inhibit its full implementation. The study will correct these flaws, extend the current prototype specification to an implementable form, and provide assurance to others that the conceptual framework is sound and warrants further development. These steps are needed if the current framework is to be presented to the SHRP program as a workable tool that can be used as a basis for developing performance-related specifications for asphalt cement and hot-mix asphalt.

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APPENDIX A

GLOSSARY OF TERMS

ANALYSIS PERIOD—The period of time for which an economic analysis of various pavement design, maintenance, and rehabilitation strategies is to be made. An analysis period may contain several maintenance and rehabilitation activities.

CONTROLLED FACTORS—The design factors over which the pavement engineer has control (e.g., asphalt content, compaction density, and asphalt layer thickness).

DESIGN FACTORS—The set of environmental, traffic, materials, and structural variables that must be considered in the development of pavement strategies.

DESIGN PERIOD—The length of time that an initially constructed or rehabilitated pavement section is designed to last before the pavement condition, as quantified by pavement condition indicators, reaches the terminal level.

FACTORIAL EXPERIMENT—A statistically designed experiment established for evaluating the effects of various independent variables, and their interactions, on a dependent variable of interest.

FUNCTIONAL PERFORMANCE—The ability of a particular roadway to fulfill its function of serving traffic, which is usually defined in terms of pavement condition indicators such as roughness or skid resistance.

LIFE-CYCLE COSTS—The costs associated with the initial construction (or reconstruction), routine maintenance, rehabilitation, user operation, user delay, and salvage value. Future costs are discounted using a selected interest (discount) rate so that comparisons can be made on the basis of value at a particular point in time. Costs are considered over some designated analysis period, which can vary in length depending on the type of analysis.

MATERIALS AND CONSTRUCTION (M&C) FACTORS—Those characteristics of materials and/or construction that can be directly or indirectly controlled.

PAVEMENT CONDITION INDICATORS—The measures of the condition of an existing pavement section at a particular point in time. When considered collectively, such indicators provide an estimate of the current overall adequacy of a particular roadway and identify deficiencies that can lead to accelerated deterioration in pavement condition.

PAVEMENT MAINTENANCE—The preservation of an entire roadway, including surface, shoulders, roadsides, structures, and any traffic control devices that are necessary for its safe and efficient utilization.

PAVEMENT PERFORMANCE—The history of pavement condition indicators over time or with increasing axle load applications.

PAVEMENT RECONSTRUCTION—The complete rebuilding or replacement of pavement structure.

PAVEMENT REHABILITATION—The work undertaken to extend the service life of an existing facility. This work comprises placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy.

PAVEMENT SECTION—A length of roadway or experimental unit for which performance measurements are made.

PAVEMENT PERIOD—The actual length of time for which a particular pavement design provides acceptable service.

PERFORMANCE-RELATED M&C FACTORS—Those characteristics of materials and/or construction that individually or interactively have an influence on pavement performance.

PRESENT SERVICEABILITY INDEX (PSI)—A number derived with a formula for estimating the serviceability rating from measurements of certain physical features of a pavement section.

R^2, COEFFICIENT OF VARIATION—The fraction of variation in the data that is accounted for by the regression model.

RELIABILITY—The probability, expressed as a percentage, that the actual traffic carried by a particular pavement section, before it reaches an unacceptable service level, is greater than the design period traffic.

SALVAGE VALUE—The value of a particular facility at the end of a life-cycle or analysis period.

SENSITIVITY ANALYSIS—A technique used to assess the relative effect of a change in the input variable(s) on the resulting output.

SERVICEABILITY LEVEL—The ability, at the time of observation, of a pavement section to serve traffic.

STRUCTURAL PERFORMANCE—The deterioration in pavement structural condition over time or with increasing axle load applications.

TERMINAL SERVICEABILITY LEVEL—The serviceability level at which the pavement section undergoes major rehabilitation in order to prevent further serviceability loss and to restore a new high level of serviceability.

TRAFFIC—The total number of equivalent 18,000-lb single-axle load applications (ESAL) that a pavement section carries during any particular time period.

USER COSTS—The costs incurred by a user of a road facility which arise from user delays or vehicle operating costs.

APPENDIX B—REVIEW OF PERFORMANCE MODELS FOR HOT-MIX ASPHALT PAVEMENTS

INTRODUCTION

Pavement performance is usually defined as the time-dependent trend in serviceability and is often expressed or plotted as a function of the number of 18-kip equivalent axle load applications. The serviceability of a pavement at any point in time is measured by pavement condition indicators. Consequently, performance is defined herein as the history of a pavement condition indicator(s) over time or with increasing axle load applications.

Pavement performance can be categorized into functional and structural performance. Functional performance relates to the ability of a particular roadway to fulfill its function of serving traffic and is usually defined in terms of pavement condition indicators such as roughness or skid resistance. Structural performance, on the other hand, relates to the deterioration in pavement structural condition over time or with increasing axle load applications. Cracking and rutting are examples of pavement condition indicators used to quantify structural pavement performance. These and other pavement condition indicators are listed in Table B-1.

A method for predicting pavement performance is an important element in the development of performance-based specifications. As part of the work plan for this study, performance models for flexible pavements were reviewed. Table B-2 represents a summarization of selected models from the literature that are available for predicting pavement performance in terms of several pavement condition indicators identified in Table B-1. The performance models reviewed are discussed in greater detail in the following sections of this appendix.

PERFORMANCE MODELS FOR PREDICTING FATIGUE CRACKING

Information on available performance models that predict the occurrence of fatigue cracking in the asphalt-bound layer of a flexible pavement structure is given in Table B-2. Most of the models reviewed were developed using a mechanistic-empirical approach in which the number of load applications to failure is determined by some measured value of the response of the pavement to load. The measured response, usually surface deflection, is generally related to the tensile strain at the bottom of the bituminous layer. This strain is then statistically correlated with pavement performance (40-50). Tensile stress is also used as a criterion for fatigue cracking in two of the models, namely, those developed for Arizona and the USSR (51,52).

Mechanistic-empirical models are generally developed in one of two ways: (1) through correlation of observed pavement performance with theoretically determined pavement response parameters (e.g., asphalt tensile strain, asphalt tensile stress), and (2) through laboratory fatigue testing of asphalt concrete samples. Models formulated using the first approach include the ARE and OAF models (40,45), which were developed from AASHO Road Test data. The British Analytical Pavement Design model (ANPAD) and those of the Asphalt Institute and Shell organizations (43,44,46,47) are examples of models formulated using the second approach. To account for differences between laboratory and field conditions, models developed from fatigue testing are usually adjusted, through application of shift or

Table B-1. Condition indicators identified as being significant for development of performance-based specifications.

Condition Indicator	Method of Measurement	Variables Affecting Deterioration in Pavement Condition: Materials	Plant	Construction	Environmental	Traffic
1. Cracking						
a. Fatigue	Visual/video surveys	• asphalt grade • asphalt source • aggregate type • additives	• asphalt content • aggregate gradation • air voids	• density (% max.) • thickness	• precipitation • freeze-thaw • temperature	X
b. Thermal	Visual/video surveys	• asphalt grade • asphalt source • aggregate type • additives	• asphalt content • aggregate gradation • air voids	• density (% max.) • thickness	• temperature	-
C. Shrinkage	Visual/video surveys	• asphalt grade • asphalt source • additives	• asphalt content • aggregate gradation • air voids	• density (% max.)	• temperature	-
2. Permanent deformation (rutting)	Measurement of transverse profile	• asphalt grade • asphalt source • aggregate type • additives	• asphalt content • aggregate gradation • air voids	• density (% max.) • thickness	• temperature • freeze-thaw • precipitation	X
3. Serviceability or roughness	Measurement of longitudinal profile	• may be influenced by materials and plant variables that affect cracking and rutting		• initial roughness after construction	• temperature • freeze-thaw • precipitation	X
4. Skid resistance	Locked wheel skid number	• aggregate type • asphalt source • asphalt grade	• asphalt content • air voids	• initial skid resistance after construction	• temperature • precipitation	X
5. Ravelling	Visual/video surveys	• aggregate type • asphalt source • additives • asphalt grade	• asphalt content • air voids • aggregate treatment	• segregation	• temperature • precipitation	X
6. Moisture damage	• Loss of modulus • Coring/ destructive testing	• asphalt source • aggregate type • additives • asphalt grade	• asphalt content • air voids • aggregate treatment	• density (% max.)	• temperature • precipitation	X
7. Wear resistance	Currently not considered in pavement condition surveys	• aggregate type	• asphalt content • air voids • amount of coarse and fine aggregates • max. aggregate size	• density (% max.)	• temperature • freeze-thaw	X

Table B-2. Summary of performance models for flexible pavements.

Distress Mode	Example Models	Input Parameters Related to Asphalt Mix
Fatigue Cracking	ARE	AC mix modulus AC mix Poisson's ratio
	Asphalt Institute	AC mix modulus Binder content Air voids content AC mix Poisson's ratio
	VESYS Cracking Model	AC mix fatigue properties (k_1, k_2) AC mix modulus AC mix Poisson's ratio
Low-Temperature Cracking	Cold	AC mix modulus-temperature relationship AC mix tensile strength-temperature relationship Thermal conductivity of AC mix Heat capacity of AC mix Absorptivity of AC mix Emissivity of AC mix Convection coefficient of AC mix
	Shahin-McCullough Model for Low-Temperature Cracking	Air voids content Binder content Volume concentration of aggregates Specific gravity of asphalt Specific gravity of aggregate AC mix coefficient of thermal expansion Asphalt penetration index Asphalt softening temperature Absorptivity of AC mix Conductivity of AC mix
Thermal Fatigue Cracking	Lytton-Shanmugham Model	Ring and ball softening point Asphalt penetration Volume concentration of aggregates Air voids content Binder content

Table B-2. Summary of performance models for flexible pavements (continued).

Distress Mode	Example Models	Input Parameters Related to Asphalt Mix
	Shahin-McCollough Model for Thermal Fatigue Cracking	Air voids content AC mix fatigue properties (k_1, k_2) Binder content Volume concentration of aggregates Specific gravity of asphalt Specific gravity of aggregate Asphalt penetration index Asphalt softening temperature AC mix Poisson's ratio
Rutting	VESYS Rut Depth Model	AC mix modulus AC mix Poisson's ratio Permanent deformation properties of AC mix (μ,α)
	Shell	AC mix modulus AC mix Poisson's ratio Bitumen viscosity Bitumen penetration Penetration index
	AGIP (Italian Asphalt Pavement Design Procedure)	Creep compliance function for AC mix
PSI/Roughness	PDMS	AC mix modulus AC mix Poisson's ratio
	AASHTO	AC mix modulus
	VESYS Roughness Model	AC mix modulus AC mix fatigue properties (k_1, k_2) Permanent deformation properties of AC mix (μ, α) AC mix Poisson's ratio
	Fernando Model	AC mix modulus AC mix Poisson's ratio

correction factors, to account for rest periods, crack propagation, and the transverse distribution of wheel loads. These shift factors vary greatly among agencies. For example, a factor of 100 is used in the British ANPAD model for fatigue cracking (46), while a factor of 18 is used in the Asphalt Institute model (53). The South African design procedure, for thick bituminous bases, applies shift factors that vary from 2 to 10, depending on the functional classification of the road (50). Because shift factors are influenced by a host of variables, such as the type of laboratory test, the type of pavement structure (e.g., thick or thin asphalt layer), traffic and environmental conditions, and the acceptable degree of cracking, it is very difficult to quantitatively determine shift factors that can be applied universally to a wide range of conditions. This explains why shift factors vary greatly from agency to agency and raises the point that shift factors are valid only when applied to the specific methodology and under the same prevailing conditions from which they were developed.

In addition to the mechanistic-empirical models, empirical and mechanistic fatigue cracking models are also available. In general, empirical models are developed on the basis of observed pavement performance and the effects of other observed factors upon performance. The Texas Flexible Pavement Design System model (FPDS) for predicting the degree of alligator cracking (54) is an empirical fatigue cracking model. Mechanistic performance models, on the other hand, refer to models that employ theoretical relationships based upon engineering mechanics to predict pavement performance. To predict performance, these models require values for the fundamental mixture response variables and the number of repeated load applications. The fatigue cracking model developed for Ohio is an example of a mechanistic performance model which incorporates principles of fracture mechanics (55). In the development of the model, a mode-one type of crack propagation was assumed, in which the rate of crack growth is related to a stress intensity factor that governs the magnitude of the local stresses in the vicinity of the crack tip. Laboratory tests are required for evaluating the constants in the crack growth model, and failure is assumed to occur when a crack propagates to the surface of the asphalt layer. No correction factors are applied to account for either the formation of alligator cracking after a crack becomes visible at the surface, or the effects of rest periods and the transverse distribution of wheel loads.

Of the 13 models reviewed, only 3 (the Texas FPDS and the modified ILLIPAVE and VESYS cracking submodels) predict the degree of cracking as a function of the number of load applications. The other 10 models predict only the number of load applications before a failure condition is reached. Both the VESYS and modified ILLIPAVE algorithms are based on Miner's hypothesis, in conjunction with a phenomenological fatigue model, to define a crack index C, where a C value of 1 indicates a failure condition. The probability density function of the crack index is determined from a stochastic analysis, and the percentage of cracked area is taken simply as the area under the probability density curve defined by $C \geq 1$. The prediction of pavement performance, or the condition indicators, as a function of time is essential to development of a performance-related specification.

Performance must be known as a function of time in order to calculate user costs.

It is interesting that 11 of the 13 models utilize linear elastic layered theory to determine pavement response. Even the latest version of VESYS uses a quasi-elastic approach to obtain the viscoelastic solution (42). The use of linear elastic layered theory, according to Monismith and Witczak (56), is reasonable if the time-dependent and nonlinear response of paving materials is recognized. Brown and Pell (57) favor the use of a successive approximation technique in which linear elastic layered theory is applied iteratively to obtain stress-compatible moduli for the unbound pavement layers.

PERFORMANCE MODELS FOR PREDICTING THERMAL CRACKING

Thermal cracking can be categorized into low-temperature cracking and thermal fatigue cracking. Low-temperature cracking occurs when thermally induced tensile stresses exceed the tensile strength of the asphalt concrete mix. In this form of thermal cracking, tensile stresses develop as a result of shrinkage caused by very cold temperature in the asphalt concrete. In the models reviewed, thermal stress is typically calculated from a pseudoelastic beam analysis, incorporating information on the time and temperature history of the pavement, asphalt, and mixture properties, and the thermal coefficient of expansion for asphalt concrete (53,58). The calculated values of thermal stress for specific temperature increments are then compared with the tensile strength of the asphalt concrete mix to determine the occurrence of low-temperature cracking.

Thermal fatigue cracking is predicted with a phenomenological fatigue model in the Shahin-McCullough procedure (58) and through a fracture mechanics approach in the model developed by Lytton and Shanmugham (59). The methodology followed in the Shahin-McCullough procedure is identical to that used in several models for predicting load-associated fatigue cracking. A phenomenological model relating the number of thermal cycles to failure to the thermal tensile strain is used together with Miner's hypothesis for determining the occurrence of thermal fatigue cracking.

In the fracture mechanics model developed by Lytton and Shanmugham (59), cracks are assumed to begin at the surface of the pavement and propagate downward as temperature cycling occurs. The rate of crack growth is modeled using the Paris and Erdogan equation, from which the number of temperature cycles required to crack a pavement can be calculated. The mechanistic model calculates the change in the stress-intensity factor which results from daily temperature cycles. Stress-intensity factors are calculated from regression equations generated from a multifactorial experiment using a finite element model of a multilayered pavement structure. Fracture parameters are determined empirically from consistency properties of the bitumens.

Field data collected in Michigan were used to validate the model, and regression analysis of observed crack frequencies on theoretical cumulative damage indices generated by the model indicates a standard error of the estimate of approximately plus or minus one crack in 50 ft (60). The mechanistic model is not directly applicable to the design of individual roadway segments because its operation requires large amounts of computer time and very detailed temperature data.

However, the model can be used to develop empirical design equations calibrated to local conditions.

A more comprehensive review of thermal cracking models including a sensitivity analysis of selected models can be found elsewhere (61). Most of the models that consider thermal cracking include binder properties as the major dependent variables (10,53), and, therefore, thermal cracking is probably more correctly addressed in a binder specification. Of the different models that have been suggested in the literature, the Shahin-McCallough (58) and the Lytton models (59) are the most realistic although additional development work is needed before they can be implemented in a binder specification (61). Statistical models, such as those developed by Hajek and Haas (62) and Fromm and Phang (63) are of limited value because they were developed for limited conditions, and, because they are not rational in nature, cannot be extrapolated to climatic conditions, materials, and pavement designs.

PERFORMANCE MODELS FOR PREDICTING RUTTING

Performance models for predicting the occurrence of rutting can be classified into (1) models which predict the number of load applications prior to the development of an unacceptable level of rutting, (2) models which predict total pavement deformation at any given number of load applications, and (3) models which estimate the permanent deformation in one or more layers for any given number of load applications. Models in the first category typically use a limiting criterion for subgrade vertical strain to ensure that the total amount of rutting will not exceed some specified failure level during the design life of the pavement. Examples of these models include those implemented in the British, Asphalt Institute, and Shell design procedures (43,44,45,47). These models predict only when a failure condition will be reached, and not the magnitude of rutting which will occur. Consequently, these models may not be suitable for certain design or research problems where estimates of actual magnitudes of rutting are required. For these cases, models in the second and third categories are needed. Inasmuch as these models can predict the amount of rutting for any given number of axle load applications, they can be used for a much wider range of applications.

The difference between category 2 and category 3 models is that the former models predict only the total amount of pavement rutting that would occur, whereas the latter models generally predict the individual layer deformations, which are then added together to obtain an estimate of the total amount of rutting that would develop. The degree of sophistication varies widely among the models in category 2. These models range from empirical (e.g., Texas FPDS rutting model (54)) to mechanistic (e.g., VESYS rutting submodel (41,42)). There are also models which can be described as mechanistic-empirical, such as those implemented in the Probabilistic Distress Models for Asphalt Pavements (PDMAP) and Waterloo Model of Distress Estimation (WATMODE) systems (53,64,65). The PDMAP rut depth model was developed using data on 32 AASHO flexible pavement sections. In the development of this model, the observed rates of rutting were correlated with the cumulative number of 18-kip equivalent single-axle loads and with theoretically determined values for surface deflection and vertical compressive stress at the bottom of the asphalt concrete. The WATMODE model predicts rut depth as a function of the equivalent pavement thickness, the number of 18-kip equivalent single-axle loads, and the elastic moduli. Laboratory-determined relationships defining the permanent deformation behavior of various materials were used to calculate rut depths for many typical pavement designs. A sensitivity analysis was also conducted to identify variables which significantly influence the development of permanent deformations. These variables were subsequently correlated with the calculated values of rut depths, resulting in the prediction equation for rutting used in the WATMODE system.

An example of a mechanistic model in category 2 is the VESYS predictive submodel for rutting. In the VESYS model, the assumption is made that the accumulation of permanent deformation can be reasonably represented as a logarithmic function in N, the number of load applications, as expressed by the following equation (42):

$$Y_p(N) = Y\,\mu_{sys}\,N^{-\alpha_{sys}} \qquad \text{(B-1)}$$

where

$Y_p(N)$ = incremental permanent deformation per load application

Y = pavement deflection under load

μ_{sys}, α_{sys} = permanent deformation properties of the pavement system, which are determined internally in the computer program

The system permanent deformation parameters, μ_{sys} and α_{sys} are determined using linear elastic layered theory in conjunction with laboratory-determined relationships that define the permanent strain behavior of each pavement layer. The permanent strain behavior for all materials is modeled as (42):

$$\epsilon_p(N) = \epsilon\mu N^{-\alpha} \qquad \text{(B-2)}$$

where

$\epsilon_p(N)$ = permanent strain per load pulse

ϵ = peak haversine load strain for a load pulse of 0.1 sec duration measured at the 200th repetition

μ,α = material constants for each layer material (derived from a regression analysis of laboratory data)

The assumption is made that the peak load strain at any cycle is composed of permanent and resilient (or recoverable) strain components. Consequently, the permanent strain at any cycle can also be expressed as:

$$\epsilon_p(N) = \epsilon - \epsilon_r(N) \qquad \text{(B-3)}$$

where

$\epsilon_r(N)$ = resilient or recoverable strain at any load cycle

Substituting Equation B-2 into Equation B-3 and dividing by the magnitude of the stress pulse gives the following expressions for the resilient compliance, $D_r(N)$, and the resilient relaxation modulus, $E_r(N)$:

$$D_r(N) = D(1-\mu N)^{-\alpha} \qquad \text{(B-4)}$$

$$E_r(N) = \frac{|E^*|N^{\alpha}}{N^{\alpha} - \mu} \qquad \text{(B-5)}$$

where

D = peak haversine load compliance at 200 load repetitions

$|E^*|$ = peak haversine load modulus at 200 load repetitions

In the VESYS model, Equation B-5 is used to compute a vector of resilient relaxation moduli, $E_r(N)$, for each layer material and for selected values of N. The moduli obtained are used in a linear elastic layered analysis to compute pavement system resilient, or unloaded, responses, $S_r(N)$. Thus, layered theory is used in a negative sense. Likewise, the peak haversine load modulus, E, for each layered material is used to compute the pavement system load response, S, and the incremental permanent deformation at any load cycle is subsequently computed as:

$$S_p(N) = S - S_r(N) = S\mu_{sys}N^{-\alpha_{sys}} \tag{B-6}$$

By regression analysis, the coefficients μ_{sys} and α_{sys} are obtained from the known values of $S_p(N)$ and S. In addition, the integration of Equation B-1 with respect to N yields the following expression for rut depth for a given analysis period (at constant temperature and rate of loading):

$$R = Y \bullet F(N) \tag{B-7}$$

where

R = accumulated rut depth

$$F(N) = \frac{\mu_{sys} N^{1-\alpha_{sys}}}{1 - \alpha_{sys}}$$

The preceding discussions show that VESYS offers a sophisticated prediction procedure for rutting which can be used under a wide range of conditions. However, because the procedure requires extensive laboratory testing and arbitrary "correction" factors to obtain realistic rut depth estimates, its use has generally been limited to research applications.

The third category of rut depth prediction models includes procedures for predicting the rutting that occurs in each pavement layer. This approach predicts rut depths by using laboratory-determined permanent deformation relationships in conjunction with either linear elastic layered theory or finite elements (66). To predict the amount of permanent deformation that would occur after a given number of load applications, each layer of the pavement structure is divided into several sublayers, as illustrated in Figure B-1. The stress state beneath the wheel load, at the center of each sublayer, is determined using an appropriate theory. The axial plastic strain is subsequently estimated from the results of repeated load tests characterizing the permanent deformation behavior of each layer material, and from the computed stress state. The total rut depth beneath the wheel load is then obtained by summing all products of the average plastic strain occurring at the center of each sublayer and the corresponding sublayer thicknesses.

Models which use the layer strain approach include those developed by Romain, Barksdale, Monismith et al., and Kirwan et al. (67-70), and the rut depth submodel incorporated in the modified ILLIPAVE version (48). The use of elasticity theory to determine stresses and strains in these procedures is only approximate in the sense that permanent deformation is assumed to have negligible effect on the stress and strain distribution. In concept, viscoelastic or viscoelastic plastic theory should be used, but because of the mathematical complexities involved, approximate methods (e.g., linear elastic layered theory, finite elements) have found wider application. Models do exist, however, which are based on linear viscoelastic

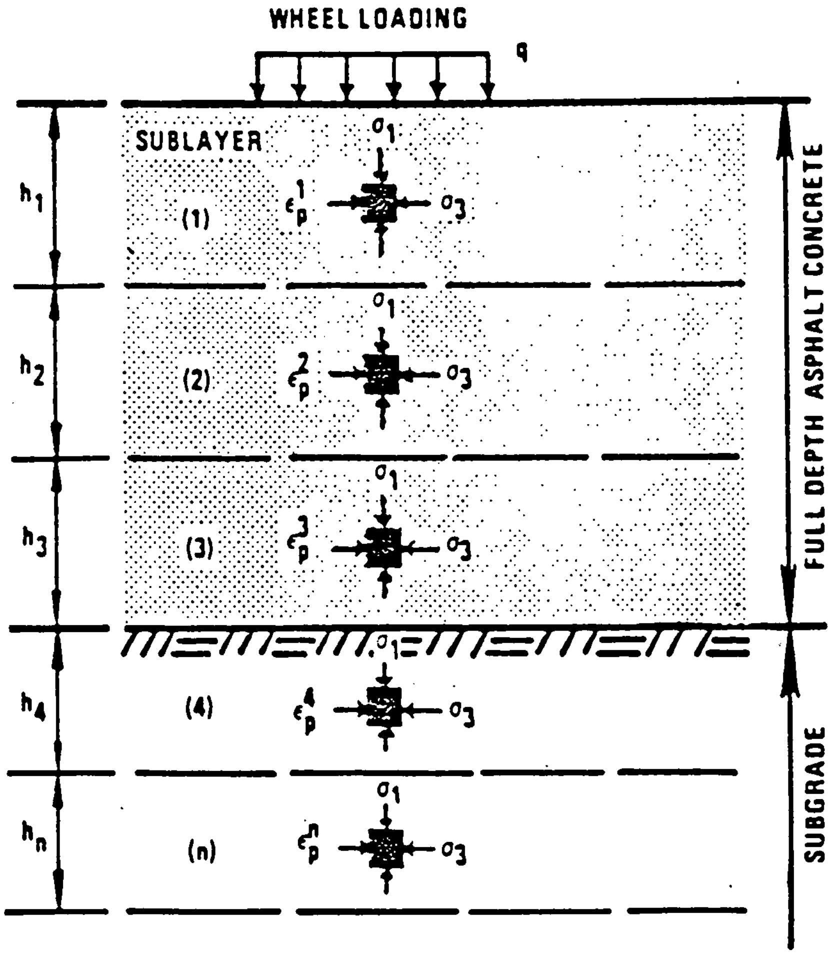

Figure B-1. Division of pavement layers into series of sublayers for calculation of rut depth using layer-strain approach (66).

theory. An example is the rut depth model incorporated in the AGIP system developed in Italy (71). This model characterizes viscoelastic behavior by a Maxwell model, and creep testing is required to determine the creep compliance function for each pavement material. These material properties are used as inputs to a computer program for determining the permanent deformation of each pavement layer.

The primary advantage of a viscoelastic approach is that moving loads can be considered directly. In a viscoelastic material, the properties are dependent upon the frequency of loading. As indicated in Figure B-2, the pulse loading time for a moving load increases with depth as a result of the distribution of the stresses. Consequently, stationary and moving loads would result in different predicted rut depths according to viscoelastic theory. This problem can be handled indirectly through procedures based on elasticity theory, by testing materials under stress conditions and durations of loading compatible with the depth at which the material would be placed in the field and with the normal range of vehicular speeds expected (66).

PERFORMANCE MODELS FOR PREDICTING PSI/ROUGHNESS

The AASHTO performance equation is a commonly used empirical performance model for predicting pavement deterioration in terms of PSI (1). This equation was developed with data from the AASHO Road Test (28) and predicts the number of 18-kip equivalent single-axle loads (ESAL's) before the PSI reaches a specified terminal serviceability level. The equation is empirically derived and relates the number of 18-kip ESAL's to pavement layer thicknesses, soil support, and environmental conditions.

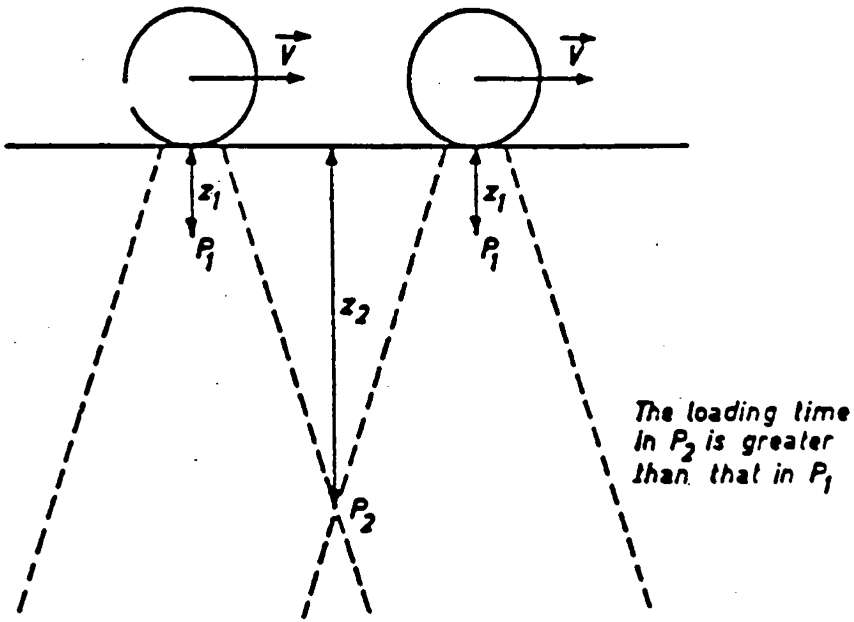

Figure B-2. Effect of moving wheel load on pulse loading time (71).

Other empirically derived performance models are the Texas FPDS prediction model for PSI and the Highway Design and Maintenance (HDM) model for roughness, developed by the World Bank (56,72). The PSI prediction equation for the Texas model was developed from data collected on in-service pavements. The deterioration in PSI with number of 18-kip ESAL's is modeled using a sigmoidal, or S-shaped, curve that recognizes a change in the rate of deterioration of a pavement as the traffic level accumulates. The sigmoidal curve is defined by the function

$$g' = e^{-(\rho/N)^{\beta}} = \frac{p_i - p}{p_i - p_f} \qquad \text{(B-8)}$$

where

- p_i = initial PSI
- p = PSI at time t
- p_f = final PSI or terminal serviceability level
- N = number of 18-kip ESAL's
- ρ, β = equation constants

Prediction equations for ρ, β, and p_f were determined by conducting regression analysis of the available field performance data. The regression equations are functions of the transformed pavement thickness, the plasticity index of the subgrade soil, the maximum Dynaflect deflection, and environmental variables.

The HDM model for roughness was developed primarily from a statistical analysis of data collected on in-service pavements in Brazil (72). The model estimates the change in roughness over a certain time interval as a function of the initial roughness, the equivalent axle loadings, and the incremental increases in rut depth, cracking, surface patching, volume of potholes, and the standard deviation of these variables. Because information on other forms of pavement distress is needed to predict changes in roughness, the model is difficult to use for long-term performance predictions and is applicable only for scheduling maintenance needs.

Other models available for predicting PSI loss with number of load applications are those incorporated in design procedures developed by Luhr and McCullough for the U.S. Forest Service (73,74). These design procedures are the Simplified Pavement Design Procedure (SPDP) and the procedures incorporated in the system framework of the Pavement Design and Management System (PDMS). The SPDP and PDMS performance models were developed through correlation of observed AASHO performance data with theoretically determined pavement response parameters (e.g., subgrade compressive strain, asphalt tensile strain). Pavement response parameters were calculated using linear elastic layered theory in conjunction with laboratory-determined properties for the AASHO Road Test materials. For the PDMS models, an iterative procedure for the calculation of pavement response was used to deal with the stress dependency of unbound pavement materials.

Roughness submodels are also incorporated in the modified ILLIPAVE and VESYS programs (41,42,48). In these submodels, a mathematical relationship between slope and rut depth variances is used to predict the progression of pavement roughness. In both the VESYS and modified ILLIPAVE procedures, the predictions of slope variance from the roughness submodel are combined with the individual predictions from the rut depth and cracking submodels to obtain PSI values, using the PSI equation developed at the AASHO Road Test. Thus, the decrease of PSI with traffic is also determined. These procedures represent the state-of-the-art in pavement performance prediction methodologies, but because they require sophisticated laboratory testing and large computer resources, their application to routine pavement design would be difficult. Additional field validation of these procedures is required. The comparison of VESYS results, for example, with observed performance in pavement test sections has been erratic (5). Rut depth predictions have sometimes been good, but a recent study showed that a version of VESYS that was calibrated to observed AASHO Road Test data did not predict PSI for field highway sections as well as did the original AASHO performance equation for flexible pavements. One reason for this may be the theoretical relationship assumed in VESYS between rut depth variance and slope variance. Although this relationship is conceptually appealing, it is not apparent in highway and AASHO Road Test sections where slope variance and rut depth variance were measured (Figure B-3).

SKID RESISTANCE MODELS

Anderson, Rosenberger, and Meyer developed a procedure for predicting end-of-season skid numbers from skid-resistance measurements made at any time during a given season (75). The procedure is empirical in that it is based on a prediction equation developed through regression analysis of data obtained from field test sites. Variables included in the prediction equation are the dry spell factor (DSF), air temperature at the time of test (AIRT), Julian calendar day (JDAY), average daily traffic (ADT), and the skid number measured on

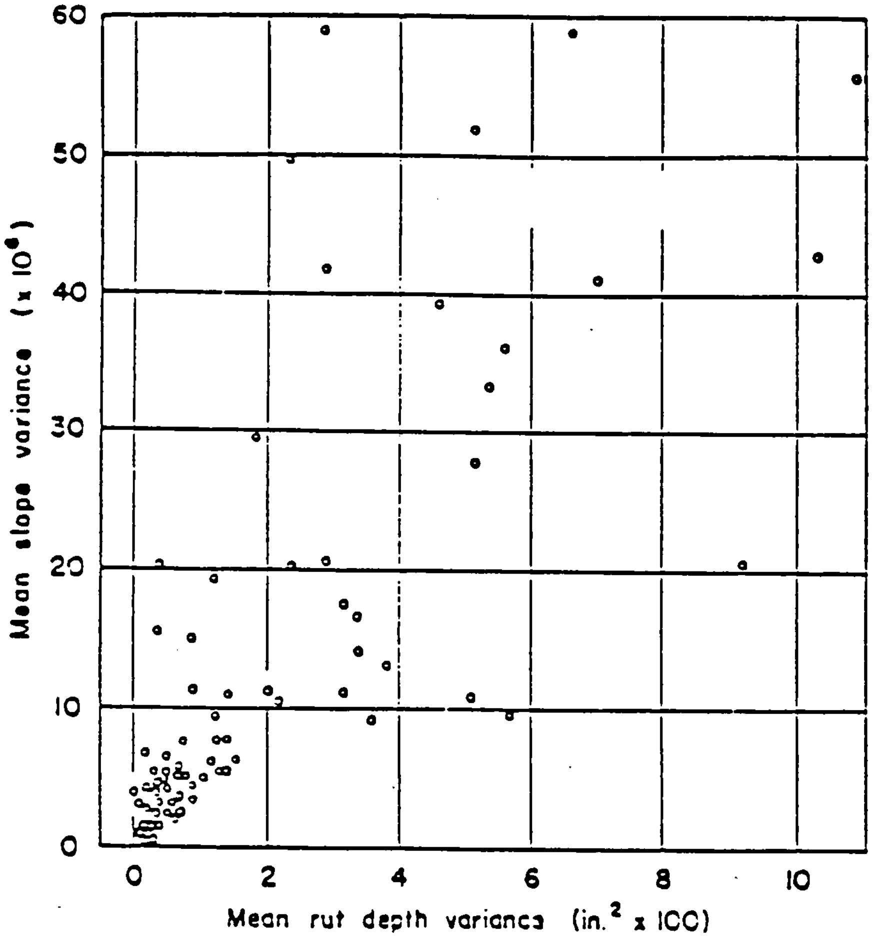

Figure B-3. Mean slope variance versus mean rut depth variance for 74 flexible pavement sections (28).

any arbitrary day within a given season. The model assumed in the analysis is expressed in the following form:

$$\ln SN_{64} = b_0 + b_1\, SN_{64F} + b_2\, ADT + b_3\, JDAY + b_4\, DSF + b_5\, AIRT + error \quad \text{(B-9)}$$

where

SN_{64} = skid resistance measured at 64 km/h (40 mph)

SN_{64F} = end-of-season skid resistance relative to a vehicle speed of 64 km/h (40 mph)

b_0, b_1, b_2, b_3, b_4, and b_5 = coefficients determined from regression analysis

In the prediction model, short-term (day-to-day) adjustments in skid number are accounted for by the dry spell factor and air temperature, whereas long-term adjustments (within season) are accounted for by Julian calendar day and average daily traffic. Only variables that can be readily obtained were included in the procedure so that it can be implemented by a typical highway agency.

The coefficients of the model given in Equation B-9 were determined using data from New York, Pennsylvania, and Virginia and are presented in Table B-1, along with the standard errors of the estimate. Although the model was developed primarily for predicting end-of-season skid numbers, it can also be used to estimate the amount of traffic required for the skid resistance to deteriorate from an initial value (SN_{64}) to a specified terminal skid level (SN_{64F}).

The coefficients in the prediction equation were found to be site-specific and varied from season to season. Therefore, caution must be exercised when applying the coefficients given in Table B-3 to other regions, and it is recommended that the coefficients in Equation B-9 be determined by each highway agency, using skid resistance data collected at the agency's test sites. The coefficients should be verified in subsequent years to confirm that they do not vary excessively from year to year.

Other models for predicting skid resistance are summarized in Table B-4. These models were also derived empirically from regression analyses of skid resistance data (64). Each equation predicts the skid resistance, relative to a vehicle speed of 40 mph, as a function of the number of load repetitions and aggregate type. The types of aggregate considered vary from a rapidly polishing, soft limestone to a group of relatively nonpolishing materials.

In a subsequent study (76), the procedure to normalize skid resistance for operational, seasonal, and weather effects was further refined. Many of the same factors that were in the earlier model are included and the effects of test speed are also taken into account. Procedures for generating a set of coefficients for a region by pavement class are described. A program to implement the procedure on a micro computer was also developed.

DISCUSSION OF PAVEMENT PERFORMANCE MODELS

A central element in the development of performance-related specifications (PRS) is a model for predicting the deterioration in pavement condition with the number of load applications. The deterioration in pavement condition can be expressed in terms of several pavement condition indicators such as cracking, rutting, and roughness. A variety of performance prediction equations is available to the pavement engineer, and these models have been reviewed herein.

Table B-3. Estimated coefficients of skid resistance model given by Equation B-9.

Bituminous Concrete

	Site-specific model coefficients, $b_j \pm s(b_j)$*			
Variable	New York	Pennsylvania	Virginia	Combined
Observations	461	832	623	1,916
Intercept	2.82 ± .0838	2.71 ± .0276	2.95 ± .0401	2.71 ± .022
SN64F	.0353 ± .00097	.0338 ± .00044	.0295 ± .00097	.0346 ± .000385
ADT/1000**	.0115 ± .0116	.0130 ± .0131	.0275 ± .0039	.0395 ± .00419
JDAY	-.00139 ± .000176	-.000918 ± .000053	-.00118 ± .000042	-.0011 ± .00004
DSF	-.027 ± .00679	-.0358 ± .00371	-.0176 ± .00345	-.026 ± .0023
AIRT	-.00044 ± .00053	-.000727 ± .000235	-.000799 ± .000238	-.00035 ± .00017

*$b_j \pm s(b_j)$ where b_j is the state-specific coefficient of variables in the model and s is the standard deviation of associated coefficient.

**ADT/1000 is used so that coefficient values are not intractably small.

Model: $\ln SN_{64} = b_0 + b_1\, SN_{64}F + b_2\, ADT/100 + b_3\, JDAY + b_4\, DSF + b_5\, AIRT + \epsilon$

Table B-4. Skid resistance models (64).

Aggregate Type	Description	Prediction Equation
Soft	Texas Georgetown Limestone	$SN = 34.6\ (N/10^6)^{-0.136}$
Soft	Central and Northern Florida	$SN = 45.4\ (N/10^6)^{-0.222}$
Soft	Virginia Limestone	$SN = 44.7\ (N/10^6)^{0.1964}$
Soft	Texas Burnett Dolomite	$SN = 40.4\ (N/10^6)^{-0.121}$
Soft	Kentucky Limestone	$SN = 46.9$ at $N = 10^6$
Soft	Wisconsin Dolomite	$SN = 43.1$ at $N = 10^6$
Soft	Georgia Limestone	$SN = 72.5\ (N/10^6)^{-0.128}$
Hard	Texas Traprock	$SN = 43.5\ (N/10^6)^{-0.096}$
Hard	Wisconsin Igneous Rock	$SN = 49.5$ at $N = 10^6$
Hard	Texas Iron Slag	$SN = 46.4\ (N/10^6)^{-0.063}$
Hard	Virginia S4, S5 Nonpolishing Aggregate	$SN = 52.1\ (N/10^6)^{-0.058}$
Hard	Georgia Siliceous Aggregate	$SN = 54.8\ (N/10^6)^{-0.044}$

N = number of repetitions of equivalent truck axles.

The number of load repetitions to failure, or the design life, is influenced by numerous factors, some of which, like those related to traffic and climate, the pavement engineer cannot control. (See Table B-5.) Consequently, in the development of PRS, these uncontrollable factors can be accounted for only indirectly by investigating how they influence those factors over which the pavement engineer has control. Figure B-2 illustrates a framework for considering the effects of various factors on pavement performance.

Blocks A and B represent, respectively, factors related to pavement material properties and those related to environmental and traffic loadings. These factors, either separately or interactively, affect the factors in the lower part of the diagram. It can be concluded from the literature review of flexible pavement performance models that most prediction procedures involve going either from levels E through G (of Figure B-2) or from levels C through G. However, for the development of PRS, prediction equations that relate performance directly to level-A factors are required, and one of the objectives of this review was to determine whether such models are available. Very few of the performance models reviewed explicitly include, as predictor variables, the design or construction factors for which specifications are normally developed. Only the British ANPAD system and the Asphalt Institute design procedure use performance models that directly incorporate level-A factors (44,46,47). In both procedures, the performance models for predicting fatigue cracking include the volume of the binder as an independent variable. In addition, the volume of the voids is used as an independent variable in the Asphalt Institute's performance equation.

Table B-5. Factors affecting pavement performance.

<u>Climatic Variation</u>

Temperature
Moisture

<u>Natural Site Variation</u>

Subsurface materials
Natural drainage
Grade and curvature

<u>Material Variation (all layers)</u>

Material properties
Material mix design
Material behavior
Material uniformity

Material supplier and contractor have some control over these variables

<u>Construction Variation</u>

Thickness
Uniformity
Density

<u>Load Variation</u>

Axle configuration and weight
Tire pressure
Position of load on roadway
Dynamic loading effects

<u>Maintenance Variation</u>

Time of maintenance
Type of maintenance
Quality of maintenance

<u>MODELS FOR MIXTURE PROPERTIES</u>

Consequently, for most of the pavement performance models reviewed, surrogate relationships are required in order to determine how pavement performance is affected by level-A factors. Of particular importance for this study are relationships between level-E and level-A factors. These relationships can be established through laboratory testing of pavement materials. However, the need for time-consuming and expensive laboratory testing complicates the design process. Therefore, prediction equations developed for a wide variety of pavement materials are used in many of the design procedures reviewed. These equations relate level-E to level-A factors, and one of the most commonly used is that embodied in Van der Poel's nomograph (<u>77</u>). This nomograph predicts the stiffness modulus of the bitumen as a function of the time of loading, the penetration index, and the temperature at which the bitumen penetration is 800. The nomograph predicts the stiffness modulus of the bitumen within a factor of 2 and is normally used in conjunction with an equation relating bitumen stiffness to mix stiffness. One such equation, developed by Heukelom and Klomp (<u>78</u>) is given by:

$$S_{mix} = S_{bit} \left(1 + \frac{2.5}{n} \; \frac{C_v}{1-C_v}\right)^n \qquad \text{(B-10)}$$

where

$n = 0.83 \log_{10} [4 \times 10^{10}]/[S_{bit}]$

S_{mix} = stiffness modulus of the mix, N/m^2

S_{bit} = stiffness modulus of the bitumen, N/m^2

C_v = volume concentration of the aggregates

The Heukelom and Klomp relationship was developed from test results obtained with mixes having approximately 3 percent air voids and C_v values ranging from 0.7 to 0.9. Inasmuch as the percentage of air voids significantly influences the stiffness modulus of the mix, corrections for air voids content in excess of 3 percent have been proposed. For example, Fijn, van Draat, and Sommer proposed the following correction to C_v for voids content in excess of 3 percent (79):

$$C_v' = \frac{100\ (C_v)}{100 + V_a - 3} \qquad \text{(B-11)}$$

where

V_a = air voids content

C_v = volume concentration of aggregates

C_v' = corrected value for volume concentration of aggregates

Laboratory work conducted by Bonnaure et al. (80) has also led to the development of a nomograph for predicting mix stiffness. This nomograph, shown in Figure B-4, has been incorporated into the Shell design procedure (43). The nomograph relates the stiffness modulus of the mix to the stiffness modulus of the bitumen, the volume percentage of the aggregates, and the volume percentage of the bitumen. The stiffness modulus of the bitumen can be determined by laboratory testing or through application of Van der Poel's nomograph. Extensive measurements on many different asphalt mixes have shown the accuracy of the Bonnaure et al. nomograph, which predicts the modulus within a factor of 1.5 to 2.0 (43).

In addition to the nomographs presented above, the Asphalt Institute has developed an equation for predicting the absolute value

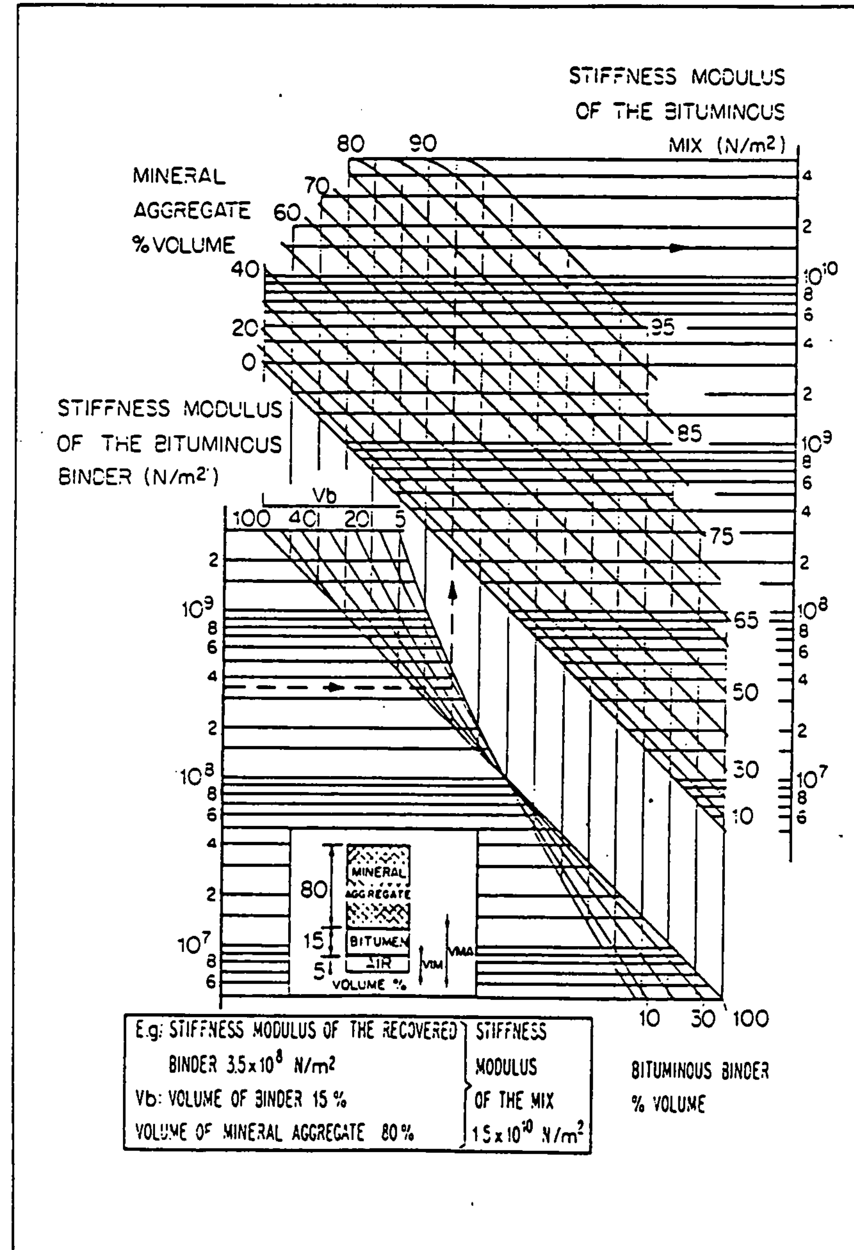

Figure B-4. Nomograph for predicting bituminous mix stiffness (80).

of the complex modulus of asphalt concrete mixes. The original equation, based on laboratory stiffness data from a total of 60 different mixes, has the following general form (44):

$$|E^*| = f\ (P_{200},\ f,\ V_v,\ \eta_{70°F},\ t_p,\ V_b) \qquad \text{(B-12)}$$

where

$|E^*|$ = absolute value of the complex modulus of the mix, psi

P_{200} = percent minus No. 200 sieve

f = frequency of loading, Hz

V_v = percent air voids

$\eta_{70°F}$ = original absolute viscosity of the bitumen measured at 70°F, 10^6 poises

t_p = temperature, °F

V_b = percent volume of binder

This relationship was developed by the Asphalt Institute from results of dynamic unconfined compression tests on a variety of mixes (5). Laboratory testing was performed by the Asphalt Institute, and the initial development of Equation B-12 was reported by Shook and Kallas (81). The form of this equation used in this study was presented earlier as Equation 4. The comparison of measured stiffnesses and those predicted using Equation 4 is shown in Figure B-5 for test results from 41 different mixes obtained at frequencies of 1, 4, and 16 Hz and at 3 temperatures. For the data points compared, the average relative error was about 23 percent.

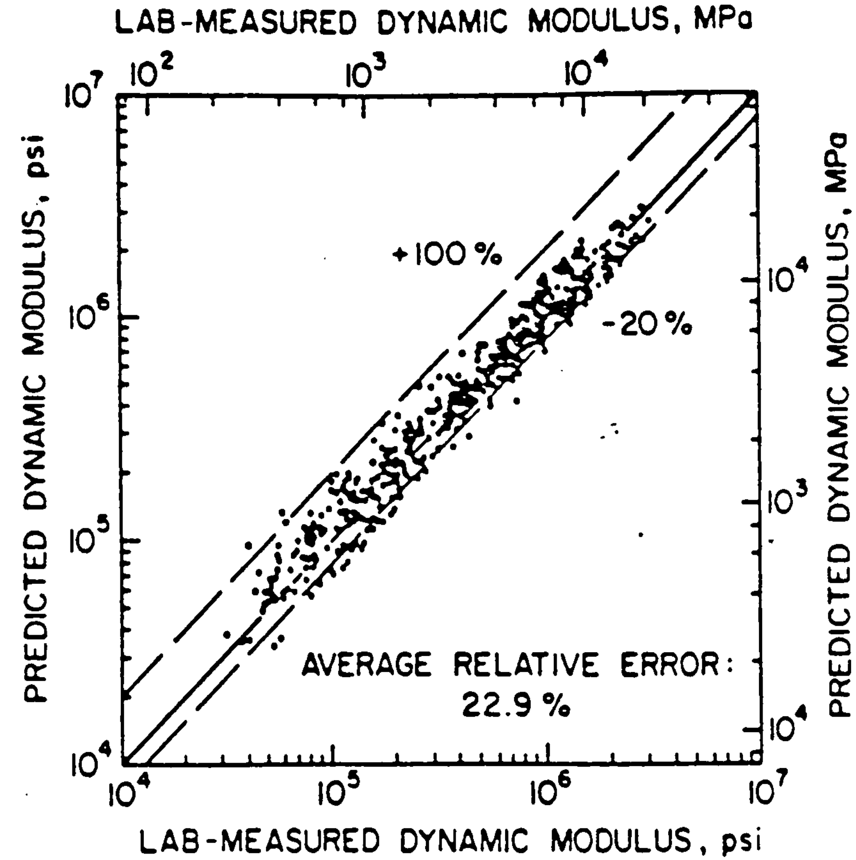

Figure B-5. Comparison of measured dynamic modulus with predicted modulus from Asphalt Institution equation (44).

In addition to prediction equations for the stiffness modulus of asphalt concrete mixes, Cooper and Pell developed a procedure for predicting the laboratory fatigue behavior of bitumen and tar-bound materials (38). Fatigue testing on a wide range of mixes was conducted in the laboratory under controlled stress conditions. Analysis of the data collected showed that the fatigue relationships for the mixes considered intersect at a common point, or focus, as illustrated conceptually in Figure B-6. For the tests conducted, Cooper and Pell observed that the focus occurred at a strain level of 6.30×10^{-4} in/in, and at a life, N, equal to 40 applications. From this result, Cooper and Pell developed a simple procedure for predicting the laboratory fatigue behavior of bituminous mixes. Essentially, the procedure involves the establishment of a mean fatigue life N_1 at a particular value of strain ϵ_1, and the connection of this point to the focus. On the basis of the fatigue test results, a regression equation for predicting the number of allowable applications for a strain level of 1×10^{-4} in/in was developed. The regression analysis showed that the fatigue life at this particular strain level is related to the binder volume and the ring and ball softening point by the equation:

$$\log_{10} N\ (\epsilon = 10^{-4}) = 4.13 \log_{10} V_B + 6.95 \log_{10} T_{R\&B} - 11.13$$

$$R^2 = 0.88 \qquad \text{(B-13)}$$

where

N = number of cycles to failure

V_B = percentage volume of the binder

$T_{R\&B}$ = ring and ball softening point, °C

ϵ = strain, in/in

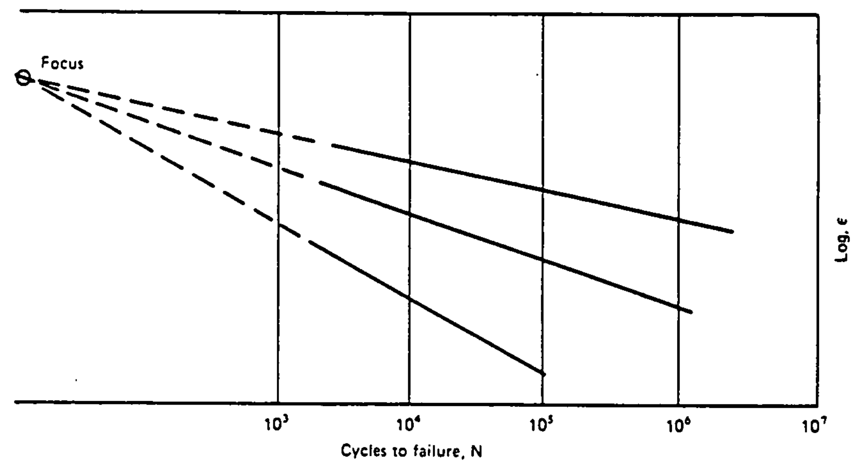

Figure B-6. Conceptual illustration of the intersection of laboratory fatigue relationships at a common point, or focus (38).

This relationship was used to develop the performance model for predicting fatigue cracking in the British design procedure. However, to account for differences between laboratory and field conditions, the fatigue life at any strain level is increased by a factor of 100, which results from a factor of 5 that accounts for the effect of rest periods and from a factor of 20 that accounts for differences in crack propagation times between laboratory and field conditions. These factors were selected from information reported by Raithby and Sterling, Ramsamooj et al., and Van Dijk (82,83,84).

CONCLUSIONS

An attempt was made to review the state-of-the-art in performance prediction methodologies for flexible pavements. A variety of selected performance models have been presented for predicting the deterioration in pavement condition with increasing axle load applications. Performance models were evaluated for their utility in predicting performance on the basis of characteristics that may be included in performance-based specifications. It was concluded that very few of the performance prediction equations reviewed explicitly include as predictor variables the design or construction factors for which specifications are normally developed. Consequently, surrogate relationships for predicting material response characteristics (Level E factors) in terms of asphalt mixture properties (Level A factors) are required, and some of these relationships have been reviewed herein.

It is emphasized that, in the application of a performance prediction equation, the required material response characteristics should be determined from characterization procedures compatible with those used in the development of the performance equation. Studies performed at The Pennsylvania State University have shown, for example, that the resilient modulus obtained from different test procedures can be significantly different, and can lead to substantial discrepancies in calculated pavement response (85). Inasmuch as pavement response is correlated with pavement performance in many rational performance models, serious errors in the performance prediction can occur if the appropriate method of characterizing pavement materials is not used.

APPENDIX C—SENSITIVITY ANALYSIS OF PAVEMENT PERFORMANCE MODELS

A sensitivity analysis is an important tool for evaluating the behavior of a performance or prediction model. A wide range of conditions (levels of the independent variables) that cover the region of interest may be used in the sensitivity analysis. Such an analysis will indicate whether the model responds realistically to the independent variables (i.e., whether the predictions are valid). A sensitivity analysis can also be used to identify the variables that require careful definition. Therefore, sensitivity analyses are a very useful tool for the development of PRS specifications. A sensitivity analysis can be used to:

- Verify that a model gives realistic predictions
- Identify the critical variables so that they may be given more careful laboratory characterization or field measurement
- Identify the critical independent variables in the model so that the allowable level of uncertainty in these variables can be determined

A sensitivity analysis should be conducted for all of the models that are used in PRS specifications.

In this appendix, a sensitivity analysis was conducted for a pavement performance model to illustrate how such an analysis may be accomplished. Specific objectives were: 1) to evaluate the sensitivity of performance predictions to various pavement design factors (i.e., asphalt concrete modulus, layer thicknesses, and coefficients defining the stress dependency of the resilient modulus of unbound pavement materials); and 2) to evaluate the effects of these pavement design factors and their interactions on predicted pavement performance.

PERFORMANCE MODEL FOR THE SENSITIVITY ANALYSIS

The performance model selected for the sensitivity analysis was developed by Fernando et al. (18). The model predicts the trend in pavement surface roughness with cumulative number of load applications, and was developed using performance data collected from flexible pavement sections at the AASHO Road Test (28). The performance model is given in Table C-1.

In the development of the model, pavement failure was assumed to be a function of the response to vehicle loadings, and it was hypothesized that the variation in pavement performance could be explained from the corresponding variation in the theoretical structural response.

While maximum asphalt tensile strain and maximum subgrade compressive strain are the most frequently used variables for predicting pavement performance, strain basin indices, developed from an evaluation of theoretical strain basins, were also examined to evaluate their usefulness as performance prediction variables. These quantities are analogous to deflection basin indices such as Surface Curvature Index (SCI), Base Curvature Index (BCI), or Base Damage Index (BDI), defined in Figure C-1, which are used as indicators of a pavement's structural integrity. Strain basin indices are therefore related to theoretical strains at different locations within a pavement structure. Figure C-2 shows a subgrade compressive strain basin for an 18,000-lb single-axle load.

The importance of strain basins in the evaluation of pavement performance is illustrated conceptually in Figure C-3, which shows plots

Table C-1. Performance model for sensitivity analysis (18).

$$\log_{10}(1 + SV) = (\beta_0 + \beta_1 \log_{10} N)/(1 + \beta_2 \log_{10} N)$$

$$\beta_1 = -0.035 - 0.220\ \beta_0 - 0.035\ \log_{10} V_3 - 0.050\ \log_{10}(1 + H_1)$$

$$\beta_2 = -0.354 + 1.232\ \beta_1 + 0.269\ \sqrt{\beta_0} - 31.958\ V_5 - 0.026\ \log_{10} T_2 + 0.007\ \log_{10}(1 + H_2)$$

where

SV = slope variance

N = cumulative number of load applications

β_0 = initial pavement surface roughness $[\log_{10}(1 + SV)]_i$

H_1 = thickness of the asphalt concrete layer, inches

H_2 = thickness of the base layer, inches

V_3 = $\varepsilon_{sg3} - \varepsilon_{sgmax}$

V_5 = $\varepsilon_{sg2} - \varepsilon_{sg1}$

T_2 = $\varepsilon_{acmax} - \varepsilon_{ac2}$

ε_{sgmax} = maximum vertical compressive strain at the top of the subgrade directly underneath the tire load

ε_{sgi} = vertical compressive strain at the top of the subgrade located along the longitudinal direction at a distance of 'i' feet from the maximum

ε_{acmax} = maximum tensile strain at the bottom of the asphalt concrete layer and directly underneath the tire load

ε_{ac2} = tensile strain at the bottom of the asphalt concrete layer located along the longitudinal direction at a distance of 2 feet from the maximum

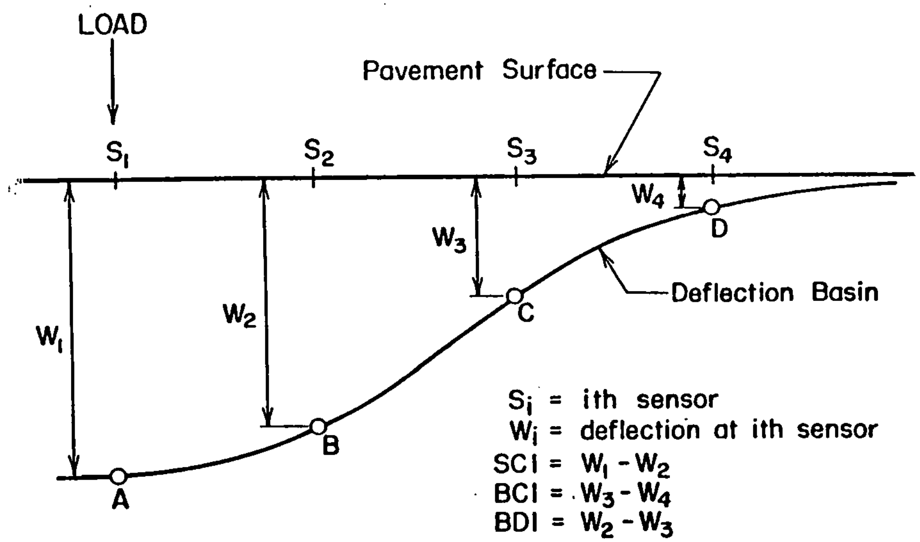

Figure C-1. Example surface deflection basin.

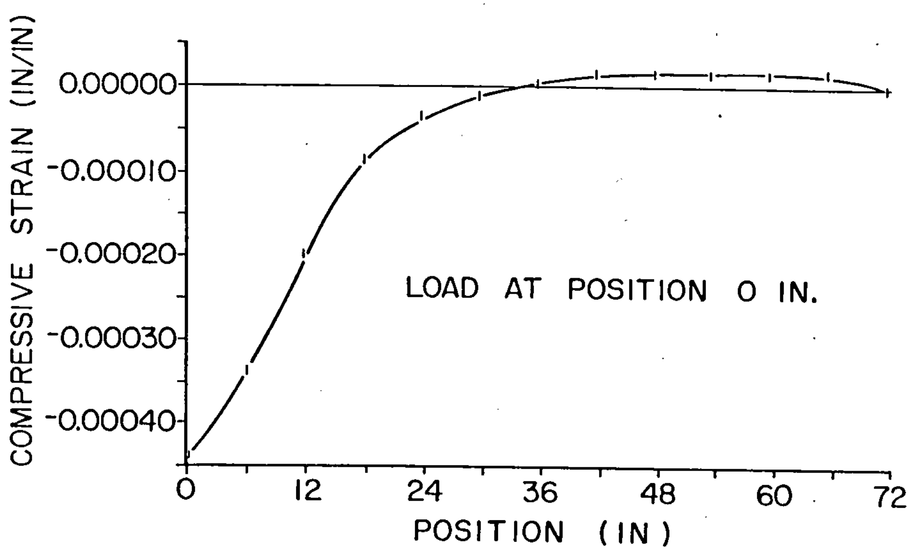

Figure C-2. Subgrade compressive strain basin for an 18-kip single axle load.

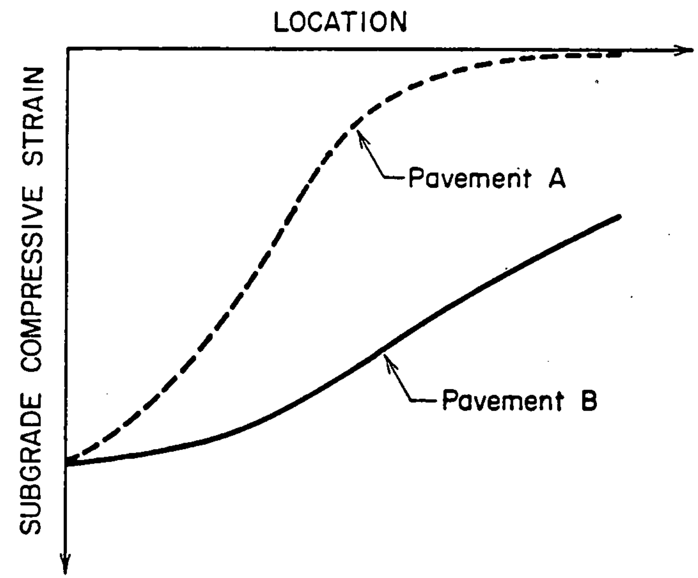

Figure C-3. Conceptual subgrade compressive strain basins for pavements A and B.

of the longitudinal distribution of subgrade compressive strains for two different pavements. If only the maximum subgrade compressive strain is considered, then the two pavements would be characterized as having the same pavement response under a given load. However, it is apparent from an examination of the strain basins in Figure C-3 that this is not the case. The strain distribution across the subgrade for pavement A is different than the strain distribution for pavement B. Inasmuch as pavement performance is logically related to how the pavement responds under a given load, indices developed from an evaluation of strain basins may provide a better explanation of the variation in performance for different pavement structures.

A detailed discussion of the development of the performance model is presented elsewhere (18). It was found that a hyperbolic equation adequately modeled the observed trends in flexible pavement performance at the AASHO Road Test. In the development of the model, pavement performance was defined to be the history of one or more pavement condition indicators over time or with increasing axle load applications. Pavement surface roughness, as quantified by slope variance (SV), was the pavement condition indicator selected for the model development.

The performance model shown in Table C-1 was evaluated by comparing observed versus predicted performance trends. Figure C-4 illustrates how the predictions from the model compare with the observed values for pavement roughness. The predictions generally compare favorably with the observed roughness data as reflected by the dark region around the line of equality. The root-mean-square (RMS) statistic for the performance predictions was found to be 0.24 with 5,895 observations. A

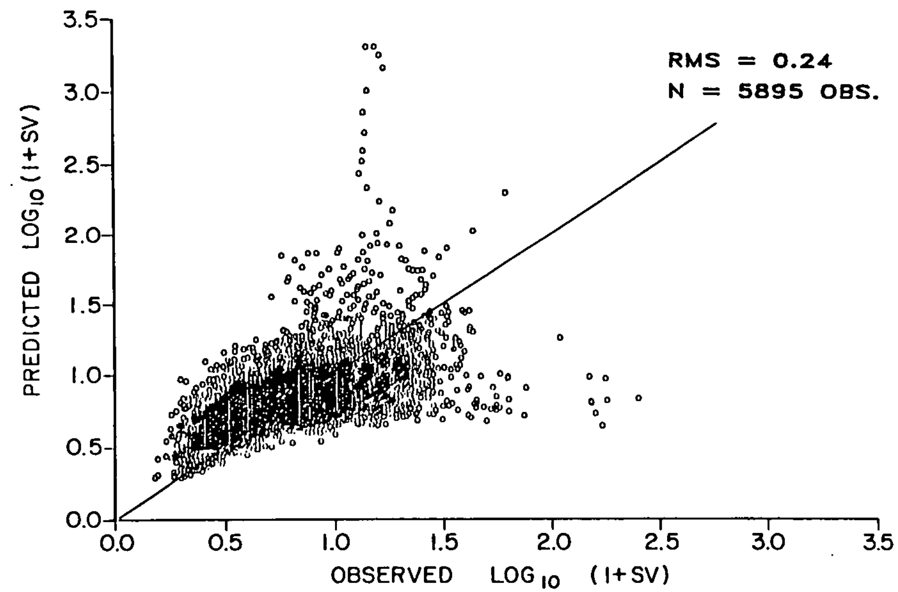

Figure C-4. Comparison of predicted $\log_{10}$ (1 + SV) from the hyperbolic model with the observed $\log_{10}$ (1 + SV).

similar statistic calculated from the observed performance data for the replicate sections at the AASHO Road Test was found to equal 0.19 with 767 observations. Replicate sections were identical pavement sections constructed at the AASHO Road Test. Thus the RMS statistic for the performance model compares favorably with the RMS statistic for the replicates, which gives a measure of the pure error in observed pavement performance.

In addition, the correlation coefficient between the predicted and observed $\log_{10}(1 + SV)$ was determined to be 0.59. In contrast, the correlation coefficient for the observed $\log_{10}(1 + SV)$ between replicates was found to equal 0.44. The fact that a higher correlation coefficient was obtained from the model predictions reflects the smoothing effect of the curve-fitting conducted as part of the model development. In addition, it further indicates that a performance model with reasonable predictive ability has been developed.

<u>SENSITIVITY ANALYSIS OF THE PERFORMANCE MODEL</u>

In order to evaluate the sensitivity of predicted performance from the model presented, a factorial experiment was established assuming the three layer pavement structure shown in Figure C-5. Eight different factors were considered in the development of the factorial experiment: 1) initial Present Serviceability Index (PSI_I); 2) asphalt concrete modulus; 3) asphalt concrete thickness; 4) granular base thickness; 5) coefficient (k_1) of the base resilient modulus-bulk stress relationship; 6) exponent (k_2) of the base resilient modulus-bulk stress relationship; 7) coefficient (m_1) of the subgrade resilient modulus-deviatoric stress relationship; and 8) exponent (m_2) of the subgrade resilient modulus-deviatoric stress relationship. The factors k_1, k_2, m_1, and m_2

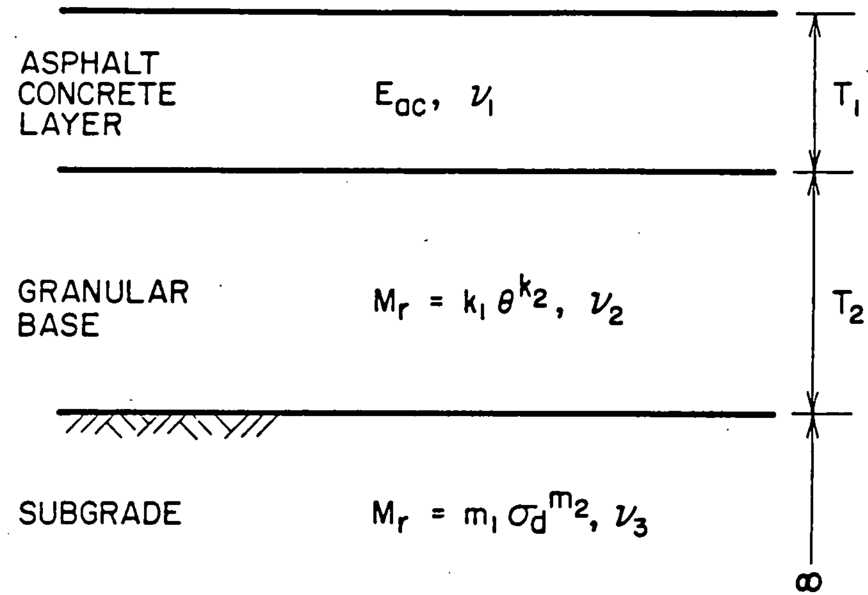

Figure C-5. Three-layer pavement model.

define the stress dependency of the resilient modulus of unbound pavement materials, as given by the following equations:

For granular materials: $M_r = k_1\theta^{k_2}$ (C-1)

For fine-grained soils: $M_r = m_1\sigma_d^{m_2}$ (C-2)

where

M_r = resilient modulus

θ = bulk stress (sum of principal stresses: $\sigma_1+\sigma_2+\sigma_3$)

σ_d = applied deviatoric stress ($\sigma_1 - \sigma_3$)

k_1, k_2, m_1, m_2 = experimental constants

Fixed values for the Poisson's ratios for the various layers, ν_1, ν_2, and ν_3, were assumed because the pavement response is not sensitive to changes in this variable. Specifically, Poisson's ratios of 0.30, 0.40, and 0.45 were assumed for the asphalt concrete, granular base, and subgrade layers, respectively.

Each factor included in the factorial experiment was varied over a range considered to be of interest for practical applications, and sufficiently wide to allow the given factor to demonstrate significant effects, if any, on predicted pavement performance. Table C-2 shows the levels established for the different factors in the factorial experiment. As may be seen from the table, three levels were selected for each factor, resulting in a ratio of 38, or 6,561 different pavement designs. Levels for the initial Present Serviceability Index were established using the following equation:

$$PSI = 4.96 - 2.01 \log_{10}(1 + SV) \quad \text{(C-3)}$$

$$R^2 = 0.80, \ N = 74 \text{ observations}$$

Table C-2. Levels of factors in the sensitivity analysis.

Factor	Levels	Units
1. Initial present serviceability index (PSI_i)	3.6; 3.9; 4.2	-
2. Asphalt concrete modulus (E_{ac})	300,000; 450,000; 600,000	psi
3. Asphalt concrete thickness (T_1)	3; 5; 7	inches
4. Granular Base thickness (T_2)	4; 7; 10	inches
5. Granular Base k_1	3000; 6000; 9000	-
6. Granular Base k_2	0.20; 0.50; 0.80	-
7. Subgrade m_1	10,000; 20,000; 30,000	-
8. Subgrade m_2	-1.00; -0.60; -0.20	-

where

PSI = Present Serviceability Index

SV = slope variance

The above equation was developed from the same data set used in the development of the AASHO PSI equation (C-2). In the determination of levels for PSI_i, the following assumed values for initial surface roughness [i.e., initial $\log_{10}(1 + SV)$] were used: 0.38, 0.53, and 0.68.

For each pavement design represented in the factorial experiment, the allowable number of 18-kip single-axle load applications was determined. An 18-kip single-axle load is commonly used as a reference load for design purposes. A terminal serviceability index of 1.5, corresponding to a final pavement surface roughness of 1.72, was used as the failure condition for predicting the allowable number of 18-kip single-axle load applications.

Multilayer linear elastic theory was used to calculate the appropriate strain basin indices for a given pavement design. An iterative application of linear elastic layer theory was conducted to obtain stress-compatible moduli. The same pavement response analysis procedure was used in the development of the performance model presented herein.

An equation relating the predicted allowable 18-kip single-axle load applications to the different factors considered in the study was determined through multiple linear regression using the model given below:

$$\begin{aligned} N_{18} = {} & \beta_0 + \sum_{i=1}^{8} \beta_i X_i + \sum_{i=1}^{8} \beta_{i+8}(3X_i^2 - 2) + \sum_{i=1}^{7} \sum_{j=i+1}^{8} \beta_{p(i,j)} X_i X_j \\ & + \sum_{i=1}^{7} \sum_{j=i+1}^{8} \beta_{q(i,j)}(3X_i^2 - 2)(3X_j^2 - 2) \\ & + \sum_{i=1}^{7} \sum_{j=i+1}^{8} \beta_{r(i,j)} X_i (3X_j^2 - 2) + \sum_{i=2}^{8} \sum_{j=1}^{i-1} \beta_{s(i,j)} X_i (3X_j^2 - 2) \end{aligned} \qquad \text{(C-4)}$$

where

N_{18} = predicted number of allowable 18-kip single-axle load applications

β = model parameter

X_i, X_j = pavement design factors

$p(i,j) = 8 + 7.5i - 0.5i^2 + j$

$q(i,j) = 36 + 7.5i - 0.5i^2 + j$

$r(i,j) = 64 + 7.5i - 0.5i^2 + j$

$s(i,j) = 101 - 1.5i + 0.5i^2 + j$

The functions $p(i,j)$, $q(i,j)$, $r(i,j)$ and $s(i,j)$ provide the appropriate subscripts for the β's for different values of the summation indices i and j. The eight different factors shown in Table C-2, and their two-way interactions were used as the independent variables, while the predicted logarithm (base 10) of the allowable 18-kip applications was used as the dependent variable. In order to evaluate the relative importance of each factor, standardized regression coefficients were determined by coding the levels of each factor shown in the table. Specifically, the low, middle, and high levels for each factor were coded as -1, 0, and +1 respectively. In addition, each main effect was decomposed into linear and quadratic components, while each interaction effect was decomposed into linear by linear, linear by quadratic, quadratic by linear, and quadratic by quadratic components. The quadratic effect is associated with the square of the level of a particular factor. In Equation C-4, the polynomial ($3X^2$ - 2) is used to generate orthogonal contrast coefficients for the evaluation of quadratic effects. Inasmuch as the low, middle and high levels of a particular factor have been coded as -1, 0, and +1 respectively, orthogonal contrast coefficients of +1, -2, and +1 are obtained from the polynomial ($3X^2$ -2). The use of orthogonal contrast coefficients in the regression analysis leads to model parameter estimates (β_i's) that do not vary when independent variables are added to or deleted from the model.

Using the eight pavement design factors from Table C-2 and their respective two-way interactions as independent variables in the regression analysis, a coefficient of determination (R^2) of 0.99 was obtained. Thus, most of the variation in the predicted allowable number of 18-kip applications was accounted for by the set of independent variables considered. In addition, approximately 90 percent of the total variation in the performance predictions was explained by the main effects. Table C-3 shows standardized model parameter estimates for the linear and quadratic components of main effects. By comparing the magnitudes of the parameter estimates, the relative importance of each factor can be evaluated. From the table, it can be seen that the linear effects are more important than the quadratic effects. In particular, the linear effects associated with the following factors are relatively important: 1) asphalt concrete thickness; 2) initial PSI; 3) asphalt

Table C-3. Standardized regression coefficients for the linear and quadratic components of main effects.

Factor	Standardized Regression Coefficient	
	Linear Component	Quadratic Component
1. Initial PSI (PSI_I)	0.414	-0.045
2. Asphalt concrete modulus (E_{ac})	0.306	-0.009
3. Asphalt concrete thickness (T_1)	0.568	0.016
4. Grandular base thickness (T_2)	0.095	0.015
5. Granular base k_1	0.055	0.004
6. Granular base k_2	0.109	0.030
7. Subgrade m_1	0.255	-0.019
8. Subgrade m_2	0.290	0.017

concrete modulus; and 4) the coefficients m_1 and m_2 that define the stress dependency of the resilient modulus of the subgrade.

In order to illustrate the relative importance of the different factors, each was varied from the low to the high level, while the other factors were fixed at one level (low, middle or high). Figures C-6, C-7, and C-8 show the effect of each of the eight factors on predicted pavement performance. The arrows in the boxes indicate whether the factor in question had a positive (pointing right) or negative (pointing left) effect on the predicted allowable number of 18-kip applications. The vertical line in each figure indicates the value for predicted performance when all variables are held at one level (low, middle or high). By adding to this value the calculated root-mean-square for the observed performance of AASHO replicate sections, the box labeled 'REP' has been constructed. The width of this box gives a measure of the unexplained variation in pavement performance, and thus provides a comparative value with which to evaluate the relative importance of the various pavement design factors. By comparing the widths of the boxes for the different factors with the width of the box for the replicates, the relative importance of each design factor, and the sensitivity of predicted performance to a particular factor can be evaluated.

From Figure C-6, it can be observed that at the low levels, predicted pavement performance is very sensitive to asphalt thickness, asphalt concrete modulus, initial PSI, and the parameters m_1 and m_2 that define the stress dependency of the subgrade resilient modulus. The effect of asphalt thickness is particularly important, and it can be inferred from Figure C-6 that for pavements constructed with weak materials on poor subgrade, performance can be significantly improved by

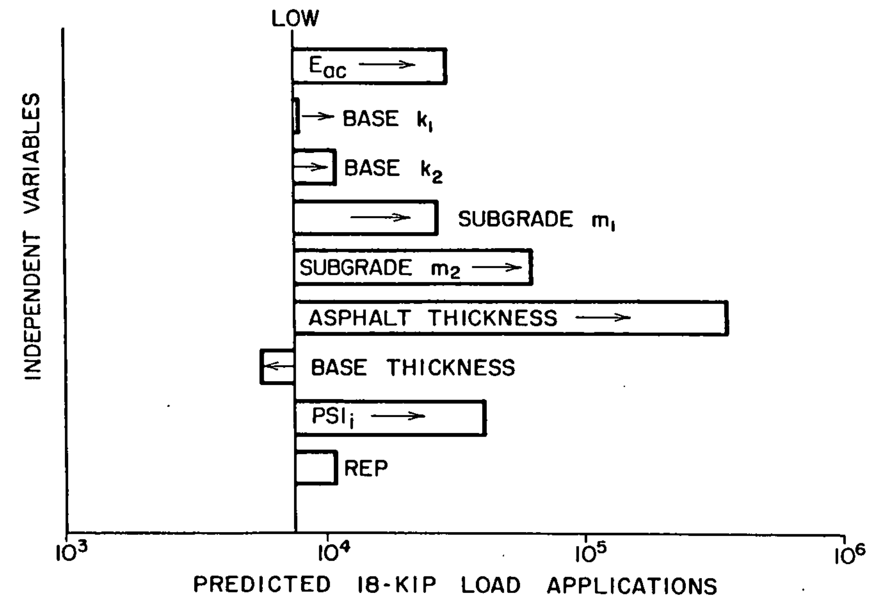

Figure C-6. Change in applications to failure when each factor is varied from low to high levels, with all other factors at low levels.

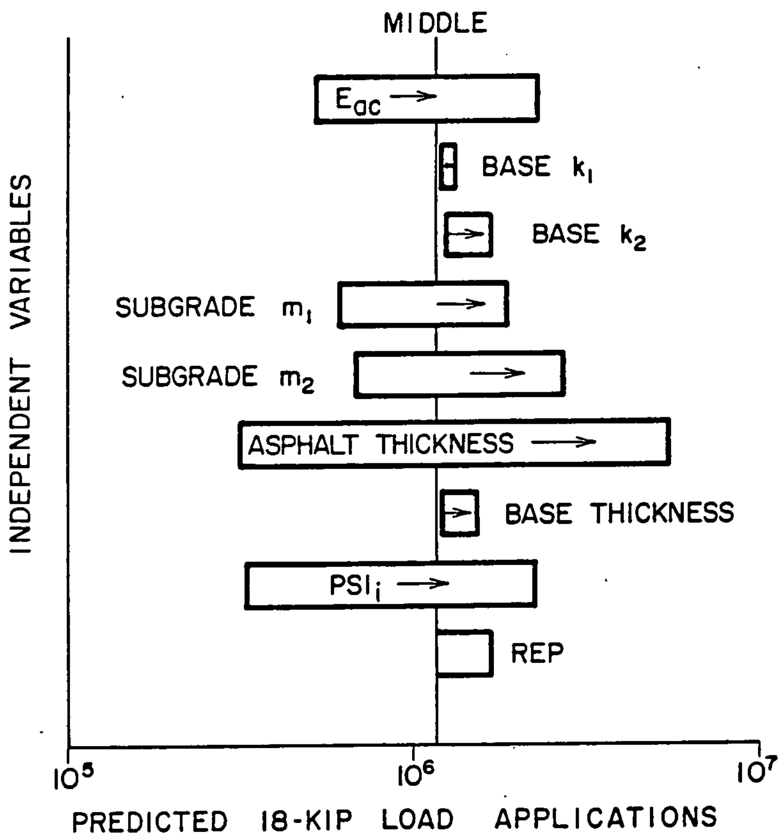

Figure C-7. Change in applications to failure when each factor is varied from low to high levels, with all other factors at middle levels.

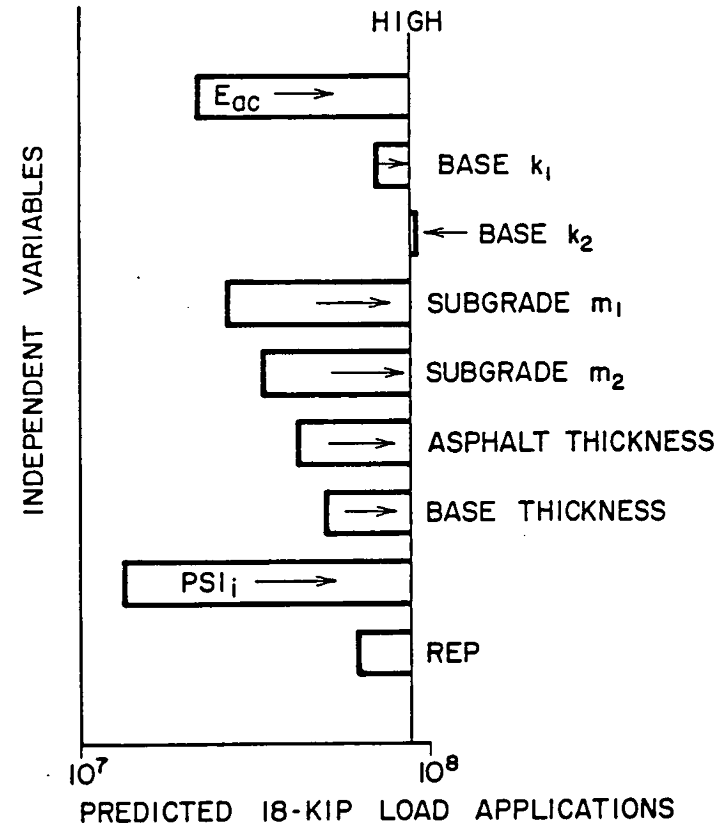

Figure C-8. Change in applications to failure when each factor is varied from low to high levels, with all other factors at high levels.

increasing the asphalt thickness. It can also be observed that the effects of base-related factors are relatively less important. The effect of the value of k_1 for the base, and the conditions considered, is relatively minor, especially when viewed in relation to the unexplained variation in pavement performance indicated by the 'REP' box. The effects of the value of k_2 for the base and base thickness are relatively larger in comparison to the effect of base k_1. However, the widths of these boxes are about the same as the width of the 'REP' box indicating that these factor effects are still less significant than those exhibited by factors associated with other pavement layers.

It is interesting to observe that for the conditions considered in Figure C-6, the increase in base thickness has a negative effect on predicted pavement performance. Increasing the base thickness from the low to the high level while keeping the other factors at their low levels led to a decrease in predicted service life. Although it can be argued that this decrease may not be significant when viewed in relation to the unexplained variation in pavement performance, it is still worthwhile to examine the various factors that might explain or justify the result obtained.

Table C-4 is a listing of 38 different effects for which the standardized regression coefficients are equal to or greater than 0.01. The effects have been ordered according to the absolute magnitudes of the regression coefficients. From the table, it can be seen that relative to the linear component of the base thickness effect T_2, the interactions between base thickness and base k_2, and between base thickness and base k_1, are significant. These interactions have standardized regression coefficients of 0.100 and 0.055, respectively,

Table C-4. Thirty-eight different effects sorted according to absolute magnitude of regression coefficient.

	Effect	Component	Standardized Regression Coefficient
1.	T_1	Linear	0.568
2.	PSI_i	Linear	0.414
3.	E_{ac}	Linear	0.306
4.	m_2	Linear	0.300
5.	m_1	Linear	0.255
6.	$k_2 * T_1$	Linear by Linear	-0.170
7.	$m_2 * T_1$	Linear by Linear	-0.125
8.	$E_{ac} * T_1$	Linear by Linear	0.118
9.	k_2	Linear	0.109
10.	$k_1 * k_2$	Linear by Linear	0.102
11.	$k_2 * T_2$	Linear by Linear	0.100
12.	T_2	Linear	0.095
13.	$k_1 * T_1$	Linear by Linear	-0.094
14.	$T_1 * T_2$	Linear by Linear	-0.075
15.	$m_1 * m_2$	Linear by Linear	0.055
16.	$k_1 * T_2$	Linear by Linear	0.055
17.	k_1	Linear	0.055
18.	PSI_i	Quadratic	-0.045
19.	$E_{ac} * k_2$	Linear by Linear	-0.042
20.	$m_1 * T_1$	Linear by Linear	-0.039
21.	$k_2 * m_2$	Linear by Linear	-0.033
22.	k_2	Quadratic	0.030
23.	$k_2 * m_1$	Linear by Linear	-0.024
24.	$E_{ac} * k_1$	Linear by Linear	-0.023
25.	$k_2 * T_1$	Linear by Quadratic	0.022
26.	$E_{ac} * m_2$	Linear by Linear	-0.021
27.	m_1	Quadratic	-0.019
28.	$k_1 * m_2$	Linear by Linear	-0.018

Table C-4. Thirty-eight different effects sorted according to absolute magnitude of regression coefficient (continued).

Effect		Component	Standardized Regression Coefficient
29.	m_2	Quadratic	0.017
30.	T_1	Quadratic	0.016
31.	$E_{ac} * T_2$	Linear by Linear	-0.016
32.	$k_1 * m_1$	Linear by Linear	-0.015
33.	T_2	Quadratic	0.015
34.	$k_2 * T_1$	Quadratic by Linear	-0.014
35.	$k_2 * T_2$	Quadratic by Linear	0.013
36.	$T_1 * T_2$	Quadratic by Linear	0.012
37.	$m_2 * T_1$	Linear by Quadratic	0.011
38.	$m_1 * T_2$	Linear by Linear	0.011

compared to a coefficient of 0.095 for the base thickness. Because low, middle, and high levels were coded as -1, 0, and +1, respectively, it can be seen that when the base thickness is at the high level (+1) and bases k_1 and k_2 are at the low levels (-1), each of the interactions between these variables and base thickness has a negative effect on predicted pavement performance (i.e., 0.095(+1) + 0.100(-1) + 0.055(-1) = -0.060). However, when all of these factors are at the high levels, a positive effect results. The practical implication of this finding is that in order to obtain any benefit to increasing base thickness, the factors k_1 and k_2 also have to be increased as a consequence of the stress dependency of the base resilient modulus. Other conditions being equal, an increase in base thickness could lead to a decrease in base modulus as a result of a reduction in bulk stress within the layer. Increasing the levels of k_1 and k_2 could help counteract this negative effect of base thickness on base resilient modulus.

At the middle levels, Figure C-7 shows that predicted service life is also very sensitive to asphalt thickness, initial PSI, asphalt concrete modulus, and the factors m_1 and m_2 defining the stress dependency of subgrade resilient modulus. In contrast, predicted service life is not as sensitive to the base-related factors, particularly when the effects of these factors are compared to the variation in the performance of AASHO replicate sections. It is interesting to note, however, that the boxes for the base related factors are to the right of the vertical line indicating the value of predicted service life when all factors are at the middle levels. This implies that for the conditions considered in Figure C-7, the middle level of each base-related factor is a point where predicted service life is a minimum. The occurrence of this condition again reflects the influence of the stress dependency of unbound pavement materials. Because the base resilient modulus is stress stiffening, whereas the subgrade resilient modulus is stress softening, conditions at which predicted performance is a maximum can exist.

At the high levels, Figure C-8 shows that predicted service life is influenced significantly by: 1) initial PSI; 2) asphalt concrete modulus; 3) the factors m_1 and m_2 defining the stress dependency of subgrade resilient modulus; 4) asphalt concrete thickness; and 5) base thickness. The effect of initial PSI is particularly important, and one can infer from the results that even if a pavement was constructed with good materials, and with thick layers, if the initial riding quality was poor, then the predicted pavement service life will be significantly less than if the initial riding quality was good. This observation warrants the inclusion of roughness as a pay factor in any performance-related M&C specification. One can also infer that two pavements with substantially different levels of initial surface roughness will yield different service lives even though the two pavements may have identical layer thicknesses and material properties. The difference in service lives may be explained by the effect of pavement surface roughness on the magnitudes of axle loadings that are applied to the pavement.

The base thickness effect shown in Figure C-8 is also consistent with an earlier observation that the levels of the base factors k_1 and k_2 must be increased if an increase in base thickness is to have a positive effect on predicted pavement performance.

EVALUATION OF THE EFFECTS OF INTERACTIONS ON PREDICTED PERFORMANCE

To further understand how predicted pavement performance is affected by the various factors considered in the sensitivity analysis, it is important to evaluate how these factors jointly affect the performance predictions. In view of the significant influence of the stress dependency of the resilient modulus of unbound pavement materials, it can be expected that predicted pavement performance will be significantly affected by some of the two-way factor interactions considered.

Figure C-9 illustrates the interaction between base k_2 and asphalt concrete thickness T_1. The low, middle, and high levels of asphalt concrete thickness are represented by the circle, cross, and diamond symbols, respectively. Solid lines, short dashed lines, and long dashed lines used to connect the different symbols represent conditions where pavement design factors other than base k_2 and T_1 are held at the low, middle, and high levels, respectively.

From Figure C-9 it can be observed that predicted pavement performance is significantly affected by asphalt concrete thickness. For any given level of base k_2, a thicker asphalt generally leads to a longer predicted service life. The effect of base k_2 is not very noticeable at the low levels of pavement design factors other than base k_2 and T_1. This is evident from the flatness of the solid lines. For these conditions, therefore, it can be inferred that increasing the asphalt concrete thickness is the best alternative to improving the predicted pavement performance. At the middle and high levels however, increasing base k_2 does have a positive effect on predicted performance. In particular, the effect of improving base k_2 is most

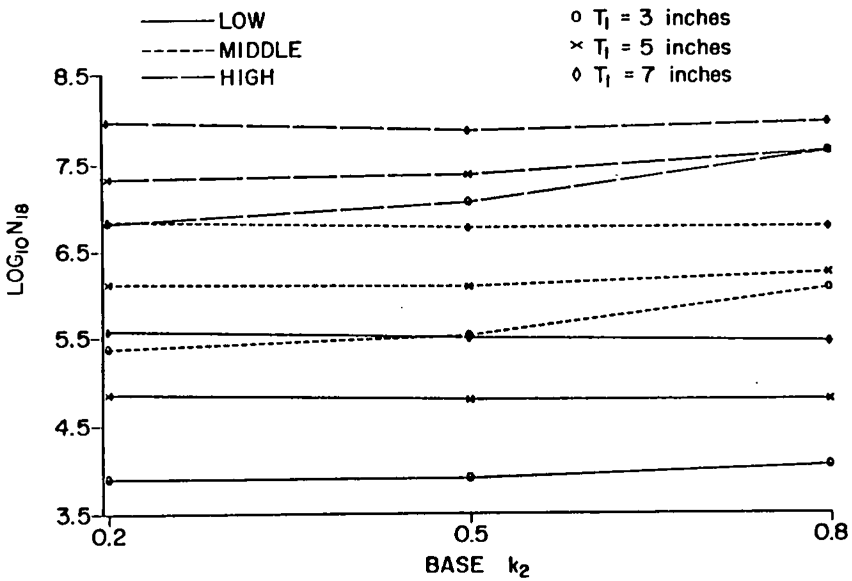

Figure C-9. Effect of base k_2 on predicted service life for different levels of asphalt concrete thickness, T_1.

significant at the low level of asphalt concrete thickness. At the high level of this particular variable, base k_2 has little effect on predicted performance. These observations again reflect the influence of the stress dependency of the resilient modulus of unbound pavement materials. At the high level of T_1, the bulk stress within the base layer would be relatively lower than that corresponding to the low level of T_1. In particular, it is possible that the bulk stress may not be sufficient to mobilize the base stiffness. Consequently, for pavements with thick asphalt layers, improving base k_2 may not have any significant beneficial effect on predicted pavement performance.

Figure C-10 illustrates the interaction between subgrade m_2 and asphalt concrete thickness. As may be expected, increasing the values of these two variables generally leads to improvements in predicted performance. This may be explained by considering that for subgrade soils, the resilient modulus varies inversely with the deviatoric stress within the layer. Consequently, constructing a thicker asphalt surface would tend to have a beneficial effect on the subgrade resilient modulus by lowering the deviatoric stress. Similarly, improving the quality of the subgrade soil by increasing the value of m_2 would have a positive effect on the subgrade resilient modulus, and consequently on pavement performance.

It is interesting to note, however, that the effect of increasing subgrade m_2 is most significant at the low level of asphalt concrete thickness. From Figure C-10, it may be observed that when factors other than subgrade m_2 and T_1 are held at the low and middle levels, the lines corresponding to a 3-in asphalt concrete thickness are relatively steeper than the lines for the other levels of this particular variable.

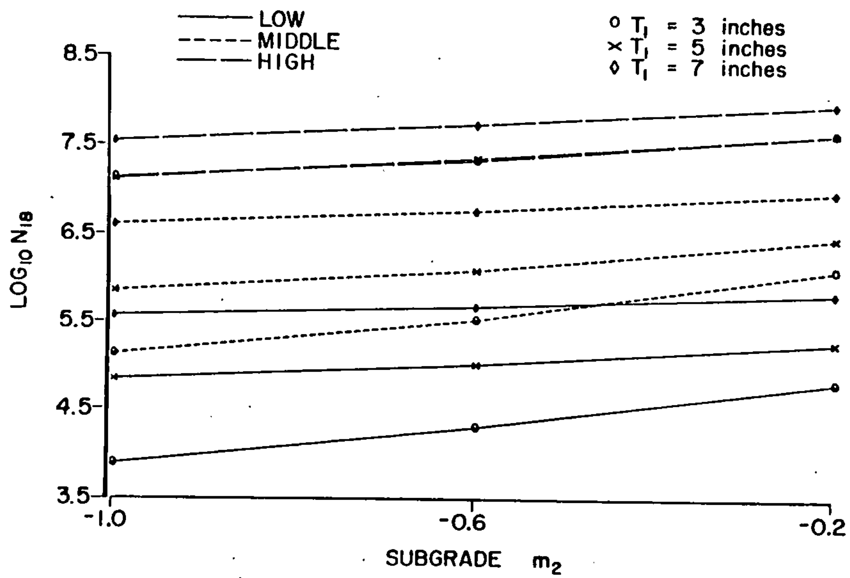

Figure C-10. Effect of subgrade m_2 on predicted service life for different levels of asphalt concrete thickness, T_1.

In addition, at the high levels, there are no significant differences between the performance predictions for 3- and 5-in-thick asphalt layers. These observations again reflect the influence of the stress dependency of the base resilient modulus.

The interaction between asphalt concrete modulus and asphalt thickness is shown in Figure C-11. It is observed that predicted service life increases with increasing asphalt modulus and asphalt thickness. The beneficial effect of asphalt thickness on pavement performance is most significant when factors other than E_{ac} and T_1 are held at the low levels. This is apparent from Figure C-11, where it is observed that the solid line for a 7-in-thick asphalt layer overlaps short dashed lines representing predictions when factors other than E_{ac} and T_1 are held at the middle levels. At the high levels, the effect of increasing asphalt concrete thickness is not as significant as it is at the low and middle levels. Thus for pavements constructed with good base and subgrade layers, and with high values of initial PSI, one can infer that increasing asphalt concrete thickness would not yield as much benefit, in terms of percent improvement in predicted service life, as it would for pavements with weak base and subgrade layers.

FINDINGS

The use of a sensitivity analysis to evaluate a pavement performance model has been demonstrated in this appendix. Based on the results of the analysis the following findings are valid:

1. Predictions of service life from the model evaluated were found to be sensitive to asphalt concrete thickness, initial PSI, asphalt concrete modulus, and the coefficients m_1 and m_2 defining the stress dependency of the resilient modulus of the

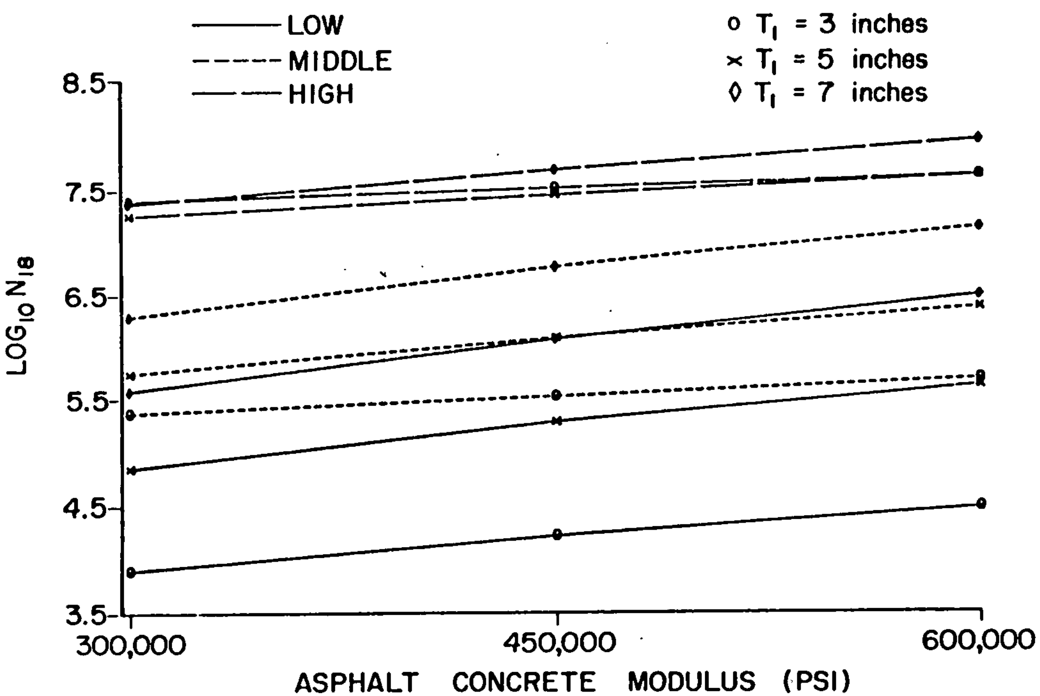

Figure C-11. Effect of asphalt concrete modulus on predicted service life for different levels of asphalt concrete thickness, T_1.

subgrade soil. The effect of initial surface roughness on predicted service life was shown to be significant. This implies that two pavements with identical layer thicknesses and material properties will yield different service lives if the levels of initial surface roughness are substantially different. Consequently, surface roughness is a valid M&C variable in a performance-related specification.

2. In general, if other factors are held constant, predicted service life improves with increases in the levels of the following factors: a) asphalt concrete thickness, b) initial PSI, c) asphalt concrete modulus, d) subgrade m_1, and e) subgrade m_2. However, the amount of improvement in predicted service life is dependent on the levels at which the other factors are held constant.

3. In general, the effects of base-related variables (i.e., base thickness, base k_1, and base k_2) depend on the levels of the other pavement design factors considered. However, the effects of base-related variables are relatively small compared to the effects of the other design factors, and to the unexplained variation in pavement performance.

4. Because of the influence of the stress dependency of unbound pavement materials, there is strong indication that optimum values for base-related variables exist for different pavement conditions.

CONCLUSIONS

The following conclusions may be drawn from the information presented:

1. When designing pavements using the performance model evaluated in this study, the consideration of the stress sensitivity of unbound layers in the pavement is important.
2. A sensitivity analysis is an important tool for evaluating the behavior of a performance model over a range of conditions considered to be of practical interest. In the application of the performance model presented herein, it is important to use the model consistent with the results of the sensitivity analysis conducted.

This analysis identified pavement design factors which significantly influence performance predictions. Some of these variables, such as the values of m_1 and m_2 for the subgrade, would be assumed for a particular job and would remain fixed in the design and specification process. Other variables, such as initial PSI and asphalt concrete thickness, are valid M&C variables. The modulus of the asphalt concrete may be an M&C variable if it is measured directly or it may be predicted by a model, such as Equation 4, that includes other M&C variables such as asphalt content and percent air voids.

APPENDIX D—EFFECT OF VARIABILITY OF DESIGN VARIABLES ON PREDICTED PAVEMENT PERFORMANCE

This appendix presents an illustrative example, in which a portion of the conceptual framework is used to show the basic principles involved in analyzing the components of pavement performance variability. This example involves an analysis of the effects of variation in design variables on the overall variation of predicted pavement performance. The variability of pavement performance is then related to life-cycle costs so that the effect of design variables on life-cycle cost can be evaluated.

It must be emphasized that the illustrative example is used only to demonstrate concepts and cannot be used to develop general conclusions, since only one pavement design is considered, under a single set of traffic and environmental conditions, using only one performance prediction model.

METHODOLOGY FOR CONSIDERING FACTORS AFFECTING PAVEMENT PERFORMANCE

The example analysis assumed a three-layer linear elastic model of the pavement structure. The design factors considered were the moduli of the surface, base, and subgrade layers; the thicknesses of the surface and base layers; and the expected design traffic expressed in terms of 18-kip equivalent single-axle loads (ESAL's). To evaluate the effect of variations in the design factors on predicted pavement performance, factorial designs were established where the levels selected for the various factors represented deviations from the values intended for those factors by the pavement design engineer. This analysis, therefore, investigated the effect of variation (tolerance) of the design factors within one design, and not the variation among different designs.

Two cases were considered. In one case, the effect of introducing variations in the values of the asphalt surface modulus and surface thickness was evaluated. For this case, it was assumed that the design, or target, values for the layer moduli and thicknesses were:

Asphalt concrete modulus	800,000 lb/in^2
Granular base modulus	50,000 lb/in^2
Subgrade modulus	5,000 lb/in^2
Surface thickness	7.5 in
Base thickness	6.0 in

Coefficients of variation of 5, 10, and 15 percent were assumed in establishing the deviations from the design value of the asphalt concrete modulus. These figures are considered representative of those found in practice and represent standard deviations of 40,000, 80,000, and 120,000 lb/in^2, respectively. For each value of the coefficient of variation, the asphalt concrete modulus was varied from -2 to +2 standard deviations of the design value, in increments of one standard deviation. For the case considered, three factorial designs were established as shown in Table D-1. In all three designs, the surface thickness was varied from 0.25 in less than the design value to 0.25 in more.

In the second case considered, deviations in the design values of all layer moduli and thicknesses were introduced. For this case, the assumed design values for these factors were:

Asphalt concrete modulus	500,000 lb/in^2
Base modulus	50,000 lb/in^2
Subgrade modulus	5,000 lb/in^2
Surface thickness	6.0 in
Base thickness	9.0 in

Table D-1. Levels of factors assumed for studying effects of variations in asphalt concrete modulus and surface thickness.

Variable	Levels	Units
Asphalt concrete modulus		
a) C.V.* = 5%	720, 760, 800, 840, 880	x 10^3 psi
b) C.V. = 10%	640, 720, 800, 880, 960	x 10^3 psi
c) C.V. = 15%	560, 680, 800, 920, 1040	x 10^3 psi
Surface thickness	7.25, 7.50, 7.75	inches
Base modulus	50,000	psi
Base thickness	6.0	inches
Subgrade modulus	5,000	psi

*C.V. = coefficient of variation

A coefficient of variation of 20 percent was used to establish the deviations from the design values of the layer moduli. Each layer modulus was varied from 80 percent of the design value to 120 percent. As in the previous case, deviations of ± 0.25 in from the design value of the surface thickness were assumed, while deviations of ± 0.50 in were used for the base thickness. The resulting factorial design is shown in Table D-2.

For each combination of layer modulus and thickness included in the factorial designs, BISAR, the computer program developed by Shell of The Netherlands (73), was used to calculate the theoretical compressive strain at the top of the subgrade resulting from an 18-kip single-axle loading. The Simplified Rational Pavement Design (SRPD) performance equation developed by Luhr (86) was then used with the calculated strain values to predict the number of allowable 18-kip single-axle load applications for each pavement condition included in the factorial designs. The SRPD performance equation was developed from AASHO Road Test data and is expressed as:

$$\log_{10} N_X = 2.15122 - 597.662\,(\epsilon_{sg}) - 1.32967\,\log_{10}(\epsilon_{sg}) + \log_{10}[PSI_i - TSI)/2.7]^{1/2} \qquad \text{(D-1)}$$

where

N_x = number of weighted applications of axle load X before the pavement reaches a specified terminal serviceability index (TSI)

ϵ_{sg} = subgrade compressive strain due to axle load X

PSI_i = initial present serviceability index

In the prediction of pavement performance using the SRPD model, an initial PSI of 4.2 and a terminal serviceability index of 1.5 were assumed. Because of the deviations from the target values of the design factors considered, a distribution of the predicted number of allowable axle load

Table D-2. Levels of factors assumed for studying effects of variations in all layer moduli and thicknesses.

Variable	Levels	Units
Asphalt concrete modulus	400, 500, 600	x 10^3 psi
Surface thickness	5.75, 6.00, 6.25	inches
Base modulus	40, 50, 60	x 10^3 psi
Base thickness	8.50, 9.00, 9.50	inches
Subgrade modulus	4, 5, 6	x 10^3 psi

applications exists for each factorial design established. The distribution of performance prediction estimates from each factorial design was quantified using probability theory and statistical techniques. Table D-3 summarizes the main steps involved in the evaluation of the effect of variability in design variables on predicted pavement performance. The error in the performance prediction equation was considered when the distribution of the predicted number of axle load applications to failure was determined. In this way, the spread in the distributions obtained reflected the deviations from design, the lack of fit, and "pure" error in the SRPD performance model.

By estimating the expected number of axle load applications per year during the design life of the pavement, the distribution of the number of allowable axle load applications can be converted to a distribution of the number of years to failure. In order to consider the error in traffic predictions, a traffic multiplier or a traffic adjustment factor is introduced. This variable reflects the difference between predicted and actual traffic and is assumed to follow a certain statistical distribution. For the example presented herein, this distribution is assumed to be lognormal, with an expected value of 1 and a variance σ^2_m. The distribution of performance estimates, expressed in terms of the number of allowable 18-kip single-axle load applications, can subsequently be converted to a distribution of the number of years to failure that reflects the variability in design and the uncertainties in the prediction of performance and design traffic. The results of the analysis conducted are presented below.

DISTRIBUTION OF PERFORMANCE ESTIMATES FROM SRPD MODEL

In functional form, the true number of allowable axle load applications to failure, for any particular pavement design, is given by:

$$N_i(\tilde{X}) = g_i(\tilde{X}) + \epsilon_i \qquad \text{(D-2)}$$

Table D-3. Steps for evaluating effect of variability in design variables on predicted pavement performance.

A. Identify design factors to be considered.

B. Evaluate the variability in design factors.

   1. Evaluate existing plant production capabilities.

   2. Evaluate existing road construction capabilities.

C. Establish a factorial experiment where levels of the various factors represent possible deviations from the pavement design selected.

D. Establish a model for predicting performance.

E. Estimate the number of axle load applications to failure for each pavement design represented in the factorial experiment.

F. Evaluate the distribution of the number of axle load applications to failure.

   1. Establish probabilities associated with each pavement design in the factorial experiment.

   2. Evaluate the uncertainty in the performance prediction equation--establish the distribution of the errors associated with the performance model.

   3. Establish the distribution of performance estimates taking into account the error in the performance model, using the information from Steps F.1 and F.2.

G. Estimate the number of axle load applications per year during the design period.

H. Transform the distribution of the number of allowable axle load applications to a distribution of the number of years to failure, using the predicted traffic rate from Step G.

I. Evaluate the variability in traffic forecasts--estimate the distribution of a traffic multiplier W (i.e., traffic adjustment factor) reflecting the error in predicted traffic.

J. Adjust the information in Step I, using the distribution of the number of years to failure established in Step H, to consider the variability in traffic predictions.

K. Evaluate the effects of the variability in design, and the uncertainties in the prediction of performance and traffic, on the life-cycle costs.

where

$N_i(\tilde{X})$ = true number of allowable axle load applications for pavement design i, represented by the vector of design variables $\tilde{X}$

$g_i(\tilde{X})$ = predicted number of allowable axle load applications from the performance model for pavement design i

ϵ_i = error in the prediction of the number of allowable axle load applications for pavement design i, assumed to have a certain frequency distribution $f_y(y)$

If all possible combinations of deviations in the design variables are considered, the probability that the number of allowable axle load applications, N', is less than or equal to some particular value $\tilde{N}$ of $N_i(X)$ is given by:

$$P_r\,[N' \leq N] = \sum_{i=1}^{n} P_r\,(\tilde{X})_i\, F_y\,[N - g_i(\tilde{X})] \qquad \text{(D-3)}$$

where

$P_r[N' \leq N]$ = probability of N' being less than or equal to some value of N of $N_i(\tilde{X})$

$P_r(\tilde{X})_i$ = probability that a certain pavement, defined by a particular vector of design variables $(X)_i$, will be constructed

$F_y(N - g_i(X))$ = value of the cumulative frequency distribution for ϵ_i evaluated at $[N - g_i\,(\tilde{X})]$

For the example presented, all of the pavement structures represented in each factorial design established were assumed to be equally likely to occur. Consequently, $Pr(\tilde{X})_i$ is equal to 1/n, where n is the number of observations in each factorial design. By differentiating Equation D-3, a relationship for numerically evaluating the frequency distribution of $N_i(X)$ is obtained. This relationship is given by:

$$F_N(N) = 1/n \sum_{i=1}^{n} f_y\,[N - g_i\,(\tilde{X})] \qquad \text{(D-4)}$$

where

$f_N(N)$ = value of the frequency distribution for the number of allowable axle load applications when $N_i(\tilde{X}) = N$

n = number of observations in the factorial design considered

$f_y[N - g_i(X)]$ = value of the frequency distribution for ϵ_i evaluated at $[N - g_i\,(X)]$

In the analysis, the errors ω_i in the prediction equation were assumed to be distributed normally with a mean of zero and a variance of $\sigma^2 y$. Thus, for any given value of N, fy $(N-g_i(X))$ can be evaluated for any pavement structure represented in the established factorial designs. By assuming a sufficient number of N's, the distribution of the performance estimates can be evaluated numerically using Equation D-4.

Figures D-1 through D-4 illustrate density curves obtained for all of the factorial designs established for this particular study. Figures D-1 through D-3 refer to case I, where only the surface modulus and thickness are varied; Figure D-4 refers to case II, where all pavement design factors are varied. In the analysis, two different values for the standard deviation of the error distribution were assumed, to illustrate the effect of the error in the performance model. These were 0.266, the standard error of the estimate (SEE) of the SRPD equation, and 0.100, an estimate of the variation in the observed performance of replicate pavement sections at the AASHO Road Test. As shown by the figures, a reduction in the error of the performance model

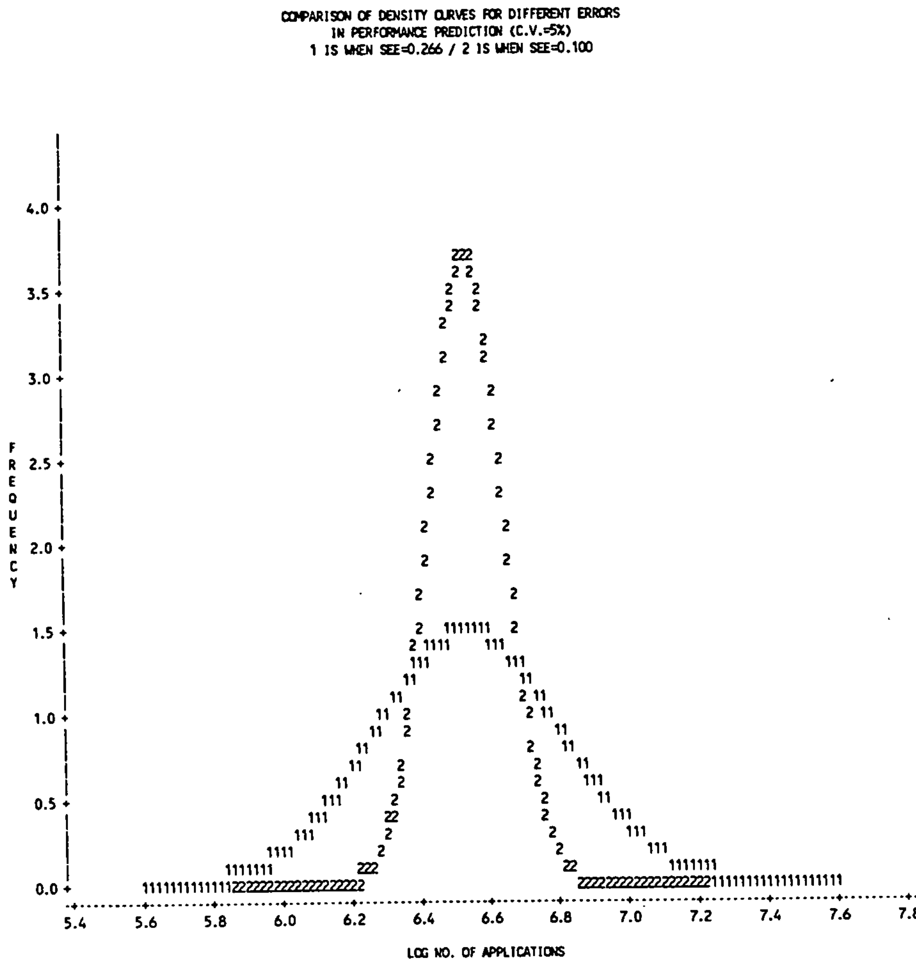

Figure D-1. Comparison of density curves for different errors in performance prediction (c.v. = 5%).

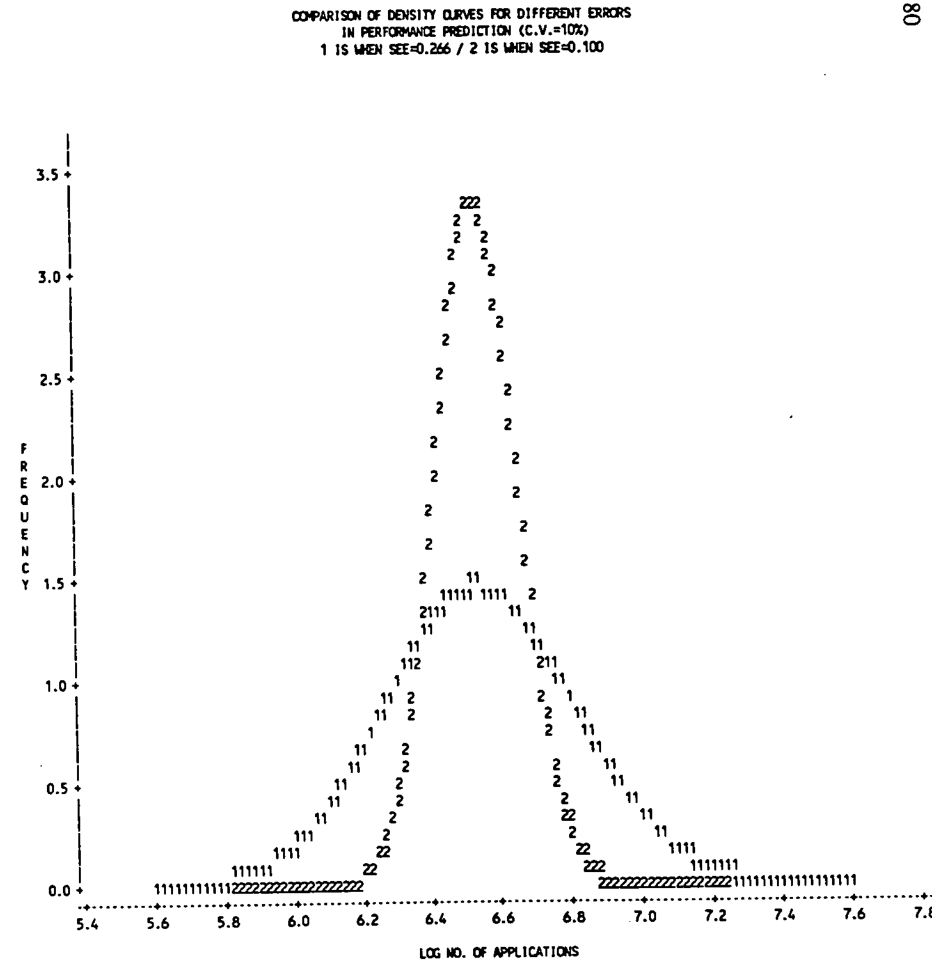

Figure D-2. Comparison of density curves for different errors in performance prediction (c.v. = 10%).

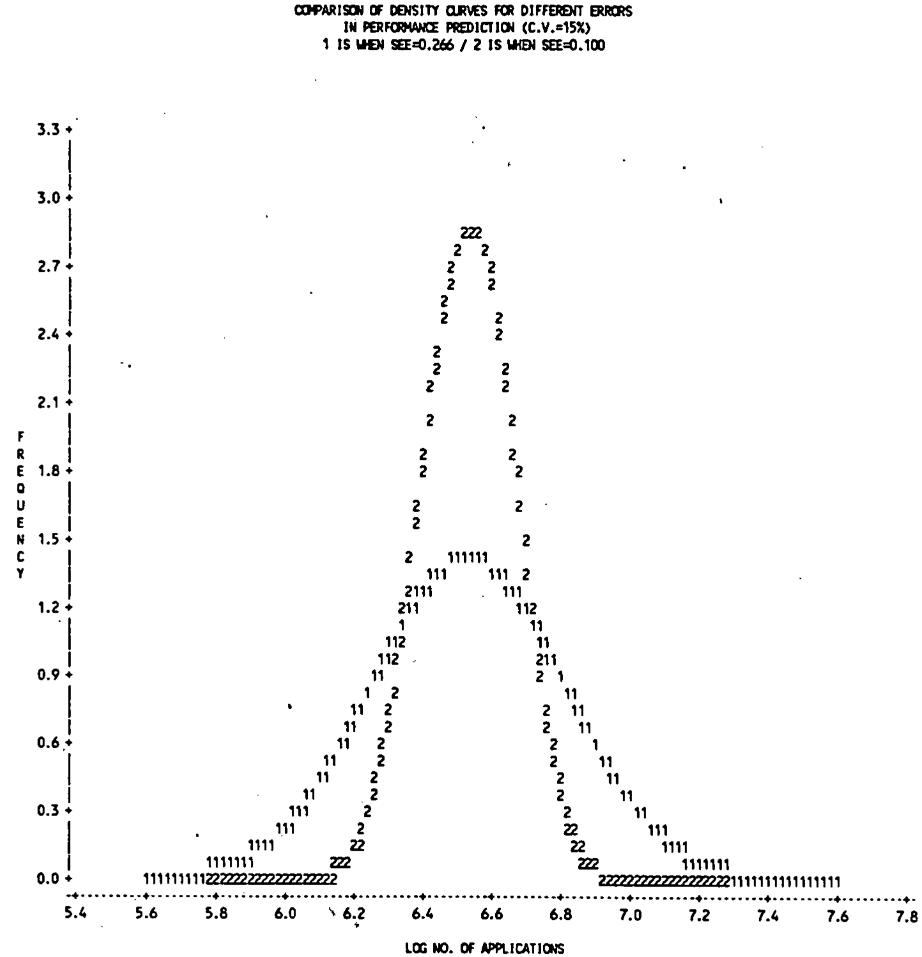

Figure D-3. Comparison of density curves for different errors in performance prediction (c.v. = 15%).

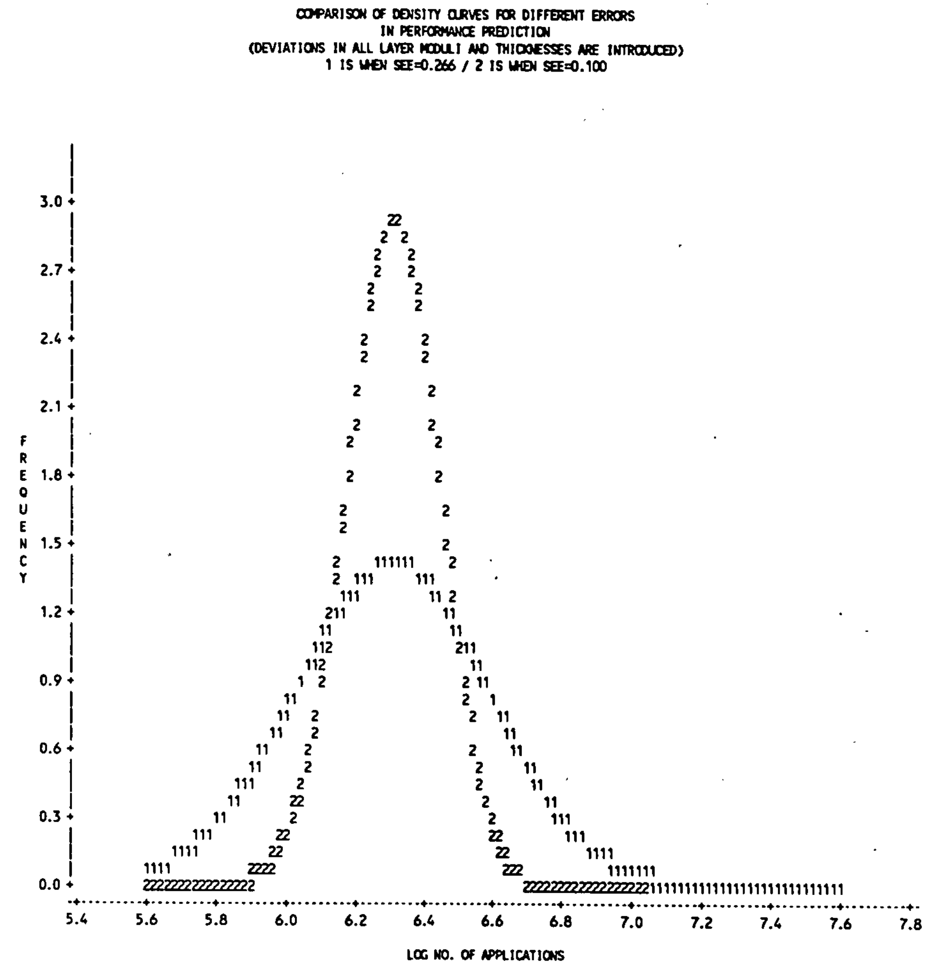

Figure D-4. Comparison of density curves for different errors in performance prediction (deviations in all layer moduli and thicknesses are introduced).

results in a density curve that is narrower and more peaked, as would be expected. The density curves obtained also appear to be normal, with each curve having a mean equal to the predicted number of applications to failure when the layer moduli and thicknesses are at the design levels.

To determine the density function for each of the curves obtained, a nonlinear model was assumed, and the parameters of the model were determined using nonlinear regression. Since the curves appear to be normal, a suitable candidate for a model is the normal density function with parameters μ_N and σ_N (mean and standard deviation, respectively). The results of the nonlinear regression analysis show that this model fits each of the curves in Figures D-1 through D-4 extremely well, further indicating the normality in the curves. Table D-4 summarizes the parameters of the normal density functions for each of the curves examined. It is interesting that for the case where deviations in asphalt concrete modulus and surface thickness are introduced, the differences in the standard deviations for different coefficients of variation are less for $\sigma_y = 0.266$ than for $\sigma_y = 0.100$. Figures D-5 and D-6 illustrate this point. Even though there are differences in the amount of variability introduced, it is possible for the error in the performance prediction equation to dominate and to mask the effect of deviations from design. This observation underscores the importance of the accuracy of pavement performance models.

DISTRIBUTION OF THE PREDICTED NUMBER OF YEARS UNTIL FAILURE

For any given value of the number of load applications to failure, the predicted number of years to failure can be determined by dividing the number of load applications by the traffic rate expected during the design life of the pavement. In the preceding section, it was indicated that the distribution of the logarithms of the number of load applications to failure

Table D-4. Parameters of the normal distribution for the logarithms (base 10) of the number of allowable load applications.

Case I. Deviations in surface modulus and thickness are introduced.

Coefficient of Variation for Surface Modulus	σ_y	μ_N	σ_N
5%	0.100	6.53	0.1080
10%	0.100	6.53	0.1188
15%	0.100	6.53	0.1370
5%	0.266	6.53	0.2691
10%	0.266	6.53	0.2733
15%	0.266	6.53	0.2806

Case II. Deviations in all layer moduli and thicknesses are introduced.

σ_y	μ_N	σ_N
0.100	6.30	0.1360
0.266	6.30	0.2812

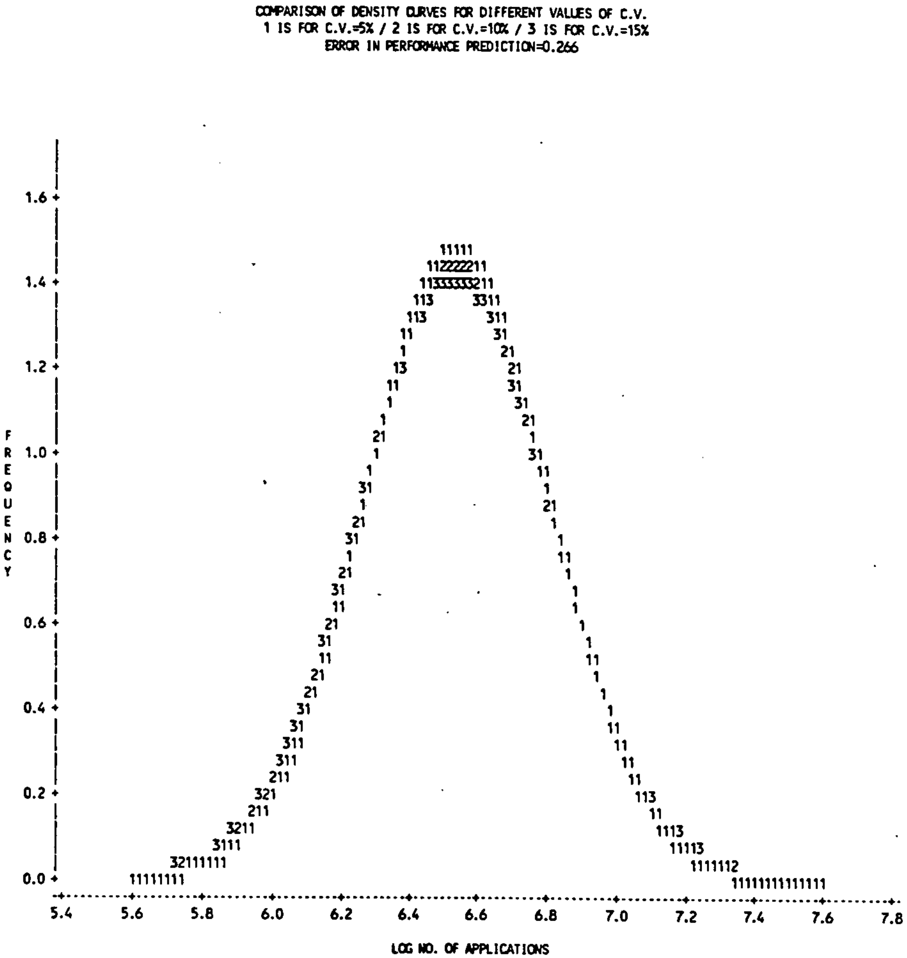

Figure D-5. Comparison of density curves for different values of coefficient of variation (error in performance prediction = 0.266).

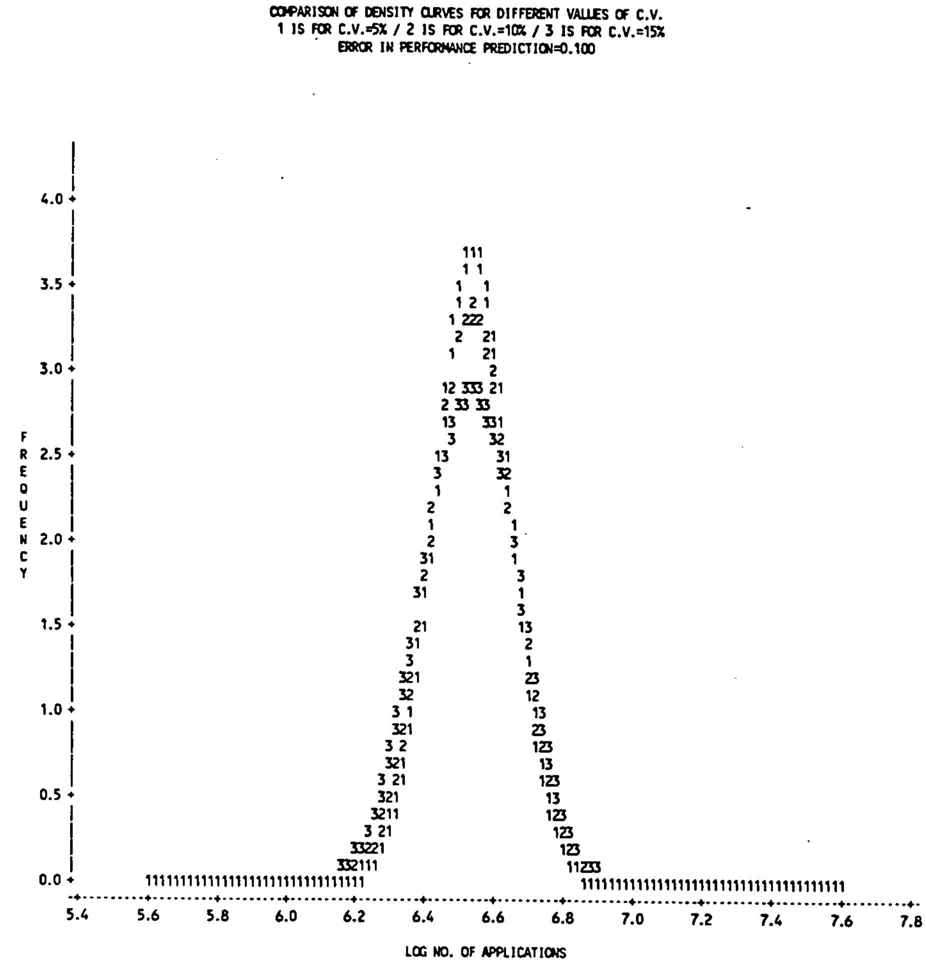

Figure D-6. Comparison of density curves for different values of coefficient of variation (error in performance prediction = 0.100).

is normal with some mean μ_N and variance σ_N^2. Consequently, the distribution of the actual number of load applications to failure should be lognormal, with parameters a_p, μ_p, and σ_p.

For the example being discussed, $a_p = 0$ and μ_p and σ_p are determined by multiplying μ_N and σ_N by $n = 10$, since logarithms of the number of load applications to failure were determined using base 10. Figure D-7 shows distributions of the actual number of load applications to failure for the case where deviations in all layer moduli and thicknesses are introduced (case II). From the values of μ_N and σ_N given in Table D-4, the parameters of the distributions shown can be calculated as follows:

For $\sigma_y = 0.266$:

$$\mu_p = n\ 10\ \mu_N = (n = 10)\ 6.30 = 14.51$$

$$\sigma_p = n\ 10\ \sigma_N = (n = 10)\ 0.2812 = 0.6475$$

For $\sigma_y = 0.100$:

$$\mu_p = n\ 10\ \mu_N = (n = 10)\ 6.30 = 14.51$$

$$\sigma_p = n\ 10\ \sigma_N = (n = 10)\ 0.1360 = 0.3132$$

If each of the distributions shown in Figure D-7 is transformed through division by a constant, R, the resulting distributions are also lognormal with parameters (μ_p - n R) and σ_p. If the constant, R, is the predicted traffic rate during the design life of the pavement, then the distribution of the number of years to failure is lognormal with the parameters as given previously. This, of course, assumes that there is no uncertainty in the predicted value of R, which does not happen in practice.

To illustrate the effect of variability in predicted traffic, a distribution for a multiplier to the traffic rate is assumed. This distribution can be defined by examining historical traffic records and

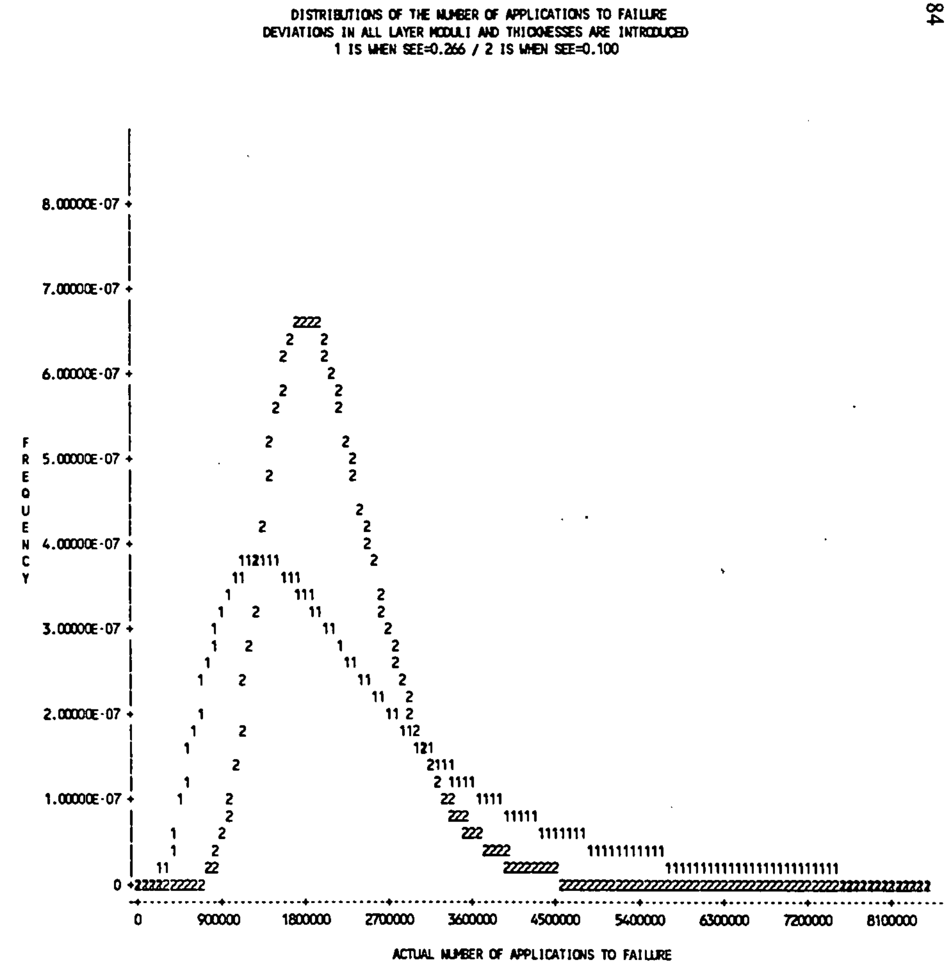

Figure D-7. Distributions of the number of applications to failure (deviations in all layer moduli and thicknesses are introduced).

determining ratios of observed versus predicted traffic rates. This distribution is unknown, but it may be quantified in future research studies from carefully maintained traffic records. For the purposes of this illustrative example, therefore, assumptions are made only about the distribution of the traffic multiplier, W. Specifically, the distribution is assumed to be lognormal with parameters a_W, μ_W, and σ_W. Hereafter, the notation $X \sim LN(a, \mu, \sigma)$ will indicate that a random variable X has a lognormal distribution with parameters a, μ, and σ. The parameters a and σ are called location and shape parameters, respectively; μ is used to define a scale parameter $b = \exp(\mu)$ (45).

For this example, a_W is assumed to be zero, and the other parameters, μ_W and σ_W, are calculated from assumptions about the expected value and variance of W. These quantities are calculated from the following equations (44):

$$E(W) = a_W + b_W \exp(\sigma^2_W/2) \qquad \text{(D-5)}$$

$$\mathrm{var}(W) = B^2_W \exp(\sigma^2_W)\,[\exp(\sigma^2_W) - 1] \qquad \text{(D-6)}$$

where

E(W) = expected value of W

var(W) = variance of W

a_W = location parameter (assumed to be zero in this example)

b_W = scale parameter

σ_W = shape parameter

In this example, E(W) is assumed to be 1, and two values for var(W) are assumed, to illustrate the effect of improvements in the accuracy of traffic predictions. Specifically, values of 0.04 and 0.25 are assumed, leading to the two distributions for W illustrated in Figure D-8. These distributions

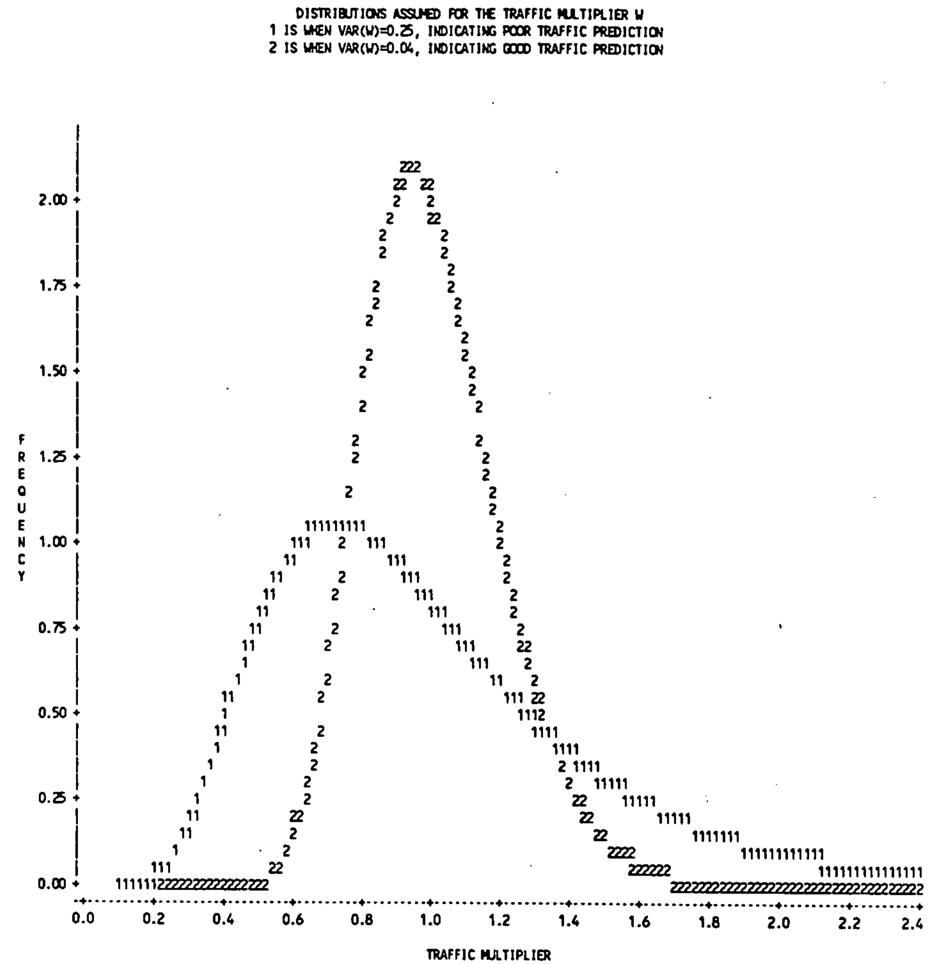

Figure D-8. Distributions assumed for the traffic multiplier, W.

are defined by the parameters $\mu_W = 0.1116$ and $\sigma_W = 0.4724$ for the case where $var(W) = 0.25$ and by the parameters $\mu_W = -0.0196$ and $\sigma_W = 0.1980$ for the case where $var(W) = 0.04$. These values were calculated from Equations D-5 and D-6, using the assumptions made for a_W, $E(W)$, and $var(W)$.

As expected, a lower value for var(W) results in a narrower and more peaked distribution. By definition, the distribution of the logarithms of the traffic multiplier should be normal, with mean $\mu_{LM} = \mu_W \log_{10} e$ and variance $\sigma^2_{LW} = (\sigma_W \log_{10} e)^2$. For var(W) values of 0.25 and 0.04, the corresponding values of σ_{LW} are 0.2052 and 0.0860, respectively. If the antilogs of these numbers are taken, values of 1.60 and 1.22 are obtained, providing an indication of the relative difference in the accuracy of traffic predictions between the two assumed distributions for the traffic multiplier, W.

The distribution of the number of years to failure, taking into account the variability in predicted traffic rates, can be determined at this point. As discussed previously, the actual number of 18-kip single-axle load applications to failure is distributed as LN $(0, \mu_p, \sigma_p)$. In addition, if the same distribution is transformed by division of a constant, R (the predicted traffic rate), the resulting distribution is also LN $[0, (\mu_p - n R), \sigma_p]$. However, because of the variability in predicted traffic rates, this distribution must be adjusted by the multiplier W, which is a random variable distributed as LN $(0, \mu_W, \sigma_W)$. Consequently, the distribution of the number of years to failure, T, is determined as the product of two lognormal distributions. By the multiplicative property of independent lognormal random variables, the distribution of T is also lognormal with parameters:

$$a_T = 0, \ \mu_T = (\mu_p - nR) + \mu_W, \ \text{and} \ \sigma_T = \sigma^2_p + \sigma_W^2 \qquad \text{(D-7)}$$

Figure D-9 shows distribution curves for the number of years to failure when the predicted traffic rate is 100,000 18-kip ESAL's/yr and when the error in the performance model, σ_y, is 0.266. These curves are for the case where deviations in all layer moduli and thicknesses are introduced (case II). Figure D-9 illustrates the effect of the error in the predicted traffic rate. For a smaller variance in the distribution of W, the curve obtained is slightly narrower than that for a larger variance, and is shifted to the right. The difference becomes more pronounced when the error in the performance model is smaller ($\sigma_y = 0.100$), as is evident in Figure D-10. This result again shows the significance of the error in the prediction equation. Even though the distribution curves obtained are only slightly different when $\sigma_y = 0.266$ (Figure D-9), the difference in life-cycle costs associated with the two distributions may be significant. An evaluation of life-cycle costs is important and is discussed below.

EVALUATION OF LIFE-CYCLE COSTS

Figure D-10 indicates how two different variations in design factors result in two different probability density distributions for predicted time to pavement failure. To evaluate the significance of a change in a time to failure distribution, it is necessary to express the distributions in terms of life-cycle costs.

The concept of considering life-cycle costs in the development of performance-based specifications was introduced in Chapter 1 and discussed in detail in Chapter 2. In this example, initial construction cost, routine annual maintenance cost, and user operating cost were considered. Rehabilitation cost, user delay cost, and salvage values were not considered because they are dependent upon decisions concerning future design and

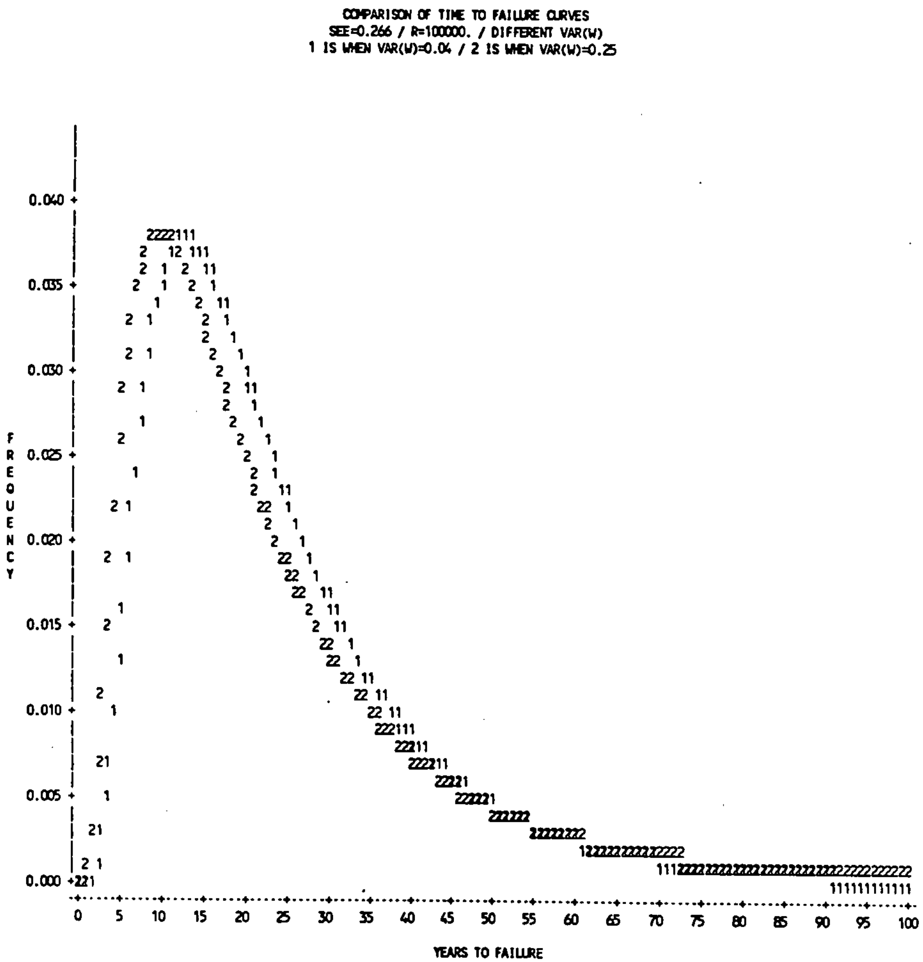

Figure D-9. Comparison of time to failure curves (SEE = 0.266/R = 100000./Different Var(W)).

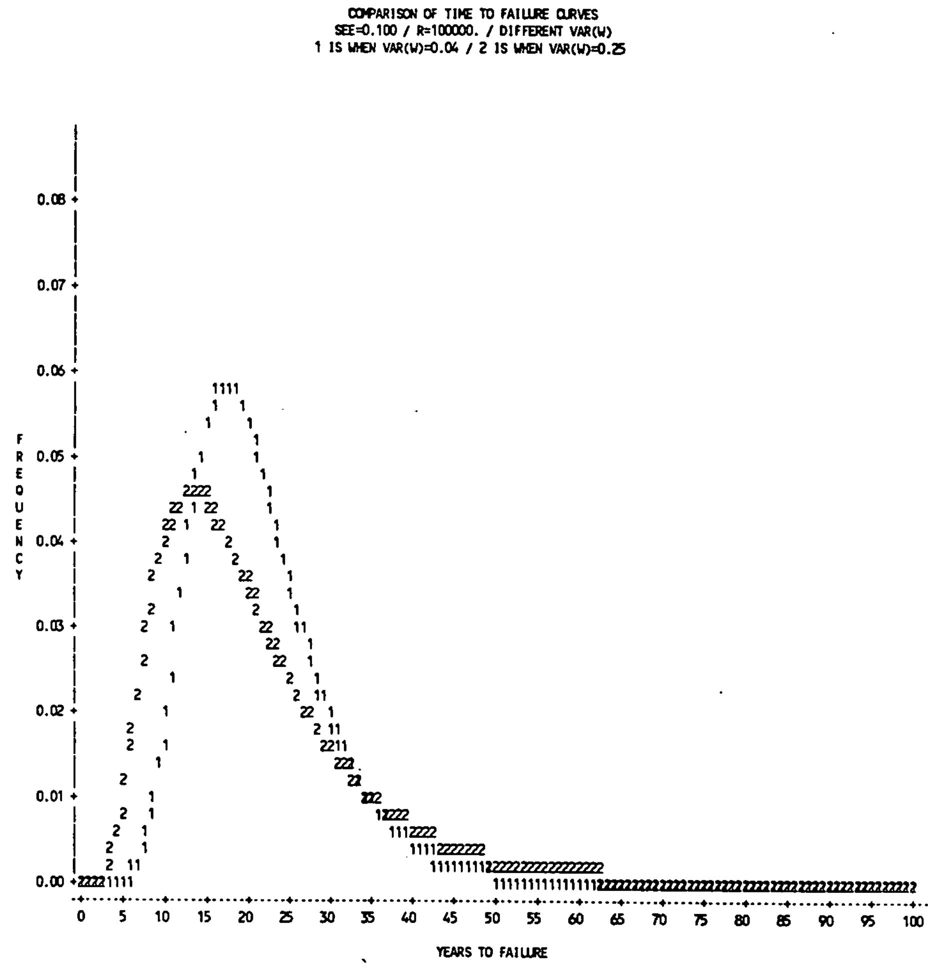

Figure D-10. Comparison of time to failure curves (SEE = 0.100/R = 100000./Different Var(W)).

construction (i.e., future performance periods). To simply the conditions, only one performance period was considered.

Total cost was expressed in terms of equivalent uniform annual cost, since the failure distribution curve represents many individual times to failure; to compare total costs associated with different times to failure in terms of net present value would be inappropriate. The following procedure was used in determining the costs.

Initial construction cost--An initial pavement construction cost of $200,000 per lane-mile was assumed for the original design on a two-lane highway. For a given time to failure, the corresponding equivalent uniform annual cost was calculated using a real discount rate of 5 percent.

Routine annual maintenance cost--Routine maintenance costs were associated with the condition of the pavement, assuming no cost for a newly constructed pavement, and $5,000 per lane-mile annual maintenance cost for a pavement in near-failure condition. For simplicity, a linear rate of pavement deterioration was assumed from initial construction to failure.

User operating cost--The additional user operating expense for driving on a road in poor condition was assumed to range from $0.0 per vehicle-mile for a newly constructed road to $.20 per vehicle-mile for a road in a failure condition. Again, pavement deterioration was assumed to be linear.

The life-cycle costs calculated using the procedure above are shown in Table D-5. The conditions considered are for cases I and II, as described earlier in this chapter. The total costs represent the total expected cost and were calculated by multiplying the cost for a given time to failure by the associated probability of failure and then summing the products over the probability density function.

Table D-5. Life-cycle costs (expressed in terms of equivalent uniform annual costs) for different pavement failure distributions (real interest rate = 5%).

Pavement Performance Model (SEE)	Traffic Rate (18-kip ESAL/yr)	Traffic Multiplier Var(W)	Initial Construction Cost ($/yr)	Routine Maintenance Cost ($/yr)	User Cost ($/yr)	Total Cost ($/yr)
0.266	100,000	0.04	19,137	2.063	22,028	43,229
0.266	100,000	0.25	21,882	2,074	22,144	46,100
0.100	100,000	0.04	16,968	2,114	22,570	41,651
0.100	100,000	0.25	19,216	2,112	22,549	43,877
0.266	500,000	0.04	72,277	2,197	137,295	211,769
0.266	500,000	0.25	85,211	2,246	140,412	227,869
0.100	500,000	0.04	60,823	2,206	137,867	200,895
0.100	500,000	0.25	72,136	2,174	135,869	210,178
[a]0.100	200,000	0.04	18,589	2,165	54,119	74,873
[b]0.100	200,000	0.04	18,742	2,164	54,097	75,004
[c]0.100	200,000	0.04	19,009	2,162	54,045	75,216

[a] C.V. = 5%
[b] C.V. = 10%
[c] C.V. = 15%

To evaluate the effect of some of the different components in the overall variance of predicted pavement performance, Table D-6 was developed to show the changes in cost that occur in Table D-5 as a result of specific factors. The results in Table D-6 indicate that the major effects of the different failure distributions are seen in the initial construction cost. In some cases, the maintenance and user costs increased (showed a negative change) with improved performance or traffic prediction. The magnitudes are small, however, and might be explained by the lognormal shape of the failure distribution curve and the discounting of future costs.

Of particular interest is the fact that the percentages in Table D-6 seem to be consistent, showing that a cost savings of 10 to 12 percent could be realized from the improved reliability of pavement performance and traffic predictions. For case I, where only the surface conditions changed and the performance and traffic predictions were at the "good" level, the results indicate that a 0.5 percent change in cost would occur when the coefficient of variation changes from 5 percent to 15 percent. This cost could be considered in developing the payment schedule for the contractor.

The actual costs determined in this exercise were considered unimportant, since only one example of a fixed set of conditions was examined. The important result is that a process has been developed whereby the effect of different factors on predicted pavement performance can be considered and related to costs, providing a technqiue that can be used to investigate the effect of materials, construction, and pavement variables on the life-cycle cost of pavements.

Table D-6. Effect of variability on life-cycle costs.

	Improve Performance Prediction				Improve Traffic Prediction				Total			
	Const	Maint	User	Total	Const	Maint	User	Total	Const	Maint	User	Total
Traffic - 500,000 ESAL/yr												
$ saved	13,075	40	4,543	17,691	11,313	-32	-1,998	9,283	24,388	40	2,545	26,974
%	15	2	3	8	16	-1	-1	4	29	2	2	12
% total	6	0	2		5	0	-1		11	0	1	
Traffic - 100,000 ESAL/yr												
$ saved	2,666	-38	-405	2,223	2,248	-2	-21	2,226	4,914	-40	-426	444
%	12	-2	-2	5	12	0	0	5	23	2	2	10
% total	6	0	-1		5	0	0		11	0	-1	

Improve Construction of Surface

	Construction	Maintenance	User	Total
Traffic - 200,000 ESAL/yr				
$ saved	420	-3	-74	343
%	2	0	0	.5

APPENDIX E—PERSPEC COMPUTER PROGRAM

The PERSPEC program demonstrates a conceptual framework by which a payment schedule can be determined based on a comparison of the calculated performance of the as-constructed and target pavements. The program contains several complicated prediction algorithms; however, these are transparent to the user. The inputs are the "target" M&C variables defined by the agency and the M&C variables "measured" after the road is constructed. The program will estimate values for the fundamental pavement response variables such as strains and modulus and pavement condition indicators such as roughness, rutting, and cracking. With this information, the cost of the as-constructed pavement is assessed based on the dollar value of the bid, agency maintenance cost, and user costs relative to the target design. The program computes performance and cost estimations on an annual basis, and then prints out an approximate payment factor.

The program has been designed to be user-friendly such that the complexities of the program are transparent to the user. It is intended that the program be considered as part of the payment schedule so that the contractor and the agency can assess, at any point in the design, building, construction, and payment processes, the consequence of design, material, and construction variables. The program also allows the contractor to assess the payment consequences of departure from target values and materials and construction that are "out of specification."

PROGRAM DESCRIPTION

PERSPEC was developed using the FORTRAN 77 programming language. A structured approach was used to preserve the modularity required for future modifications.

The default subroutine is of particular interest to the agency. Here, condition and cost information can be established depending upon a geographic region. Also, the agency can control the amount of information displayed by the program. This feature will be valuable during the debugging and analysis stages.

As an example of program modularity, a decision may be made in the future to add a finite element program that calculates pavement cracking resulting from thermal effects. The programmer need only add to the main, input, defaults, and performance routines. The decision on what to add will be based on the need for user-input information and known geographical conditions. The agency may choose to include input variables like "Average Annual Solar Lumination" in the defaults routine. Variables for which the user must supply values can be included in the input routine. A description of the input required for the program is given in Table E-1.

The programmer, when modifying the program, should also note that the performance subroutine contains all performance-related values. This decision was made for ease of future modification of the models to include their mutual dependence. For example, the independent performance equation for roughness can easily be changed to depend upon the interactive effects of cracking and rutting. A flow diagram for the program is shown in Figure E-1.

Table E-1. Program input

<u>MAIN INPUT</u>:

1. Total number of runs to be made
2. User-selected name for each run, e.g., Run 1

<u>WITCZAK INPUT</u>:

1. Percent aggregate passing no. 200 sieve, percent
2. Percent air voids, percent
3. Percent asphalt by weight of mix, percent

<u>ELSYM5 INPUT</u>:

1. Thickness of layer no. 1, inches
2. Thickness of layer no. 2, inches
3. Elastic modulus of layer no. 2, lb/in^2
4. Elastic modulus of layer no. 3, lb/in^2

<u>TRAFFIC INPUT</u>:

1. 18-kip ESAL's yr design lane
2. Percentage growth, percent per year
3. Vehicles per day, design lane

<u>COST INPUT</u>:

1. Bid price, dollars per square yard
2. Discounted interest rate

START	
DEFAULTS	All default parameters are set in this routine
GREET	This routine greets the user with general information
INPUT	This routine handles the input for all other subroutines
WITCZAK	Calculates elastic modulus of layer #1 as input to ELSYM5 (<u>8</u>).
ELSYM5	Calculates the principal strains at two given locations (<u>87</u>).
TRAFFIC	Calculates traffic array and cumulative ESAL's.
PERFORMANCE	Calculates annual rutting, roughness, and cracking (<u>18</u>).
COST	Associates cost with roughness, rutting, and cracking.
PAYMENT	Outputs percentage of bid price paid to the contractor.
STOP	

Figure E-1. PERSPEC flow diagram.

SAMPLE RUN

Three sample runs were made for demonstration of the PERSPEC concept. The outputs indicates that the program is well-behaved with respect to maximal variations of the M&C variables while all other inputs are held constant. A sample input file is contained in Table E-2 and simple output are contained in Tables E-3 through E-5. The following conditions were used for the three runs:

1. RUN 1: Target specification.
2. RUN 2: Below target specification.
3. RUN 3: Above target specification.

SOURCE CODE

The source code for the PERSPEC program is listed at the end of this appendix. The ELSYM5 code has been omitted since it is very large and does not explicitly aid in the understanding of the basic concepts.

Table E-2. Input data for demonstration runs.

Number of runs = 3

Run 1--Name of this run

7--Percent passing No. 200, percent
5.5--Percent air voids, percent
6.5--Percent asphalt by weight of mix, percent
4--Thickness of layer 1, in
6--Thickness of layer 2, in
50,000--Modulus of layer 2, lb/in^2
15,000--Modulus of layer 3, lb/in^2
200,000--18-kip ESAL's in design lane
5--Percentage growth in traffic, percent
5,000
10--Bid price, $\$/yd^2$
5--Discounted interest rate, percent

Run 2--Name of this run

4--Percent passing No. 200, percent
8--Percent air voids, percent
8--Percent asphalt by weight of mix, percent
4--Thickness of layer 1, in
6--Thickness of layer 2, in
50,000--Modulus of layer 2, lb/in^2
15,000--Modulus of layer 3, lb/in^2
200,000--18-kip ESAL's in design lane
5--Percentage growth in traffic, percent
5,000
10--Bid price, $\$/yd^2$
5--Discounted interest rate, percent

Run 3--Name of this run

10--Percent passing No. 200, percent
2--Percent air voids, percent
4.5--Percent asphalt by weight of mix, percent
4--Thickness of layer 1, in
6--Thickness of layer 2, in
50,000--Modulus of layer 2, lb/in^2
15,000--Modulus of layer 3, lb/in^2
200,000--18-kip ESAL's in design lane
5--Percentage growth in traffic, percent
5,000
10--Bid price, $\$/yd^2$
5--Discounted interest rate, percent

Table E-3. PERSPEC results for Run 1.

Year of Failure
Rutting: 11.1 Roughness: 8.1 Cracking: 12.8

Year	Rutting	Roughness	Cracking
1	0.00608	4.15889	0.00121
2	0.02431	4.03557	0.00485
3	0.05469	3.83004	0.01092
4	0.09722	3.54229	0.01941
5	0.15191	3.17233	0.03032
6	0.21875	2.72015	0.04366
7	0.29774	2.18576	0.05943
8	0.38889	1.56916	0.07763
9	0.49219	0.87034	0.09825
10	0.60764	0.08931	0.12129

Year No.	Agency Cost	User Cost	Annual Cost	Rate
1	0.00	0.00	10.50	1.050
2	0.00	0.06	5.41	0.538
3	0.02	0.35	3.81	0.367
4	0.07	1.33	3.21	0.282
5	0.20	3.83	3.24	0.231
6	0.46	9.27	3.89	0.197
7	0.95	19.85	5.32	0.173
8	1.78	38.80	7.83	0.155
9	3.12	70.68	11.79	0.141
10	5.15	121.70	17.72	0.130

Table E-4. PERSPEC results for Run 2.

Year of Failure
Rutting: 9.0 Roughness: 6.0 Cracking: 8.3

Year	Rutting	Roughness	Cracking
1	0.00929	4.12484	0.00289
2	0.03717	3.89936	0.01154
3	0.08364	3.52355	0.02597
4	0.14869	2.99742	0.04616
5	0.23233	2.32097	0.07213
6	0.33456	1.49420	0.10387
7	0.45537	0.51710	0.14137
8	0.59477	-0.61031	0.18465
9	0.75276	-1.88805	0.23370
10	0.92933	-3.31612	0.28852

Year No.	Agency Cost	User Cost	Annual Cost	Rate
1	0.00	0.01	10.51	1.050
2	0.01	0.17	5.47	0.538
3	0.06	1.01	4.07	0.367
4	0.23	3.81	3.96	0.282
5	0.64	10.97	4.99	0.231
6	1.48	26.57	7.50	0.197
7	3.03	56.93	12.09	0.173
8	5.69	111.29	19.65	0.155
9	9.95	202.73	31.33	0.141
10	16.44	349.07	48.63	0.130

Table E-5. PERSPEC results for Run 3.

Year of Failure
Rutting: 16.1 Roughness: 13.2 Cracking: 25.7

Year	Rutting	Roughness	Cracking
1	0.00290	4.18458	0.00030
2	0.01161	4.13833	0.00122
3	0.02612	4.06125	0.00274
4	0.04643	3.95334	0.00486
5	0.07254	3.81459	0.00760
6	0.10446	3.64501	0.01094
7	0.14218	3.44460	0.01489
8	0.18571	3.21335	0.01945
9	0.23503	2.95127	0.02462
10	0.29016	2.65836	0.03039

Year No.	Agency Cost	User Cost	Annual Cost	Rate
1	0.00	0.00	10.50	1.050
2	0.00	0.01	5.38	0.538
3	0.00	0.07	3.70	0.367
4	0.01	0.25	2.89	0.282
5	0.04	0.71	2.48	0.231
6	0.08	1.72	2.33	0.197
7	0.17	3.69	2.39	0.173
8	0.31	7.21	2.71	0.155
9	0.55	13.13	3.33	0.141
10	0.91	22.61	4.34	0.130

SOURCE CODE FOR PERSPEC

```
c***********************************************************************
c****  main  ****  main  ****  main  ****  main  ****  main  ****  main*
c***********************************************************************
      program perfspec
c
c     This is the main program.
c
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      integer runs
      logical ask,prt0,prt1,prt2,prt3

      open(5,file='my.dat')
      open(6,file='CON',CARRIAGECONTROL='LIST')
      call defaults
      if(prt0)call greet


      if(ask)write(6,*)'Enter Number of Runs (1-20)'
      read(5,*)runs
      if((runs.lt.1).or.(runs.gt.20))then
      write(6,*)'Error Invalid Number of Runs'
      write(6,*)'Program Terminated'
      stop
      else
      if(prt0)then
        write(6,*)' '
        write(6,*)'Program is running...'
        write(6,*)' '
        endif
      endif

      do 10,numrun=1,runs
        call input
        call witz
        call elsym5
        call traf
        call perform
        call cost
        call paymt
   10 continue

      stop
      end
```

```
c************************************************************************
c**** defaults **** defaults **** defaults **** defaults **** defaults *
c************************************************************************
      subroutine defaults
c
c     All default parameters are set in this routine.
c
      common /de2pe/frutt,frough,fcrack
      common /de2co/acfru,acfro,acfcr,ucfru,ucfro,ucfcr,convfact
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      logical ask,prt0,prt1,prt2,prt3

c     ask --> prompts user for information.
      ask=.false.
c     prt0 --> Prints out liability information.
      prt0=.true.
c     prt1 --> Prints out performance information.
      prt1=.true.
c     prt2 --> Prints out cost information.
      prt2=.true.
c     prt3 --> Prints out payment information.
      prt3=.true.
c     frutt --> Failure level of rutting.
      frutt=0.75
c     frough --> Failure level of roughness.
      frough=1.5
c     fcrack --> Failure level of cracking.
      fcrack=0.2
c     acfru --> Agency cost for failure due to rutting. ($/sq-yd/mi)
      acfru=1.75
c     acfro --> Agency cost for failure due to roughness. ($/sq-yd/mi)
      acfro=0.50
c     acfcr --> Agency cost for failure due to cracking. ($/sq-yd/mi)
      acfcr=1.00
c     ucfru --> User cost for failure due to rutting. ($/vehical/mi)
      ucfru=0.10
c     ucfro --> User cost for failure due to roughness. ($/vehical/mi)
      ucfro=0.05
c     ucfcr --> User cost for failure due to cracking. ($/vehical/mi)
      ucfcr=0.01
c     convfact --> Conversion factor vehicals / day to /year/sq-yd.
      convfact=0.051846

      return
      end
```

```
c************************************************************************
c**** greet  **** greet  **** greet  **** greet  **** greet  **** greet*
c************************************************************************
      subroutine greet
c
c     This routine greets the user with general information.
c
      write(6,*)'*******************************************************'
      write(6,*)'*******************************************************'
      write(6,*)'*******************************************************'
      write(6,*)'*****               PERFSPEC                     *****'
      write(6,*)'*****              NCHRP 1026-A                  *****'
      write(6,*)'*****                May-1988                    *****'
      write(6,*)'*****                                            *****'
      write(6,*)'*****   This program was developed by the        *****'
      write(6,*)'*****   Pennsylvania Transportation Institute    *****'
      write(6,*)'*****   to illustrate a framework by which a     *****'
      write(6,*)'*****   contractor can be paid based on an       *****'
      write(6,*)'*****   assesment of road quality.               *****'
      write(6,*)'*****                                            *****'
      write(6,*)'*****   This program was coded in standard       *****'
      write(6,*)'*****   Fortran 77. A structured approach was    *****'
      write(6,*)'*****   used in programming for ease of          *****'
      write(6,*)'*****   modification.                            *****'
      write(6,*)'*****                                            *****'
      write(6,*)'*****   The Pennsylvania Transportation          *****'
      write(6,*)'*****   Institute will assume no liability       *****'
      write(6,*)'*****   for any loss (direct or indirect) due    *****'
      write(6,*)'*****   to the use of this program.              *****'
      write(6,*)'*****                                            *****'
      write(6,*)'*****                                            *****'
      write(6,*)'*******************************************************'
      write(6,*)'*******************************************************'
      write(6,*)'*******************************************************'
      return
      end
```

```
c***********************************************************************
c****  input  ****  input  ****  input  ****  input  ****  input  ******
c***********************************************************************
      subroutine input
c
c     This subroutine handles the input for all routines.
c
      common /in2wi/p200,void,pac
      common /in2el/thkns1,thkns2,emod2,emod3,z1,z2
      common /in2tr/esal,growth,vehical
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      real*8 p200,void,pac
      logical ask
      character*80 title
      read(5,5)title
      write(6,5)title
 5    format(a80)
c
c     witczak input
c
      if (ask) write(6,*)'Percent aggregate passing #200 sieve (4-10)?'
      read(5,*)p200
      if (ask) write(6,*)'Percent air voids (3-8)?'
      read(5,*)void
      if (ask) write(6,*)'Percent asphalt by weight of mix (5-8)?'
      read(5,*)pac
c
c     elsym5 input
c
      if (ask) write(6,*)'Thickness of layer #1 (2-8)?'
      read(5,*)thkns1
      if (ask) write(6,*)'Thickness of layer #2 (2-8)?'
      read(5,*)thkns2
      if (ask) write(6,*)'Elastic modulus of layer #2 (50,000)?'
      read(5,*)emod2
      if (ask) write(6,*)'Elastic modulus of layer #3 (5,000)?'
      read(5,*)emod3
      z1=thkns1-.01
      z2=thkns1+thkns2+.01
c
c     traffic input
c
      if (ask) write(6,*)'18 kip ESAL/yr design lane (200,000)?'
      read(5,*)esal
      if (ask) write(6,*)'Percentage growth (5)?'
      read(5,*)growth
      if (ask) write(6,*)'Vehicals per day, design lane (5,000)?'
      read(5,*)vehical
c
c     global input
c
      if (ask) write(6,*)'Bid price $ per square yard (10)?'
      read(5,*)bid
      if (ask) write(6,*)'Discount (interest) rate (5)?'
      read(5,*)rate
      rate=rate/100

      return
      end
```

```
c***********************************************************************
c****  witz  ****  witz  ****  witz  ****  witz  ****  witz  ****  witz*
c***********************************************************************
      subroutine witz
c
c     This routine uses Witczak's equation to determin the modulus.
c
      common /in2wi/p200,void,pac
      common /wi2el/emod1
      real*8 loge1,p200,freq,void,visc,pac,temp
      freq=1.0d0
      visc=10.0d0
      temp=77.0d0
c
      loge1=5.553833d0+0.028829d0*p200/(freq**0.17033d0)-0.03476d0*void
     &+0.070377d0*visc+0.000005d0*temp**(1.3d0+0.49825d0*dlog10(freq))
     &*(pac**0.5d0)-0.00189d0*temp**(1.3d0+0.49825d0*dlog10(freq))
     &*(pac**0.5d0)/(freq**1.1d0)+0.931757d0/(freq**0.02774d0)
c
      emod1=real(10.0d0**(loge1))
      return
      end
```

```
c***********************************************************************
c****  traf  ****  traf  ****  traf  ****  traf  ****  traf  ****  traf*
c***********************************************************************
      subroutine traf
c
c     This routine determins the traffic vs. time.
c
      common /in2tr/esal,growth,vehical
      common /tr2pe/cumesal
      common /tr2co/trafs
      real trafs(50),cumesal(50)
c
      sum=0
      do 10 i=1,50
      sum=sum+(((1+(growth/100))**float(i))*esal)
      cumesal(i)=sum
      trafs(i)=(1+(growth/100))**(float(i))*vehical
10    continue

      return
      end
```

```
c**************************************************************************
c****  perform  ****  perform  ****  perform  ****  perform  ***********
c**************************************************************************
      subroutine perform
c
c     This subroutine determins performance.
c
      common /el2pe/pel,pe3
      common /tr2pe/cumesal
      common /pe2co/rutts,roughs,cracks
      common /de2pe/frutt,frough,fcrack
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      real cumesal(50),rutts(50),roughs(50),cracks(50)
      logical ask,prt0,prt1,prt2,prt3

c     Calculate # of applications to failure.

      arutt=30*(1/pe3)**1.5
      arough=10.0**((2.15122)-(597.662*pe3)-(1.32967*log10(pe3)))
      acrack=1.33e-1*(1/pel)**2.0

c     Calculate year of each failure.

      do 10 i=2,50
      span=cumesal(i)-cumesal(i-1)
      if((cumesal(i-1).le.arutt).and.(cumesal(i).ge.arutt))then
        yrutt=float(i)+((arutt-cumesal(i-1))/span)
        endif
      if((cumesal(i-1).le.arough).and.(cumesal(i).ge.arough))then
        yrough=float(i)+((arough-cumesal(i-1))/span)
        endif
      if((cumesal(i-1).le.acrack).and.(cumesal(i).ge.acrack))then
        ycrack=float(i)+((acrack-cumesal(i-1))/span)
        endif
 10   continue

      if(arutt.lt.cumesal(1))yrutt=arutt/cumesal(1)
      if(arough.lt.cumesal(1))yrough=arough/cumesal(1)
      if(acrack.lt.cumesal(1))ycrack=acrack/cumesal(1)
      if(arutt.eq.0)yrutt=50
      if(arough.eq.0)yrough=50
      if(acrack.eq.0)ycrack=50

      if(prt1)write(6,*)' '
      if(prt1)write(6,*)'Year of failure '
      if(prt1)write(6,4)yrutt,yrough,ycrack
      if(prt1)write(6,*)' '
 4    format('Rutting:',f4.1,'   Roughness:',f4.1,'   Cracking:',f4.1)

c     Calculate relative annual performance.
      if(prt2)write(6,*)'Year     rutting     roughness     cracking'
      if(prt2)write(6,*)' '

      do 20 i=1,50
        rutts(i)=(float(i)/yrutt)**2.0*frutt
        roughs(i)=4.2-((float(i)/yrough)**2.0*(4.2-frough))
        cracks(i)=(float(i)/ycrack)**2.0*fcrack
        if((i.le.20).and.(prt1))then
          write(6,6)i,rutts(i),roughs(i),cracks(i)
        endif
 6      format('   ',i4,'  ',f9.5,'      ',f9.5,'      ',f9.5)
        rutts(i)=(rutts(i)/frutt)**2
        roughs(i)=((4.2-roughs(i))/(2.7*frough))**2
        cracks(i)=(cracks(i)/fcrack)**2
 20   continue
      return
      end
```

```
c*********************************************************************
c****  cost  ****  cost  ****  cost  ****  cost  ****  cost  ****  *****
c*********************************************************************
      subroutine cost
c
c     This routine determins the annual cost of a road.
c
      common /tr2co/trafs
      common /co2pa/totcost
      common /pe2co/rutts,roughs,cracks
      common /de2co/acfru,acfro,acfcr,ucfru,ucfro,ucfcr,convfact
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      real trafs(50),roughs(50),rutts(50),cracks(50)
      real agccost(50),usrcost(50),totcost(50)
      real cagccost(0:50),cusrcost(0:50),rates(0:50)
      logical ask,prt0,prt1,prt2,prt3

c     Initalize cumelative arrays. (fortran's retention property)

      do 10 i=0,50
      if(i.eq.0)then
        n=1
      else
        n=i
      endif
      cagccost(i)=0
      cusrcost(i)=0
      rates(i)=(rate*(1+rate)**i)/((1+rate)**n-1)
 10   continue

c     Calculate annual and cumelative agency, user, and total costs.

      if(prt2)then
        write(6,*)' '
        write(6,*)'  Year     Agency     User    Annual'
        write(6,*)'   No.       Cost     Cost      Cost      Rate'
        write(6,*)' '
      endif

      do 20 i=1,50

c       Calculate agency costs.

        agccost(i)=rutts(i)*acfru+roughs(i)*acfro+cracks(i)*acfcr
        cagccost(i)=cagccost(i-1)+agccost(i)

c       Calculate user costs.

      rutts(i)=rutts(i)*trafs(i)*convfact
      roughs(i)=roughs(i)*trafs(i)*convfact
      cracks(i)=cracks(i)*trafs(i)*convfact
        usrcost(i)=rutts(i)*ucfru+roughs(i)*ucfro+cracks(i)*ucfcr
        cusrcost(i)=cusrcost(i-1)+usrcost(i)

c       Calculate total costs.

        totcost(i)=(cagccost(i)+cusrcost(i)+bid)*rates(i)
```

```
c       Output values to the standard output device.

      if((i.le.20).and.(prt2))then
        write(6,6)i,cagccost(i),cusrcost(i),totcost(i),rates(i)
      endif
 6      format('     ',i2,'  ',f8.2,'  ',f8.2,'  ',f8.2,' ',f8.3)

 20   continue

      return
      end


c*********************************************************************
c****  paymt  ****  paymt  ****  paymt  ****  paymt  ****  paymt  ******
c*********************************************************************
      subroutine paymt
c
c     This routine the percentage of bid price to be paid.
c
      common /co2pa/totcost
      common /global/numrun,ask,bid,rate,prt0,prt1,prt2,prt3
      real totcost(50),at,ac
      logical ask,prt0,prt1,prt2,prt3

      save at

c
c     Search for the lowest cost "ac" and year "lc".
c
      ac=totcost(1)
      lc=1
      do 10 i=2,20
      if(totcost(i).le.ac)then
        ac=totcost(i)
        lc=i
      endif
 10   continue

c
c     Output the payment amount based on the performance equations.
c
      if((numrun.gt.1).and.(prt3))then
        payment=bid-(ac-rate+1)*(((1+rate)**lc-1)/(rate*(1+rate)**lc))
        percent=payment/bid*100
        write(6,*)' '
        write(6,7)payment
 7      format('Payment = $',f5.2)
        write(6,8)percent
 8      format('% of Bid =',f4.0)
        write(6,*)' '
      else
        at=ac
      endif

      return
      end
```

APPENDIX F—EXPERIMENT PLANS

Experiment plans were developed for typical laboratory and field studies that are needed to fully implement the PRS specification experiment. The experiment plan for the laboratory study deals primarily with the sensitivity of the Level E FMRV variables to changes in the materials variables. The field experiment, which is much more comprehensive in scope, deals with the verification and development of the relationships between pavement performance and the M&C and FMRV variables. An abbreviated version of the proposed laboratory study was completed as part of this project, whereas the field study was only proposed. Ideally, this study, or one similar to it, will be executed as part of the Strategic Highway Research Program (SHRP), presumably as part of the Special Pavement Studies (SPS) program.

LABORATORY STUDY

The models that are needed to develop PRS specifications for hot-mix asphalt concrete include those that relate M&C variables to FMRV variables (A-E models). These models are required because the FMRV variables may not, in general, be suitable as measures upon which the acceptance of materials and construction can be based. For example, regardless of the development of fundamental mixture tests (AAMAS system), there are certain materials variables, such as asphalt content and percent air voids, that will likely remain as quality control, acceptance, and payment variables. The FMRV variables will tend to be tedious to evaluate, require highly trained personnel and specialized equipment, and be difficult to complete in a timely manner. Therefore, the M&C variables will likely serve as low-cost, rapidly performed surrogates for the FMRV variables. If they are to be satisfactory surrogates, they must reliably and accurately estimate the FMRV variables. The purpose of the laboratory experiment was to demonstrate how a laboratory study can be designed to validate existing or develop new models that relate materials variables to FMRV variables.

Study Objective

There are several models in the literature that relate M&C-type variables to the more fundamental material response variables (see Appendix B). However, these models exhibit certain critical shortcomings that limit their usefulness to this project:

- The models, or available data bases upon which they are based, are weighted in favor of target values for the M&C variables and, therefore, cannot account for the effects of M&C nonconformance (i.e., "out of specification" materials and construction).
- The models or available data are often limited to a narrow range of mixture characteristics; rendering their extrapolation to a wide range of mixture behaviors is inappropriate.
- Most existing models do not address the question of M&C variable interaction.

As a consequence of the limitations of the existing models and data bases, several laboratory studies were planned that could be used to identify which of the commonly used M&C variables are performance-related, as evidenced by a statistically strong relationship with one or more of the FMRV variables.

In the event that the conventional M&C variables do not appear to be acceptable surrogates for the FMRV (Level E) variables, it will be necessary to look for additional or replacement variables. This would most likely point to the adoption of more fundamental M&C variables, such as diametral modulus or tensile strength, which may be reasonable and desirable given recent and anticipated developments in the testing of hot-mix asphalt (3).

Choice of Study Variables

Some judgment was needed for the selection of the M&C variables to be incorporated into the laboratory study. A careful distinction must be made between M&C variables and mixture design variables. A brief overview of the mix design process illustrates this distinction. At the present time, the practice in most States is to source-accept the components in a mix, specifying a range of accepted test values by a maximum or minimum value. For example, the LA abrasion value for coarse stone could be specified as being less than 40 percent, or the 140 °F viscosity for the asphalt cement could be specified as being 2000 ± 200 poises. Adjustments in payment are generally not allowed for materials falling outside these specification limits. The materials are simply rejected if they are not within the limits.

The usual sequence in the mix design process is, first, to select the aggregates, usually consisting of two or more size fractions from one or more sources. The aggregates are then blended in various proportions until an acceptable trial gradation is achieved. Trial batches are then mixed with varying amounts of asphalt cement. The batches are compacted and then tested in accordance with the Marshall or Hveem procedure. An optimum asphalt content for the mix is calculated from the test results. The optimum mix design for the particular aggregate source and aggregate gradation is established on the basis of these results. Selected properties of the optimum mix (air voids, stability, flow, VMA, R values, etc.) are checked against the masterband, and if the design values fall within the masterband, the optimum design values are specified as target values for the particular mixture design. Clearly, different mix designs will have different target values, but all mixes that fall within the masterband are considered to be of equal performance.

A list of factors that could be included in the laboratory study is given in Table F-1. The variables are listed according to:

- Mixture design variables
- Mixture design acceptance criteria
- M&C acceptance variables
- Conditioning effects
- Response variables

The mixture design variables include properties of both the aggregates and the asphalt cement. A much larger listing would result if all of the variables in Table 3 (Chapter 2) were included. Table F-1 provides a realistic listing that will result in mixtures with widely varying properties. The variables in Table F-1 also reflect the distress mechanisms that are currently of concern to materials engineers: surface and subgrade rutting, moisture damage, fatigue cracking, and thermal cracking.

After a mixture design has been completed, it is accepted on the basis of largely empirical criteria (not to be confused with M&C acceptance criteria) which include air voids, stability, flow, voids

Table F-1. Variables in an ideal laboratory study.

Type of Variables	Number of Levels	Description of Levels
1. Mixture Design Variables		
Gradation, Overall gradation	2	Dense, open
Gradation, Percent passing No. 200	2	4-5%, 7-8%
Aggregate stability, Crush count	2	Angular crushed, gravel with 50% crush
Absorption	2	Absorptive, nonabsorptive
Moisture susceptibility	2	TSR > 0.95, 0.60 <TSR<0.70
Rheologic type	2	High and low c parameter
Temperature susceptibility	2	High and low pen index
Aging susceptibility	2	High and low aging index
Asphalt grade	2	AC-20, AC-5
2. Mixture Design Acceptance Criteria		
Design asphalt content	NA	Dependent on mix design
Design air voids	NA	Dependent on mix design
Design VMA	NA	Dependent on mix design
Design stability	NA	Dependent on mix design
Design flow	NA	Dependent on mix design
Tensile strength ratio	NA	Dependent on mix design
3. M&C Acceptance Variables		
/ Gradation (% < No. 200)	3	Target, target +2%, +4%
/ Asphalt content	3	Target, target +0.6%, +1.2%
/Air voids	3	Target, target +3%, +6%
Thickness (yield)	NA	Not mix related
Roughness	NA	Not mix related
4. Conditioning Effects		
Aging, Asphalt hardening	2	None, lab aged
Moisture conditioning	2	None, lab conditioned

Table F-1. Variables in an ideal laboratory study (continued).

Type of Variables	Number of Levels	Description of Levels
5. Response Variables		
Strength	3 specimens	Diametral tensile strength
Modulus	3 specimens	Diametral tensile modulus
Fatigue parameter	3 specimens	Diametral tensile testing
Creep parameters	3 specimens	Diametral tension testing
Low-temperature fracture	3 specimens	Diametral tension testing

Note: A full factorial study would require 55,296 cells. A 1/8 factorial requires 6,912 cells. With 9 specimens per cell, this study would require 62,208 experimental units; full replication would require 497,664 specimens.

in the mineral aggregate (VMA), and percent voids filled with asphalt (VFA). Some States include other mixture acceptance criteria, such as asphalt content and tensile strength ratio (TSR). Maximum or minimum limits are given in typical mix design specifications, and, if the measured properties of the proposed mix fall within the specification masterband, the mix is acceptable. These acceptance criteria are dependent upon the mixture design variables and, therefore, cannot be considered independent variables. For example, the VMA is determined by the gradation and asphalt content of the mix. Because of this dependency, the mixture design acceptance criteria are not suitable as M&C variables because they place the supplier in a position of double jeopardy which is legally indefensible.

The M&C variables in Table F-1 were chosen as potential candidates for a laboratory study in response to the following questions:

1. Are they currently being used by highway agencies?
2. Are they simple and easy to perform?
3. Are they related to performance?
4. Are they factors over which the contractor has control?
5. Are they free of double jeopardy?

Asphalt content and gradation are controlled by the hot-mix producer. Air voids, thickness, and roughness are under the control of the paving contractor, which may or may not be the same corporate entity as the hot-mix producer. Thickness is not generally used as a basis for payment; yield, for example, square yards per ton of placed material, is the usual basis for payment. Air voids content is synonymous with pavement density and, for payment purposes, is usually measured as a percentage of the optimum mixture design (laboratory) density.

Roughness is not totally within the control of the contractor, but is affected by the roughness and stiffness of the underlying layer. Although thickness and roughness are not appropriate laboratory study variables, they will be considered as input to the performance predictors (Levels E-G) when the sensitivity of the M&C variables is studied. Therefore, they are included in Table F-1.

Current A-E models, for the most part, neglect the effect of mix conditioning on the FMRV variables. During service, mixes are exposed to moisture and to long-term hardening though oxidation and stearic hardening. Failure to include these effects is one of the major shortcomings of the existing data bases. These shortcomings are being addressed in the on-going SHRP program, especially the A-003A contract, "Performance-Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures." Therefore, accommodations were made for laboratory aging and moisture conditioning, as shown in Table F-1.

<u>Recommended Experimental Design</u>

It was obvious that available resources would be insufficient to complete the laboratory experiment given in Table F-1. Therefore, to demonstrate the design process, two alternative designs were developed. The first experiment design, shown in Table F-2, is the more straightforward of the two designs. This experiment design would provide data for the AASHTO design equation and for mechanistic models that predict fatigue cracking and surface and subgrade rutting. Thermal cracking and moisture damage are not addressed, but the experiment design includes specimen aging.

In the first experiment design, the diametral tensile strength and modulus are measured directly. The creep and fatigue parameters are

Table F-2. Variables for 1/4 replication, 2^8 study.

Type of Variables	Number of Levels	Description of Levels
<u>1. Mixture Design Variables</u>		
Gradation	2	Dense, open
Aggregate stability, Crush count	2	Angular crushed, gravel with 50% crush
Absorption	2	Absorptive, nonabsorptive
Aging susceptibility	2	High and low aging index
<u>2. Mixture Design Acceptance Criteria</u>		
Design asphalt content	NA	Dependent on mix design
Design air voids	NA	Dependent on mix design
Design VMA	NA	Dependent on mix design
Design stability	NA	Dependent on mix design
Design flow	NA	Dependent on mix design
Tensile strength ratio	NA	Dependent on mix design
<u>3. M&C Acceptance Variables</u>		
/ Gradation (% < No. 200)	2	Target, target +3%
/ Asphalt content	2	Target, target +1%
/Air voids	2	Target, target +4%
Thickness (yield)	NA	Not mix related
Roughness	NA	Not mix related
<u>4. Conditioning Effects</u>		
Aging, Asphalt hardening	2	None, lab aged

Table F-2. Variables for 1/4 replication, 2^8 study (continued).

Type of Variables	Number of Levels	Description of Levels
<u>5. Response Variables</u>		
Strength	2 specimens	Diametral tensile strength
Modulus	3 specimens	Diametral tensile modulus
Fatigue parameters	3 specimens	Diametral tensile testing
Creep parameters	3 specimens	Diametral tension testing

Note: A full factorial study would require 256 cells. A 1/4 factorial requires 64 cells. With 2 replicates, this study would require 128 experimental units, or specimens. Direct measurement of creep and fatigue response variables will increase the number of test specimens to 320.

measured using three specimens for each cell, with the actual testing performed in the repeated diametral mode. Gyratory compaction is used to prepare the specimens. With a one-quarter replication of a 2^8 factorial design, there are 64 experimental cells, requiring 320 test specimens. As an alternative, the fatigue and creep parameters could be estimated using measured tensile strength and modulus data. This would reduce the number of test specimens to 128.

The second study is more comprehensive in that both low-temperature cracking and moisture damage are addressed. The experimental plan is described in Table F-3. Three additional study variables, aggregate stripping potential, asphalt temperature susceptibility, and moisture conditioning, have been added to account for the additional distress modes. Aggregate gradation is included only in terms of dense versus open gradation. In this study, creep and fatigue properties would be estimated from empirical procedures, and thermal cracking would be addressed with an additional set of three specimens tested at low temperature. This study permits the evaluation of the sensitivity of all four of the commonly cited distress modes: surface and subgrade rutting, thermal cracking, and fatigue. A one-eighth replication of a 2^{10} factorial experiment can be obtained by use of the defining equations:

$$x_1 + x_2 + x_3 + x_4 + x_5 = 0,\ 1 \text{ (Model 1)}$$

$$x_4 + x_5 + x_6 + x_7 + x_8 = 0,\ 1 \text{ (Model 2)}$$

$$x_1 + x_4 + x_7 + x_8 + x_9 + x_{10} = 0,\ 1 \text{ (Model 3)} \qquad \text{(F-1)}$$

With this choice for a fractional factorial, no aliasing occurs among the 10 main effects and the 45 two-way interactions. This should permit a close estimation of these effects and, furthermore, make it

Table F-3. Study variables for 1/8 replication, 2^{10} study.

Type of Variables	Number of Levels	Description of Levels
1. Mixture Design Variables		
Gradation, Overall gradation	2	Dense, open
Aggregate stability, Crush count	2	Angular crushed, gravel with 50% crush
Stripping potential	2	TSR > 0.95, 0.60 <TSR<0.70
Temperature susceptibility	2	High and low pen index
Aging susceptibility	2	High and low aging index
2. Mixture Design Acceptance Criteria		
Design asphalt content	NA	Dependent on mix design
Design air voids	NA	Dependent on mix design
Design VMA	NA	Dependent on mix design
Design stability	NA	Dependent on mix design
Design flow	NA	Dependent on mix design
Tensile strength ratio	NA	Dependent on mix design
3. M&C Acceptance Variables		
/ Gradation (% < No. 200)	2	Target, target +3%
/ Asphalt content	2	Target, target +1%
/ Air voids	2	Target, target +4%
Thickness (yield)	NA	Not mix related
Roughness	NA	Not mix related
4. Conditioning Effects		
Aging, Asphalt hardening	2	None, lab aged
Moisture, Water sensitivity	2	None, lab conditioned

Table F-3. Study variables for 1/8 replication, 2^{10} study (continued).

Type of Variables	Number of Levels	Description of Levels
5. Response Variables		
Strength	2 specimens	Diametral tensile strength
Modulus	3 specimens	Diametral tensile modulus
Fatigue parameters	3 specimens	Diametral tensile testing
Creep parameters	3 specimens	Diametral tension testing
Low-temperature fracture	3 specimens	Diametral tension testing

Note: A full factorial study would require 1,024 cells. A 1/8 factorial requires 128 cells. With 2 replicates, this study would require 256 experimental units, or specimens. Direct measurement of creep and fatigue and low-temperature fracture will increase the number of specimens to 1,024.

possible to fit a response surface, which provides the relationships among the control variables and the responses.

Experiment Design Used for this Project

After careful consideration of the experiment design shown in Table F-4, the design was further modified for the laboratory study conducted as part of the project. First, to provide a range in mixture properties, two aggregates and two asphalt cements were selected. The two aggregates gave a relatively stable (high strength), non-adsorptive aggregate and a relatively unstable (low strength), adsorptive aggregate. The two asphalt cements provided an asphalt that was relatively low in aging susceptibility and an aggregate that is relatively susceptible to aging. The two aggregates and asphalts were selected to provide a range in sensitivity to the aging process and to the modulus and strength response variables.

The asphalt content and percent passing No. 200 were selected because they are typical materials variables, are factors that the contractor can control, and are contained in the complex modulus equation that was to be verified (Equation 4). Percent air void was chosen because it is well known that this variable has a large effect on performance and is also contained in the equation that was to be validated. Conditioning was chosen because future specifications will undoubtedly contain some consideration of aged specimens that more accurately replicate service conditions.

The response variables were chosen because they address the major distress mechanisms that were identified earlier. The number of test specimens were kept to an absolute minimum. Two indirect tension specimens and four creep-fatigue specimens were considered marginal in

Table F-4. Laboratory experiment design.

	Type of Variables	Number of Levels	Description of Levels
1.	Asphalt source	2	High and low aging index
2.	Aggregate source	2	High and low stability
3.	Asphalt content	2	Target and below target
4.	Percent passing No. 200	2	Target and above target
5.	Compacted air voids	2	Target and above target
6.	Conditioning	2	Aged and unaged
7.	Response variables		
	• Tensile strength	2 specimens	
	• Complex modulus	4 specimens	
	• Dynamic creep	4 specimens	
	• Fatigue	4 specimens	

Note: Full factorial for five M&C variables gives 5^2 or 32 cells. One-half replication plus all target mixes gives 18 cells. Six specimens per cell, aged and unaged, gives a total of 216 specimens.

number but were the maximum allowable given the resources and time available for the laboratory study.

Diametral test specimens 2.5 inches thick and 4 inches in diameter compacted with Marshall compaction were used in the study. Different air void levels were obtained by adjusting the number of compaction blows. All of the testing was done in the diametral mode using techniques described by Kennedy (7,14). Further details regarding the experimental techniques can be found in Appendix G.

FIELD STUDY

Since performance-based specifications are designed to relate M&C variables to expected pavement performance, the next step after studying the relationships between M&C variables (Level A) and basic material response parameters (Level E) is to determine how the material response parameters affect pavement performance (Level G). The critical constraint in this investigation is that the Level E factors are not expected to affect pavement performance independently of the levels of the Level A variables that generate the Level E response. In other words, pavement performance is expected to be affected by both level-E and level-A factors. For example, pavement performance will probably be affected by the stiffness (modulus) of the asphalt concrete, but how that stiffness is achieved (through lower air voids, stiffer asphalt cement, different aggregate shape) probably will also affect the future pavement performance.

The field experiment design received considerable attention of the project team. Specifically, in order to keep the size of the experiment plan within reason, the number of factors that can be studied must be severely limited and the experimental control must be very high. Otherwise, the effects of all possible factors that influence pavement performance will be confounded with each other, and very little understanding of the process of pavement deterioration will result. This means that it will not be possible to use existing pavement sections, since an old pavement, regardless of the quality of the records kept of as-built construction and subsequent maintenance, would not have been built to a specific level within a designed experiment. Also, the study must be as localized as is practical so that nearly identical traffic loadings and environmental conditions will be applied on all of the pavement sections. The use of a fractional factorial experiment, with different cells of that experiment occurring in different areas with different temperature and moisture environments, different site conditions, and different traffic conditions, would require that many more factors be considered than is practical.

For the reasons discussed above, the experiment plan for the field study must concentrate on controlled variation of the M&C factors (Level A), with the secondary objective of providing a wide range of material responses (Level E) that can be measured and considered statistically as covariates. In this way, the principal factors of interest (Level A) will drive the experiment, while the effects of Level A and Level E factors can be studied in terms of how they relate to pavement performance.

In addition to the variables associated with the asphalt concrete layer, other factors, which can be generally categorized as site conditions, can affect future pavement performance. The wide range of site-related factors that could affect pavement performance is summarized in Table F-5. This table describes the individual factors

Table F-5. Factors evaluated in the development of the experimental plan for a field study.

Type of Factor	Example Pavement Design/ Construction Considerations	Assessment of Factors	Data Elements in Study
1. Level of Service	Interstate vs. farm-to-market	Not studied, assume high level	None
2. Traffic	High volume vs. low volume	Important factor that is quantified by traffic surveys; assume high volume	None
3. Geometry	Curve vs. tangent Flat vs. grade Pavement width	Not studied Not studied Covered under channelization Accounted for by drainage/subgrade	None
4. Shoulder	Paved vs. nonpaved Type of paving	Not studied Not studied	None
5. Construction Variability: Uncontrolled/ Nonmeasureable Factors	Type of equipment Expertise of contractor	Parameters observed and approximately noted in database	Qualitative documentation of construction process
6. Performance Constraints	Time to failure Trend in pavement condition indicator(s) over time	Not studied per se, defined in performance model	Observed/design N_f Observed/ permissible levels of pavement condition indicators

Table F-5. Factors evaluated in the development of the experimental plan for a field study (continued).

Type of Factor	Example Pavement Design/ Construction Considerations	Assessment of Factors	Data Elements in Study
7. Environmental	Temperature Moisture Rate of temperature change	Accounted for by different SHRP regions	None
8. Drainage	Type of drainage Internal vs. edge drains Cut vs. fill Rate of water removal	Important design factor, difficult to quantify	Adequacy of drainage: (good, typical, poor)
9. Subgrade	Type-soil classification Plastic vs. nonplastic Gradation Permeability Shrink/swell potential Freeze/thaw susceptibility	Summarized by stiffness of soil and moisture sensitivity	Three levels of stiffness Two levels of moisture sensitivity
	Degree of saturation during service Subgrade characteristics	Accounted for by drainage	
	CBR Resilient modulus Repeated plastic deformation Strength Variability	Monitored during time of construction	
	Construction-related w % @ compaction Percentage density achieved Type of compaction equipment Soil fabric	Constructed at target values	

Table F-5. Factors evaluated in the development of the experimental plan for a field study (continued).

Type of Factor	Example Pavement Design/ Construction Considerations	Assessment of Factors	Data Elements in Study
10. Base/Subbase	Bound vs. unbound Type of binder (rigid vs. flexible) -hot AC -emulsions -cement -lime/fly ash Type of construction -in place -plant Fundamental response -fatigue parameters -modulus -repeated plastic deformation Construction-related -thickness -w % @ compaction -percent density -type of compaction equipment -curing time	1. Classified as rigid, flexible, unbound open, or unbound dense 2. Characterized base properties in the laboratory	Four levels for base type (rigid, flexible, unbound open, unbound dense Two stiffness levels per base type
11. AC Mix	Fundamental response -fatigue parameters -modulus -creep compliance	-Measured fundamental response characteristics in laboratory and related to H&C factors	Five level E factors (fatigue parameters,

Table F-5. Factors evaluated in the development of the experimental plan for a field study (continued).

Type of Factor	Example Pavement Design/ Construction Considerations	Assessment of Factors	Data Elements in Study
	-strength -low-temperature characteristics Type of mix -hot-mix AC -recycled -emulsions Construction-related factors -density -thickness -compaction -roughness Plant-related factors -aggregates -aggregate gradation -additive -asphalt grade	Back-calculated in situ modulus from deflection measurements	modulus, creep compliance, strength, low-temperature fracture character-istics)
12. Maintenance	High-maintenance vs. low-maintenance	Important factor affecting service life	Three levels of maintenance (none, typical, high)

and indicates whether it will be possible to study their effects during an experimental field investigation. Two basic approaches can be taken for studying the effects of the variables. An investigation can be made by designing a 2^n factorial experiment where all independent variables are placed at two levels. This type of experiment would be exploratory in nature because discovery of the relative importance of the variables and their interactions is desired. However, since each variable is considered at only two levels, very little can be said about the nature of the true response function.

If the experiment should be definitive over a reasonable range of values for the independent variables, then several of the variables must be used at three levels. Only if the effect of a variable can safety be assumed to be linear in the range of interest can it be studied at only two levels. Accordingly, a high level of engineering judgment must be exercised in order to decide if two or three levels will be used for a variable. The rule, if the resulting data base will be used in developing performance-based payment schedules, must be to use three levels unless there is a good reason to assume that two will be sufficient.

After the variables have been listed and the number of levels for the variables has been chosen, a $2^n \times 3^m$ factorial experiment will result. This experiment will probably be too large to conduct; for example, if n=5 and m=6, then the full factorial experiment would require 23,328 experimental units. An experiment of this size would require the use of a fractional factorial, where it would be possible to estimate the main effects and two-way interactions with no aliases among them. However, with a large experiment, even a fractional factorial may become too large to implement practically.

On the basis of the judgment and experience of the research team, the experimental design shown in Table F-6 is recommended for the field study of pavement performance as it relates to M&C variables and the development of PRS specifications. The independent variables were selected from among those being studied as part of the laboratory investigation. The results of the laboratory study will provide important information concerning which levels should be chosen for the independent variables in order to obtain a satisfactory response surface for investigation.

The chosen factorial experiment is based on a design of a 2^n experiment, which will provide an exploratory investigation to obtain information on which variables could be studied in more detail in a future three-level factorial investigation. The inclusion of a three-level experiment at this stage is impractical, given the large number of factors still under consideration.

The variables related to site condition were reduced to one variable, stiffness of the pavement foundation, in an effort to reduce the number of factors to a manageable level, and with the idea that this factor is a rational means of expressing the structural support conditions that affect the upper asphalt layer. The site condition factor (pavement foundation stiffness) will vary along the roadway, even if it is carefully controlled during construction. For this reason, special attention should be paid to quantifying the in situ variation of foundation stiffness by using deflection-measuring devices. This

Table F-6. Factors included in the experimental plan for exploratory field study.

Factor	Levels	No. of Levels
A. Mixture Design Variables		
1. Gradation	Dense, open	2
2. Aggregate stability, Crush count	Angular crushed, Gravel with 50% crush count	2
3. Absorption	Absorptive, nonabsorptive	2
4. Aging susceptibility	High, low	2
B. M&C Acceptance Variables		
1. Gradation (%<No. 200)	Target, +10% of target	2
2. Asphalt content	Target, -20% of target	2
3. Air voids	Target, -5% of target	2
C. Construction-related Variables		
1. Initial roughness	High, low	2
2. AC thickness	Thick, thin	2
D. Subgrade Support	Stiff, soft	2

quantified variation could then be used in the analysis of the variation of pavement performance.

In all sections, the pavement performance will be monitored by quantifying the pavement condition indicators. Traffic will be quantified according to volume, type of vehicle, and axle load. Because of the extreme influence of traffic upon pavement performance, the pavement sections should be constructed within one section of a high-volume roadway. If a 500-ft section is chosen as the length of the experimental unit, then the experiment shown in Table F-6 would require approximately 15 lane-miles of roadway. If this procedure is followed in an area of uniform traffic (i.e., no major traffic entrances or exits), the effect of traffic will be constant and will not be confounded with the effects of the study variables. The experimental plan shown in Table F-6 would be repeated in each of four regional areas (i.e., freeze/wet, freeze/dry, no freeze/wet, no freeze/dry).

The project team realizes that this experimental plan poses considerable problems in terms of construction and implementation of the study. However, there is no simple procedure for conducting an experiment in which a very large number of factors affects the response variables of interest while keeping the size of the experiment within practical limits.

APPENDIX G—LABORATORY STUDY

SPECIMEN PREPARATION AND TEST RESULTS

Two aggregate and two asphalt sources were used in the laboratory study, generating four different mixtures. These mixtures were then varied by: removing asphalt cement; adding dust/percent passing No. 200; and reducing the load of compaction to increase the as-compacted air voids. To produce aged specimens, the completed specimens were aged in a 140 °F forced-draft oven for 12 days.

The Marshall mix design procedure was used to establish the optimum asphalt content for the four aggregate-asphalt cement combinations. The properties of the four design or target mixes are given in Table 3 (Chapter 2). It is important to note that in order to develop the complete relationship, treatment levels lying to both sides of the target values of the dust content, air voids, and asphalt content should be selected. However, for this limited demonstration study, only two of the possible three treatment levels for each of the three M&C variables were selected. Furthermore, the study performed was a half replication of the complete factorial experiment, meaning that only one-half of the 32 possible treatment combinations was examined.

For the percent passing the No. 200 sieve, the treatment levels were established as the target percentage as per the mix design and this target percentage plus 40 percent. The two asphalt content treatment levels selected were the optimum and 15 percent below the optimum. The treatment levels for air voids were selected as the mix design air voids at the optimum asphalt content and this target air voids plus 4 percent additional voids. To determine the required number of Marshall hammer blows to provide the +4 percent air voids, an experiment for each aggregate-asphalt cement combination was conducted to generate a curve of specimen air voids versus number hammer blows.

Upon completion of the mixture designs and selection of treatment levels, sufficient specimens were prepared for each mixture treatment combination (experiment cell). The treatment combinations used in the study are shown in Table G-1. The one-half replication results in 16 experimental cells. Including all 4 target designs resulted in 18 treatment combinations. Twelve specimens for each mixture were required per treatment combination, producing 216 test specimens. Six specimens received no special conditioning. Six were subjected to the accelerated aging treatment. Moisture effects and test temperature were not considered for this study.

The six specimens for the no conditioning or aging experiment for each mixture were assigned randomly to provide two specimens for the determination of tensile strength and four specimens for repeated diametral testing.

It was discovered during the early stages of the specimen preparation that the method used to mix the aggregate and asphalt cement had a dramatic influence on the air voids of the compacted specimens. The original mix designs incorporating the two aggregate and the two asphalt cement sources were initially performed in a 5-qt Hobart mechanical mixer using a standard wire mixing whip. To prepare the necessary 12 specimens per mixture, a 20-qt Hobart mechanical mixer was initially used. It was deemed desirable to prepare all 12 specimens at one time to avoid batch-to-batch variations which would be encountered

Table G-1. Laboratory test results.

No	Sample No	Lab No	Agg Type	AC Type	% Dust	% AC	% Air Voids	Aging	Nf	Fat Stress	gnu	alpha	Witzak $\lvert E^* \rvert$	Measure $\lvert E^* \rvert$	Tens Stress	Tens Strength	Blows	% Dust	% AC	% Air Voids	Bulk Sp Gr
1	101	1 -1	0	0	0	0	0	0								222	75	5.4	5.8	3.7	2.446
2	102	1 -2	0	0	0	0	0	0	39550	84	.453	.579	485000	453000	21.0		75	5.4	5.8	3.2	2.460
3	103	1 -3	0	0	0	0	0	1								261	75	5.4	5.8	3.2	2.460
4	104	1 -4	0	0	0	0	0	0	52930	86	.414	.498	493000	475000	21.5		75	5.4	5.8	3.0	2.465
5	105	1 -5	0	0	0	0	0	1								240	75	5.4	5.8	3.0	2.465
6	106	1 -6	0	0	0	0	0	0									75	5.4	5.8	3.1	2.462
7	107	1 -7	0	0	0	0	0	0	7590	130	.399	.402	462000	462000	32.5		75	5.4	5.8	3.8	2.444
8	108	1 -8	0	0	0	0	0	0	26680	127	.262	.397	N/A	[736000]	31.8		75	5.4	5.8	3.8	2.444
9	109	1 -9	0	0	0	0	0	1	11110	154	.753	.601	N/A	597000	38.5		75	5.4	5.8	4.0	2.439
10	110	1 -10	0	0	0	0	0	1	32670	113	.126	.383	N/A	426000	28.3		75	5.4	5.8	3.1	2.462
11	111	1 -11	0	0	0	0	0	1									75	5.4	5.8	3.1	2.462
12	112	1 -12	0	0	0	0	0	1	8270	175	.177	.255	N/A	[743000]	43.8		75	5.4	5.8	3.4	2.455
13	201	2 -1	0	0	0	1	1	1								189	28	5.4	5.0	9.8	2.325
14	202	2 -2	0	0	0	1	1	0	20450	67	.488	.418	327000	398000	16.8		28	5.4	5.0	9.9	2.321
15	203	2 -3	0	0	0	1	1	0								147	28	5.4	5.0	10.0	2.320
16	204	2 -4	0	0	0	1	1	1	27960	100	.622	.487	N/A	612000	25.0		28	5.4	5.0	10.0	2.318
17	205	2 -5	0	0	0	1	1	0	16540	76	.088	.279	314000	311000	19.0		28	5.4	5.0	10.4	2.310
18	206	2 -6	0	0	0	1	1	1								185	28	5.4	5.0	10.5	2.305
19	207	2 -7	0	0	0	1	1	1	13960	123	.716	.446	N/A	804000	30.8		28	5.4	5.0	10.1	2.316
20	208	2 -8	0	0	0	1	1	0	6220	100			317000	349000	25.0		28	5.4	5.0	10.3	2.311
21	209	2 -9	0	0	0	1	1	1	8250	146	.224	.356		[360000]	36.5	1	28	5.4	5.0	10.0	2.318
22	210	2 -10	0	0	0	1	1	1	41120	92	.523	.506		606000	23.0	0	28	5.4	5.0	9.8	2.324
23	211	2 -11	0	0	0	1	1	0								168	28	5.4	5.0	9.8	2.325
24	212	2 -12	0	0	0	1	1	0	54580	59	.675	.524	327000	428000	14.8		28	5.4	5.0	9.9	2.323
25	301	3 -1	0	0	1	1	0	1	18750	167	.188	.465	N/A	[485000]	41.8		75	7.5	5.0	5.2	2.431
26	302	3 -2	0	0	1	1	0	1									75	7.5	5.0	5.3	2.427
27	303	3 -3	0	0	1	1	0	0								223	75	7.5	5.0	5.3	2.426
28	304	3 -4	0	0	1	1	0	0	50410	103	1.179	.686	571000	586000	25.8		75	7.5	5.0	4.1	2.458
29	305	3 -5	0	0	1	1	0	1	55420	135	.561	.553	N/A	871000	33.8		75	7.5	5.0	4.4	2.449
30	306	3 -6	0	0	1	1	0	0								234	75	7.5	5.0	4.4	2.450
31	307	3 -7	0	0	1	1	0	0	6720	167	.721	.552	553000	619000	41.8		75	7.5	5.0	4.5	2.448
32	308	3 -8	0	0	1	1	0	1	36080	153	.234	.473	N/A	606000	38.3		75	7.5	5.0	4.3	2.452
33	309	3 -9	0	0	1	1	0	0	15130	140	.585	.558	567000	559000	35.0		75	7.5	5.0	4.2	2.455
34	310	3 -10	0	0	1	1	0	1	12430	186	.383	.475	N/A	676000	46.5		75	7.5	5.0	4.1	2.457
35	311	3 -11	0	0	1	1	0	1								264	75	7.5	5.0	4.1	2.458
36	312	3 -12	0	0	1	1	0	0	15530	123	.230	.421	563000	[385000]	30.8		75	7.5	5.0	4.3	2.452
37	401.	4 -1	0	1	1	0	0	1	90380	118	.170	.571	N/A	509000	29.5		75	7.5	5.8	2.1	2.474
38	402	4 -2	0	1	1	0	0	0	27290	124	.379	.565	546000	345000	31.0		75	7.5	5.8	2.3	2.470
39	403	4 -3	0	1	1	0	0	0	104190	93	.177	.510	546000	374000	23.3		75	7.5	5.8	2.3	2.470
40	404	4 -4	0	1	1	0	0	1								211	75	7.5	5.8	1.8	2.482
41	405	4 -5	0	1	1	0	0	0	162350	73	.196	.402	559000	[538000]	18.3		75	7.5	5.8	2.0	2.475
42	406	4 -6	0	1	1	0	0	0								176	75	7.5	5.8	1.9	2.478
43	407	4 -7	0	1	1	0	0	1	18400	156	.324	.494	N/A	430000	39.0		75	7.5	5.8	1.9	2.478
44	408	4 -8	0	1	1	0	0	0								177	75	7.5	5.8	1.9	2.479
45	409	4 -9	0	1	1	0	0	1								222	75	7.5	5.8	2.2	2.471
46	410	4 -10	0	1	1	0	0	1	6010	168	.188	.342	N/A	[329000]	42.0		75	7.5	5.8	1.5	2.484
47	411	4 -11	0	1	1	0	0	0	13100	98	.074	.580	568000	363000	24.5		75	7.5	5.8	1.8	2.482
48	412	4 -12	0	1	1	0	0	1	31870	129	.430	.521	N/A	536000	32.3		75	7.5	5.8	1.4	2.492

Table G-1. Laboratory test results (continued).

No	Sample No	Lab No	Agg Type	AC Type	% Dust	% AC	% Air Voids	Aging	Nf	Fat Stress	gnu	alpha	Witzak $\lvert E^* \rvert$	Measure $\lvert E^* \rvert$	Tens Stress	Tens Strength	Blows	% Dust	% AC	% Air Voids	Bulk Sp Gr
49	501	5 -1	1	1	0	0	0	1	11420	127	.475	.491	N/A	389000	31.8		75	5.9	6.6	3.1	2.284
50	502	5 -2	1	1	0	0	0	0	47550	65	.623	.539	411000	[312000]	16.3		75	5.9	6.6	3.3	2.281
51	503	5 -3	1	1	0	0	0	1	23700	104	.569	.557	N/A	380000	26.0		75	5.9	6.6	3.3	2.281
52	504	5 -4	1	1	0	0	0	0	16420	78	.324	.478	424000	198000	19.5		75	5.9	6.6	2.9	2.290
53	505	5 -5	1	1	0	0	0	0								141	75	5.9	6.6	3.2	2.282
54	506	5 -6	1	1	0	0	0	1	54200	91	.210	.443	N/A	402000	22.8		75	5.9	6.6	3.2	2.282
55	507	5 -7	1	1	0	0	0	1								167	75	5.9	6.6	2.9	2.291
56	508	5 -8	1	1	0	0	0	1	8260	145	.670	.560	N/A	408000	36.3		75	5.9	6.6	3.4	2.277
57	509	5 -9	1	1	0	0	0	1								163	75	5.9	6.6	2.9	2.289
58	510	5 -10	1	1	0	0	0	0	3460	103			430000	200000	25.8		75	5.9	6.6	2.7	2.295
59	511	5 -11	1	1	0	0	0	0	16590	70	.489	.464	437000	232000	17.5		75	5.9	6.6	2.5	2.300
60	512	5 -12	1	1	0	0	0	0								134	75	5.9	6.6	2.5	2.302
61	601	6 -1	0	0	1	0	1	1								165	28	7.5	5.8	5.9	2.378
62	602	6 -2	0	0	1	0	1	0	5240	112	.558	.371	433000	401000	28.0		28	7.5	5.8	5.8	2.382
63	603	6 -3	0	0	1	0	1	0								140	28	7.5	5.8	6.2	2.371
64	604	6 -4	0	0	1	0	1	1								165	28	7.5	5.8	5.7	2.384
65	605	6 -5	0	0	1	0	1	0	29190	59	.871	.496	429000	392000	14.8		28	7.5	5.8	5.9	2.380
66	606	6 -6	0	0	1	0	1	1	20750	98	.406	.403	N/A	508000	24.5		28	7.5	5.8	5.7	2.384
67	607	6 -7	0	0	1	0	1	1	13160	122			N/A		30.5		28	7.5	5.8	5.5	2.389
68	608	6 -8	0	0	1	0	1	0	15540	77			440000	384000	19.3		28	7.5	5.8	5.6	2.386
69	609	6 -9	0	0	1	0	1	1	51580	88	.304	.481	N/A	522000	22.0		28	7.5	5.8	5.7	2.385
70	610	6 -10	0	0	1	0	1	1	10510	135	.717	.485	N/A	470000	33.8		28	7.5	5.8	5.6	2.386
71	611	6 -11	0	0	1	0	1	0	94980	50	.340	.448	440000	467000	12.5		28	7.5	5.8	5.6	2.387
72	612	6 -12	0	0	1	0	1	0								148	28	7.5	5.8	5.8	2.381
73	701	7 -1	0	1	0	1	0	0								178	75	5.4	5.0	5.5	2.433
74	702.	7 -2	0	1	0	1	0	1	10000	193	.180	.418	N/A	446000	48.3		75	5.4	5.0	5.5	2.433
75	703	7 -3	0	1	0	1	0	0	4650	175	.045	.516	447000	422000	43.8		75	5.4	5.0	5.5	2.435
76	704	7 -4	0	1	0	1	0	1	15690	172	.102	.372	N/A	[357000]	43.0		75	5.4	5.0	5.4	2.435
77	705	7 -5	0	1	0	1	0	1	33500	150	.197	.464	N/A	506000	37.5		75	5.4	5.0	5.1	2.442
78	706	7 -6	0	1	0	1	0	0	12260	152	.320	.542	465000	354000	38.0		75	5.4	5.0	4.9	2.448
79	707	7 -7	0	1	0	1	0	1								221	75	5.4	5.0	4.7	2.454
80	708	7 -8	0	1	0	1	0	0									75	5.4	5.0	4.9	2.447
81	709	7 -9	0	1	0	1	0	0	54090	106	.377	.514	454000	504000	26.5		75	5.4	5.0	5.2	2.440
82	710	7 -10	0	1	0	1	0	0	24620	128			458000	429000	32.0		75	5.4	5.0	5.1	2.443
83	712	7 -12	0	1	0	1	0	1	23750	137	.110	.361	N/A	490000	34.3		75	5.4	5.0	6.1	2.416
84	713	7 -13	0	1	0	1	0	1								258	75	5.4	5.0	6.1	2.416
85	801	8 -1	1	0	1	0	0	0								150	75	8.3	6.6	1.5	2.327
86	802	8 -2	1	0	1	0	0	0	34740	76	.481	.429	554000	362000	19.0		75	8.3	6.6	1.5	2.328
87	803	8 -3	1	0	1	0	0	1								151	75	8.3	6.6	1.7	2.324
88	804	8 -4	1	0	1	0	0	1	9880	124	.194	.364	N/A	[288000]	31.0		75	8.3	6.6	1.1	2.337
89	805	8 -5	1	0	1	0	0	0								124	75	8.3	6.6	1.3	2.332
90	806	8 -6	1	0	1	0	0	1	19430	101	.246	.347	N/A	387000	25.3		75	8.3	6.6	1.3	2.322
91	807	8 -7	1	0	1	0	0	0	4410	122	1.499	.503	559000	350000	30.5		75	8.3	6.6	1.4	2.329
92	808	8 -8	1	0	1	0	0	0	16200	89	.495	.442	563000	303000	22.3		75	8.3	6.6	1.3	2.333
93	809.	8 -9	1	0	1	0	0	1								194	75	8.3	6.6	1.3	2.332
94	810	8 -10	1	0	1	0	0	1	40550	91	.344	.374	N/A	[694000]	22.8		75	8.3	6.6	1.0	2.340
95	811	8 -11	1	0	1	0	0	0	106610	66	.862	.557	577000	313000	16.5		75	8.3	6.6	1.0	2.340
96	812	8 -12	1	0	1	0	0	1	7320	134	.659	.442	N/A	469000	33.5		28	8.3	6.6	1.2	2.334

Table G-1. Laboratory test results (continued).

No	Sample No	Lab No	Agg Type	AC Type	Dust	% AC	% Air Voids	Aging	Nf	Fat Stress	gnu	alpha	Witzak |E*|	Measured |E*|	Ten Stress	Ten Strength	Blows	% Dust	% AC	% Air Voids	Bulk Sp Gr
97	901	9 -1	0	1	0	0	1	1	33560	88	.222	.408	N/A	[292000]	22.0		28	5.4	5.8	7.8	2.340
98	902	9 -2	0	1	0	0	1	0	55170	47	.241	.352	322000	336000	11.8		28	5.4	5.8	7.7	2.344
99	903	9 -3	0	1	0	0	1	1								142	28	5.4	5.8	8.0	2.337
100	904	9 -4	0	1	0	0	1	0								117	28	5.4	5.8	7.2	2.357
101	905	9 -5	0	1	0	0	1	0	6310	102	.361	.189	328000	343000	25.5		28	5.4	5.8	7.5	2.350
102	906	9 -6	0	1	0	0	1	1								154	28	5.4	5.8	7.8	2.243
103	907	9 -7	0	1	0	0	1	1	11000	·125	.339	.464	N/A	446000	31.3		28	5.4	5.8	8.0	2.338
104	908	9 -8	0	1	0	0	1	1	24040	112	.259	.433	N/A	393000	28.0		28	5.4	5.8	8.3	2.329
105	909	9 -9	0	1	0	0	1	0								124	28	5.4	5.8	7.7	2.344
106	910	9 -10	0	1	0	0	1	0	25790	60	.430	.408	312000	292000	15.0		28	5.4	5.8	8.1	2.344
107	911	9 -11	0	1	0	0	1	1	30150	100	.219	.431	N/A	[336000]	25.0		28	5.4	5.8	7.8	2.342
108	912	9 -12	0	1	0	0	1	0	11240	76	.800	.523	307000	238000	19.0		28	5.4	5.8	8.3	2.329
109	1001	10 -1	1	0	0	1	0	0	101110	44	.831	.583	437000	410000	11.0		75	5.9	5.7	5.0	2.265
110	1002	10 -2	1	0	0	1	0	0								169	75	5.9	5.7	5.1	2 263
111	1003	10 -3	1	0	0	1	0	1	39560	105	.542	.566	N/A	475000	26.3		75	5.9	5.7	5.3	2.259
112	1004	10 -4	1	0	0	1	0	1								242	75	5.9	5.7	4.6	2.275
113	1005	10 -5	1	0	0	1	0	1	62200	96	.246	.501	N/A	[391000]	24.0		75	5.9	5.7	5.3	2.259
114	1006	10 -6	1	0	0	1	0	0	11270	114	.489	.472	430000	462000	28.5		75	5.9	5.7	5.2	2.262
115	1007	10 -7	1	0	0	1	0	1	17610	122	.298	.416	N/A	491000	30.5		75	5.9	5.7	4.8	2.271
116	1008	10 -8	1	0	0	1	0	0	28830	68	2.965	.838	423000	[538000]	17.0		75	5.9	5.7	5.4	2.256
117	1009	10 -9	1	0	0	1	0	0								173	75	5.9	5.7	5.5	2.255
118	1010	10 -10	1	0	0	1	0	0	8010	97	.437	.478	430000	[242000]	24.3		75	5.9	5.7	5.2	2.261
119	1011	10 -11	1	0	0	1	0	1								212	75	5.9	5.7	5.2	2.261
120	1012	10 -12	1	0	0	1	0	1	8120	151	.592	.432	N/A	658000	37.8		75	5.9	5.7.	5.5	2.256
121	1101	11 -1	1	0	0	0	1	0								117	16	5.9	6.6	6.8	2.192
122	1102	11 -2	1	0	0	0	1	1								153	16	5.9	6.6	6.9	2.189
123	1103	11 -3	1	0	0	0	1	1	30420	72	.600	.459	N/A	457000	18.0		16	5.9	6.6	6.7	2.194
124	1104	11 -4	1	0	0	0	1	1	7320	109	1.092	.517	N/A	372000	27.3		16	5.9	6.6	6.5	2.200
125	1105	11 -5	1	0	0	0	1	1								128	16	5.9	6.6	6.6	2.196
126	1106	11 -6	1	0	0	0	1	0	2240	93	1.857	.476	339000	349000	23.3		16	5.9	6.6	6.3	2.203
127	1107	11 -7	1	0	0	0	1	0	10460	52	.733	.354	313000	314000	13.0		16	5.9	6.6	7.3	2.181
128	1108	11 -8	1	0	0	0	1	0								98	16	5.9	6.6	7.2	2.183
129	1109	11 -9	1	0	0	0	1	1	12760	87	.685	.444	N/A	395000	21.8		16	5.9	6.6	7.3	2.180
130	1110	11 -10	1	0	0	0	1	0	26830	34	.586	.393	310000	[210000]	8.5		16	5.9	6.6	7.4	2.178
131	1111	11 -11	1	0	0	0	1	1	27390	61	.667	.490	N/A	[260000]	15.3		16	5.9	6.6	6.9	2.189
132	1112	11 -12	1	0	0	0	1	0	1640	126			336000		31.5		16	5.9	6.6	6.4	2.202
133	1201	12 -1	1	1	1	1	0	1								184	75	8.3	5.7	3.4	2.302
134	1202	12 -2	1	1	1	1	0	0	72700	87	.406	.561	527000	356000	21.8		75	8.3	5.7	3.4	2.301
135	1203	12 -3	1	1	1	1	0	1	70000	119	.357	.489	N/A	[798000]	29.8		75	8.3	5.7	3.7	2.296
136	1204	12 -4	1	1	1	1	0	1	64900	107	.045	.332	N/A	477000	26.8		75	8.3	5.7	2.8	2.318
137	1205	12 -5	1	1	1	1	0	0								164	75	8.3	5.7	2.9	2.314
138	1206	12 -6	1	1	1	1	0	0	131050	62	.483	.536	548000	[497000]	15.5		75	8.3	5.7	2.9	2.313
139	1207	12 -7	1	1	1	1	0	0	10610	107	.085	.255	575000	300000	26.8		75	8.3	5.7	2.3	2.330
140	1208	12 -8	1	1	1	1	0	1	16830	140	.152	.393	N/A	359000	35.0		75	8.3	5.7	2.3	2.331
141	1209	12 -9	1	1	1	1	0	0								167	75	8.3	5.7	2.3	2.331
142	1210	12 -10	1	1	1	1	0	1								208	75	8.3	5.7	2.3	2.329
143	1211	12 -11	1	1	1	1	0	0	4950	140			575000	320000	35.0		75	8.3	5.7	2.3	2.328
144	1212	12 -12	1	1	1	1	0	1	32560	130	.305	.463	N/A	612000	32.5		75	8.3	5.7	2.5	2.324

Table G-1. Laboratory test results (continued).

No	Sample No	Lab No	Agg Type	AC Type	Dust	% AC	% Air Voids	Aging	Nf	Fat Stress	gnu	alpha	Witzak |E*|	Measured |E*|	Ten Stress	Ten Strength	Blows	% Dust	% AC	% Air Voids	Bulk Sp Gr
145	1301	13 -1	1	1	1	0	1	1	29390	87	.618	.561	N/A	300000	21.8		75	8.3	6.6	4.5	2.246
146	1302	13 -2	1	1	1	0	1	0								110	75	8.3	6.6	4.2	2.251
147	1303	13 -3	1	1	1	0	1	0	34820	59	.910	.545	419000	269000	14.8		75	8.3	6.6	4.4	2.248
148	1304	13 -4	1	1	1	0	1	1	37640	76	.333	.427	N/A	336000	19.0		75	8.3	6.6	3.2	2.275
149	1305	13 -5	1	1	1	0	1	1								135	75	8.3	6.6	3.3	2.273
150	1306	13 -6	1	1	1	0	1	0	6170	81	.238	.353	457000	265000	20.3		75	8.3	6.6	3.3	2.273
151	1307	13 -7	1	1	1	0	1	0								99	75	8.3	6.6	4.9	2.235
152	1308	13 -8	1	1	1	0	1	1	12440	99	.569	.512	N/A	280000	24.8		75	8.3	6.6	4.6	2.242
153	1309	13 -9	1	1	1	0	1	1								142	75	8.3	6.6	4.5	2.245
154	1310	13 -10	1	1	1	0	1	0	9760	71	.326	.325	443000	233000	17.8		75	8.3	6.6	3.7	2.264
155	1311	13 -11	1	1	1	0	1	0	24750	52	.590	.466	461000	214000	13.0		75	8.3	6.6	3.2	2.275
156	1312	13 -12	1	1	1	0	1	1	6970	113	.886	.554	N/A	282000	28.3		75	8.3	6.6	3.8	2.262
157	1401	14 -1	1	1	0	1	1	1								124	75	5.9	5.7	10.5	2.149
158	1402	14 -2	1	1	0	1	1	1	63310	54	.343	.459	N/A	[242000]	13.5		75	5.9	5.7	10.3	2.154
159	1403	14 -3	1	1	0	1	1	0								100	75	5.9	5.7	10.4	2.152
160	1404	14 -4	1	1	0	1	1	0	2120	90	1.452	.416	286000	[331000]	22.5		75	5.9	5.7	9.7	2.170
161	1405	14 -5	1	1	0	1	1	1	25010	71	.614	.441	N/A	435000	17.8		75	5.9	5.7	9.6	2.172
162	1406	14 -6	1	1	0	1	1	0								98	75	5.9	5.7	9.4	2.175
163	1407	14 -7	1	1	0	1	1	0	20640	39	.825	.449	290000	[192000]	9.8		75	5.9	5.7	9.5	2.173
164	1408	14 -8	1	1	0	1	1	0	5800	67	1.071	.462	283000	247000	16.8		75	5.9	5.7	9.8	2.167
165	1409	14 -9	1	1	0	1	1	1	56460	56	.107	.321	N/A	323000	14.0		75	5.9	5.7	10.0	2.162
166	1410	14 -10	1	1	0	1	1	1								122	75	5.9	5.7	10.3	2.155
167	1411	14 -11	1	1	0	1	1	0	35900	32	.649	.398	286000	272000	8.0		75	5.9	5.7	9.7	2.170
168	1412	14 -12	1	1	0	1	1	1	10210	90	.319	.395	N/A	[266000]	22.5		75	5.9	5.7	10.3	2.154
169	1501	15 -1	1	0	1	1	1	0									75	8.3	5.7	7.5	2.217
170	1502	15 -2	1	0	1	1	1	1	35700	78	.602	.523	N/A	403000	19.5		75	8.3	5.7	7.3	2.221
171	1503	15 -3	1	0	1	1	1	1								172	75	8.3	5.7	7.4	2.218
172	1504	15 -4	1	0	1	1	1	1	10590	109	.503	.360	N/A	589000	27.3		75	8.3	5.7	7.3	2.221
173	1505	15 -5	1	0	1	1	1	0	27770	61	1.209	.595	425000	291000	15.3		75	8.3	5.7	6.7	2.235
174	1506	15 -6	1	0	1	1	1	0								124	75	8.3	5.7	7.3	2.222
175	1507	15 -7	1	0	1	1	1	1	22800	90	.548	.430	N/A	544000	22.5		75	8.3	5.7	7.5	2.216
176	1508	15 -8	1	0	1	1	1	0	6440	78	1.159	.482	398000	310000	19.5		75	8.3	5.7	7.5	2.216
177	1509	15 -9	1	0	1	1	1	1								165	75	8.3	5.7	7.4	2.218
178	1510	15 -10	1	0	1	1	1	0	79130	33			415000	355000	8.3		75	8.3	5.7	7.0	2.229
179	1511	15 -11	1	0	1	1	1	0								131	75	8.3	5.7	7.0	2.229
180	1512	15 -12	1	0	1	1	1	1	7750	129			N/A		32.3		75	8.3	5.7	6.9	2.231
181	1601	16 -1	0	1	1	1	1	1	34300	114	.202	.446	N/A	[354000]	28.5		75	7.5	5.0	8.3	2.349
182	1602	16 -2	0	1	1	1	1	0	28190	77	.442	.545	389000	[553000]	19.3		75	7.5	5.0	8.3	2.350
183	1603	16 -3	0	1	1	1	1	1								180	75	7.5	5.0	8.3	2.350
184	1604	16 -4	0	1	1	1	1	0	5480	113	.445	.369	386000	371000	28.3		75	7.5	5.0	8.4	2.346
185	1605	16 -5	0	1	1	1	1	1	6790	166	.305	.386	N/A	532000	41.5		75	7.5	5.0	8.4	2.345
186	1606	16 -6	0	1	1	1	1	0								133	75	7.5	5.0	8.4	2.344
187	1607	16 -7	0	1	1	1	1	0								155	75	7.5	5.0	8.8	2.336
188	1608	16 -8	0	1	1	1	1	1	16000	146	.300	.501	N/A	414000	36.5		75	7.5	5.0	9.2	2.326
189	1609	16 -9	0	1	1	1	1	1								164	75	7.5	5.0	9.0	2.330
190	1610	16 -10	0	1	1	1	1	1	37380	104	.238	.452	N/A	371000	26.0		75	7.5	5.0	7.7	2.364
191	1611	16 -11	0	1	1	1	1	0	250640	32			392000		8.0		75	7.5	5.0	8.2	2.353
192	1612	16 -12	0	1	1	1	1	0	8630	100	.100	.252	386000	349000	25.0		75	7.5	5.0	8.4	2.346

Table G-1. Laboratory test results (continued).

No	Sample No	Lab No	Agg Type	AC Type	% Dust	% AC	% Air Voids	Aging	Nf	Fat Stress	gnu	alpha	Witzak |E*|	Measure |E*|	Ten Stress	Ten Strength	Blows	% Dust	% AC	% Air Voids	Bulk Sp Gr
193	1701	17	-1	0	1	0	0	0	0	93590	69	.188	.450	481000	[247000]	17.3		75	5.4	5.8	2.7	2.472
194	1702	17	-2	0	1	0	0	0	0								169	75	5.4	5.8	2.8	2.470
195	1703	17	-3	0	1	0	0	0	1	13270	160	.210	.442	N/A	414000	40.0		75	5.4	5.8	2.5	2.476
196	1704	17	-4	0	1	0	0	0	0	19070	103	.602	.511	455000	[520000]	25.8		75	5.4	5.8	3.2	2.458
197	1705	17	-5	0	1	0	0	0	1	69670	100	.463	.574	N/A	558000	25.0		75	5.4	5.8	3.6	2.448
198	1706	17	-6	0	1	0	0	0	1								206	75	5.4	5.8	3.9	2.440
199	1707	17	-7	0	1	0	0	0	0	8010	128	.901	.510	444000	368000	32.0		75	5.4	5.8	3.7	2.446
200	1708	17	-8	0	1	0	0	0	1	29760	127	.305	.540	N/A	427000	31.8		75	5.4	5.8	4.2	2.433
201	1709	17	-9	0	1	0	0	0	0								178	75	5.4	5.8	3.7	2.445
202	1710	17	-10	0	1	0	0	0	1	14000	148	.619	.455	N/A	442000	37.0		75	5.4	5.8	3.8	2.444
203	1711	17	-11	0	1	0	0	0	1								202	75	5.4	5.8	3.6	2.448
204	1712	17	-12	0	1	0	0	0	0	29630	84	.399	.460	430000	454000	21.0		75	5.4	5.8	4.1	2.436
205	1801	18	-1	1	0	0	0	0	0	70590	62	.917	.597	482000	360000	15.5		75	5.9	6.6	1.9	2.308
206	1802	18	-2	1	0	0	0	0	1								211	75	5.9	6.6	2.2	2.301
207	1803	18	-3	1	0	0	0	0	1	42330	97	.751	.517	N/A	[947000]	24.3		75	5.9	6.6	2.3	2.297
208	1804	18	-4	1	0	0	0	0	0								166	75	5.9	6.6	2.3	2.297
209	1805	18	-5	1	0	0	0	0	0	6820	109	.787	.518	467000	367000	27.3		75	5.9	6.6	2.3	2.296
210	1806	18	-6	1	0	0	0	0	1	37870	87	.519	.532	N/A	365000	21.8		75	5.9	6.6	2.3	2.295
211	1807	18	-7	1	0	0	0	0	1	79120	79	.925	.600	N/A	542000	19.8		75	5.9	6.6	2.3	2.297
212	1808	18	-8	1	0	0	0	0	0								175	75	5.9	6.6	2.5	2.292
213	1809	18	-9	1	0	0	0	0	0	18540	88	.976	.662	460000	353000	22.0		75	5.9	6.6	2.5	2.292
214	1810	18	-10	1	0	0	0	0	1	8970	106	.262	.369	N/A	[256000]	26.5		75	5.9	6.6	2.6	2.290
215	1811	18	-11	1	0	0	0	0	0	14950	77	.194	.358	463000	226000	19.3		75	5.9	6.6	2.4	2.296
216	1812	18	-12	1	0	0	0	0	1								168	75	5.9	6.6	2.7	2.288

by producing four three-specimen batches in the 5-qt mixer. However, the air voids in compacted specimens mixed in the 20-qt mixer were found to be 2 to 3 percent lower than the specimens mixed in the 5-qt mixer regardless of the aggregate and asphalt cement used. All other variables including mixing temperature, compaction temperature, and number of Marshall hammer blows were held constant. This comparison between the small and large mixer was repeated several times with consistent results. Therefore, the use of the large mixer was abandoned and the specimens were manufactured in the 5-qt mixer in batches of three specimens per batch. This effect of mixing method on compacted air voids is cause for concern and should be pursued.

As previously noted, the within-treatment cell specimens were randomly divided into two groups of six specimens. One group, the "unaged" specimens, was conditioned only by permitting the specimens to remain undisturbed at room temperature (70 to 75 °F) for a period of 4 days before testing. This period of time was selected to permit the steric hardening to occur within the specimens. The second set of six specimens, the "aged" specimens, was conditioned in a forced-air oven at 140 °F for a period of 12 days. All testing of the specimens occurred at 77 °F.

The complex modulus, |E*|; creep parameters, nu and alpha; and fatigue parameters, K_2' and N_2, were determined for each specimen through repeated diametral testing conducted at a temperature of 77 °F using a continuous, 10-Hz haversine loading. For each mixture, four unaged and four aged specimens were tested. Within each group of four specimens, a range of stress was applied to result in a range of fatigue life from approximately 5,000 to 100,000 cycles.

The continuous haversine loading was held constant throughout the test period; no conditioning static or cyclic preload was applied. Two LVDT's, one of each side of the specimen, were used to monitor the diametral deflections. A personal computer data acquisition and analysis program was used to collect the load and horizontal deflection data and to calculate the complex modulus defined as the peak-to-peak stress divided by peak-to-peak strain. The program was also prepared to permit the determination of the phase angle between the load and deflection curves. Determining this phase shift would permit the decomposition of the complex modulus into the elastic and viscous components.

Considerable difficulty was encountered with the hardware and software used to discriminate the in-phase and out-of-phase components of |E*|. Given the limited resources available for the laboratory experiment, it was necessary to abandon further measurements of E* and E". Thus, only |E*| was used in the analysis.

The specimen horizontal creep and vertical creep curves for this study were captured with a strip chart recorder. The specimen vertical creep curve was used to monitor the progress of the fatigue life. The fatigue life, N_f, of the specimen was defined as the number of cycles when the observed strain departed from a linear change as a function of the number of loading cycles.

The testing of the four within-treatment cell specimens at different stress levels proved effective in providing reasonable estimates of the fatigue parameters K_2' and N_2 for the aged and unaged specimens, and the results are summarized in Table G-1.

The fatigue parameters, K_2' and N_2, are the intercept and slope, respectively, of an assumed linear relationship between the logarithm of the applied tensile stress and the logarithm of the fatigue life defined by this equation:

$$N_f = K_2' \frac{1}{D\delta}^{N_2} \qquad \text{(G-1)}$$

where

N_f = fatigue life

$D\delta$ = stress difference

K_2' = the antilog of the intercept value of the logarithmic relationship between fatigue life and stress difference

N_2 = slope of the logarithmic relation between fatigue life and stress difference

Kennedy (13) has reported that using the stress difference instead of the applied tensile stress will shift the fatigue relationship in general agreement with the relationship determined by other testing procedures.

The creep parameters, nu and alpha, where determined from the horizontal creep curve for each specimen and are presented for each specimen in Table G-1. The determination of nu and alpha was accomplished using the procedures described in Kennedy and Anagnos (13).

Limited success was achieved in determining the complex modulus and creep parameters by the testing procedure described above. Particular difficulties were observed in the measurement of the horizontal deflections. As previously noted, the horizontal deflections were monitored by two LVDT's, one on each side of the specimen. The deflections were captured by the data acquisition system separately, then combined to provide the measure of the total horizontal deflection. The use of the individual monitoring of the deflections on each side of the specimen is in contrast to the commonly used practice of wiring the two LVDT's through a common signal conditioner. Monitoring these outputs separately revealed, in many instances, that the deflections from one LVDT were many times the magnitude of the deflection of the other LVDT. Also, in a number of instances, the combined deflection curve of the two LVDT outputs appeared reasonable, approximated a haversine, but the individual outputs were badly distorted. Based upon the above observations, the individual LVDT recorded output for each specimen was visually analyzed. If the abnormalities described above were observed, the complex modulus and creep parameters for that specimen were coded as unacceptable and removed from analysis data base.

The researchers believe that the primary reason for the high level of unacceptable data is the difficulty in obtaining adequate seating and alignment between the loading platens and the test specimen and rocking of the test specimen within the test apparatus. The test apparatus used for this study was constructed to the design and tolerances of the apparatus recently developed as part of an FHWA study. This new design promised the elimination of rocking; however, the experiences of the research team was otherwise.

Two unaged and aged specimens from each mixture treatment were tested for tensile strength at 77 °F using the indirect tensile test procedures desired by Kennedy (13). The control strain rate of 2 in per min was used. This procedure worked well, and the data are summarized in Table G-1.

ANALYSIS OF TEST RESULTS

The laboratory experiment was designed and carried out in order to illustrate the role of the experimental method in the development of payment schedules that are based upon predicted pavement performance. This experiment had two purposes, both related to the need to predict fundamental response variables (resilient modulus and the tensile strength were considered in the analysis) by means of the values of certain materials and construction variables.

A model is given in the literature for predicting the resilient modulus of a mixture, Equation 4. This model is based upon a study of the modulus of a series of mixtures weighted toward their design. The question as to the possible usefulness of such predictions when the target values are not achieved must be answered. An experiment was designed and carried out for this purpose. The experiment was designed in such a manner that it also provided a sufficient data base for the construction and evaluation of regression models for predicting the resilient modulus in the event that Equation 4 was not appropriate in this setting.

The experiment was also designed and carried out for the purpose of developing and evaluating regression models based upon materials and construction variables for predicting the tensile strength of mixtures. This experiment was also designed for the purpose of illustrating the procedure and not with the goal of producing a general model for wide use in the future.

In both experiments the materials and construction variables considered were:

1. Asphalts (two levels)
2. Aggregates (two levels)
3. Dust (two levels)
4. Asphalt Content (two levels)
5. Air Voids (two levels)

A complete factorial experiment would require 32 experimental mixtures for each replication. It was felt that no three-way or higher interactions would be important. Since a half replication of this experiment would provide for no confounding of the main effects and two-way interactions, the most efficient use of the experiment would be achieved by designing an experiment with three replications of a half replication of the 2^5 experiment for the resilient modulus experiment and two replications of the same half replication for the tensile strength experiment. Thus, the experiments required the use of 16 distinct mixtures. It was felt that the mixtures at the target levels which were not in this design (two of the targets were and two were not) should also be studied and they were added to the design. Thus, there are 18 distinct mixtures studied in the experiment.

The levels for the asphalt content and the amount of dust (fines) added to the mixture were carefully controlled at their chosen levels. However, the chosen levels for the air voids would only be approximately achieved and their measured values were used in all analyses.

In order to evaluate the usefulness of Witczak's model (Equation 4) for predicting the resilient modulus for these mixtures, it is useful to consider the simple correlation of the Witczak predicted values and the actual measured values. Using the data for all the unaged mixtures this correlation is found to be 0.31. This low value makes it clear that Witczak's method is not generally acceptable for mixtures which have nonconforming materials and construction values. The graph (Figure 21) provides a clear picture of the way in which the four mixtures differ from the Witczak predictions.

When each of the four aggregate-asphalt combinations is considered separately, it is found that for the one combination, aggregate 0 and asphalt 0, Witczak's predictions were quite good. The graph of these is given as Figure G-1. The simple correlation in this case between Witczak's predictions and the measured values was 0.87.

These results suggest that there may be aggregates and asphalts for which Witczak's method for predicting the resilient modulus may be useful even for nonconforming values of the material and construction variables. However, they also suggest that there are also certain combinations for which the method must not be used. It may also be informative to consider the extent to which some other model could predict these values observed in this experiment.

The resilient modulus experiment was designed so that a model of the following form could be used:

$$\text{Modulus} = B_0 + B_1 (\text{Aggregate}) + B_2 (\text{Asphalt}) + B_3 (\text{Dust}) + B_4 (\text{AC}) + B_5 (\text{AV}) + B_6 (\text{Dust*AC}) + B_7 (\text{Dust*AV}) + B_8 (\text{AC*AV}) + \text{error} \qquad \text{(G-2)}$$

When this model is applied to all the data, it is found that the only interaction term of any importance is the AC*AV. It was also observed that Dust is of very little importance, but it was retained in the model. The resulting fitted model with the estimated coefficients is:

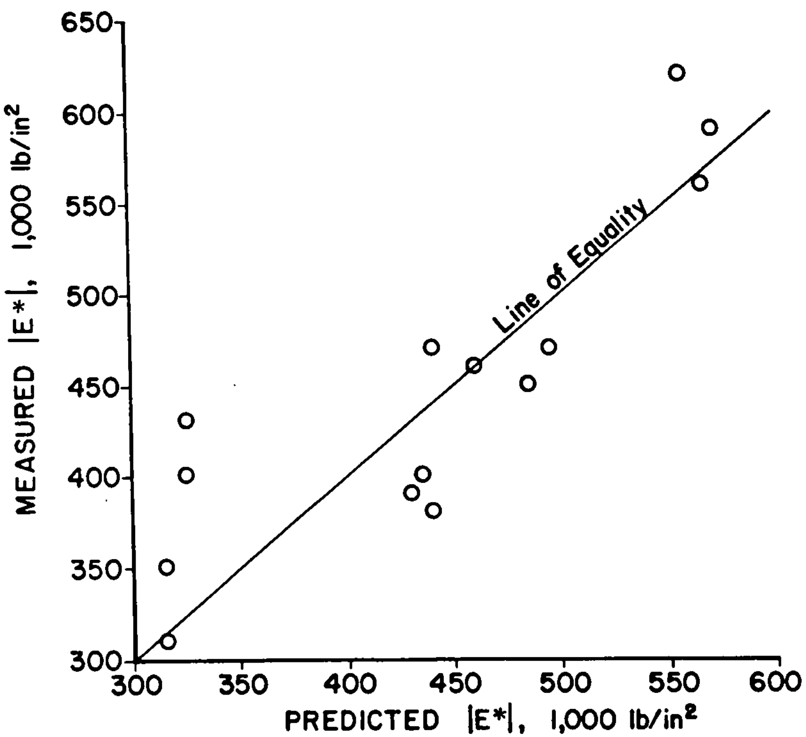

Figure G-1. Measured modulus vs. predicted by Witczak.

$$\text{Modulus} = 1{,}653{,}142 - 12{,}689 * \text{Aggregate} - 95{,}491 * \text{Asphalt} - 6{,}838 * \text{Dust} - 182{,}668 * \text{AC} - 77{,}058 * \text{AV} + 11{,}049\ (\text{AC} * \text{AV}) \quad \text{(G-3)}$$

The R^2 for this fitted model was 0.80 and the coefficient of variation was 12 percent so that this regression model appears to be quite satisfactory. The graph of the measured resilient modulus versus values predicted by this regression equation is presented in Figure G-2. The model used allowed for an aggregate and an asphalt simple additive effect. If there is to be a model which uses some property of the asphalt and aggregate in this simple manner this procedure would have allowed for all such choices. For example, if the asphalt viscosity is to provide a simple additive function (of any form) to the prediction equation, the present model would have accounted for such. However, the present model would not have allowed for the coefficients of the Dust, AC, AV, and AC*AV to depend upon the values of the aggregate and asphalt and hence isn't really very general. In this sense, the most general modeling would simply model the modulus separately for each combination of aggregate and asphalt and thereby allow all coefficients to be specific for the particular combination. This would provide the best possible predictions if there is sufficient data available for each combination so that the coefficients may be accurately estimated.

The experiment which was designed and carried out in order to develop and evaluate a model to predict the tensile strength as a function of the variables aging (aged or not), Dust, Ac, and AV consisted of two replications of a half replication of the 2^5 factorial experiment and these were repeated for both the aged and unaged specimens. This would provide for the estimation of the coefficients in a regression model which had all factors and their two-way interactions

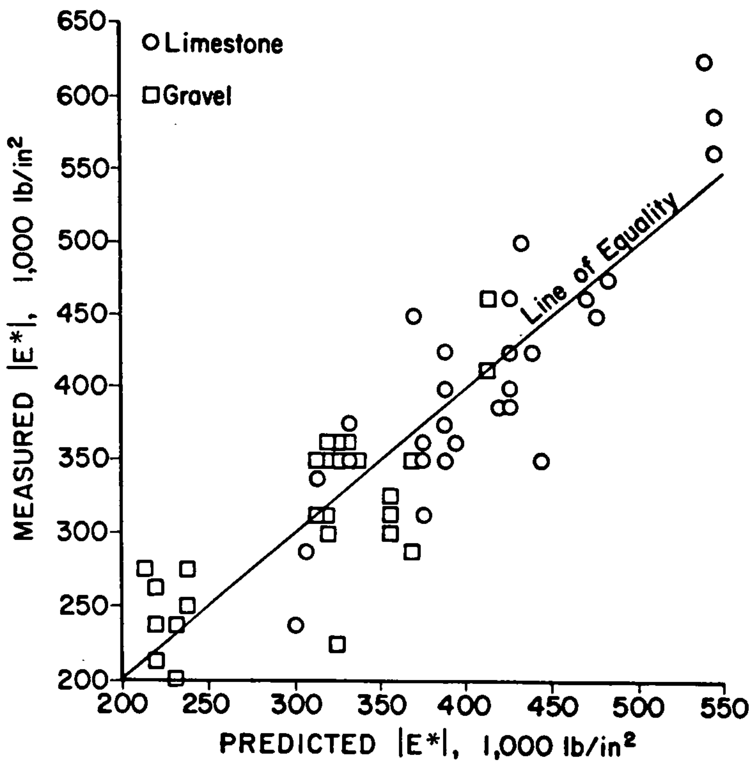

Figure G-2. Measured modulus vs. predicted by regression.

present and, in addition, allow sufficient degrees of freedom for the estimation of the variance of the experimental error.

When the data were fitted to the simple model:

$$\text{T Strength} = B_0 + B_1\ (\text{Aging}) + B_2\ (\text{Dust}) + B_3\ (\text{AC}) + B_4\ (\text{AV}) + \text{error} \qquad \text{(G-4)}$$

it was found that a quite satisfactory prediction equation was produced. The value of R^2 was 82 percent, which would seem to be quite good. The resulting fitted model is:

$$\text{T Strength} = 613 + 32\ (\text{Aging}) - 9\ (\text{Dust}) - 57\ (\text{AC}) - 13\ (\text{AV}) \qquad \text{(G-5)}$$

It is also noted that the coefficient of variation was 11 percent which is also quite good. This good performance of the fitted model is all the more impressive when it is observed that no specific properties of the aggregates and asphalts were used in the model, i.e., a quite satisfactory model for these four combinations of aggregates and asphalts uses only the values of the Dust, AC, AV, and aging.

It was not the intent of this experimental program to develop and evaluate a general prediction equation for the tensile strength of asphalt mixtures, and the results, while impressive, are only for the set of these aggregates and asphalts. The purpose of the experiment was to demonstrate the design of an experiment from which such a model could be developed and evaluated, using only a very limited amount of laboratory resources for this purpose. The results are best illustrated by the graph of the measured tensile strength versus the values predicted by the regression model in Figure G-3.

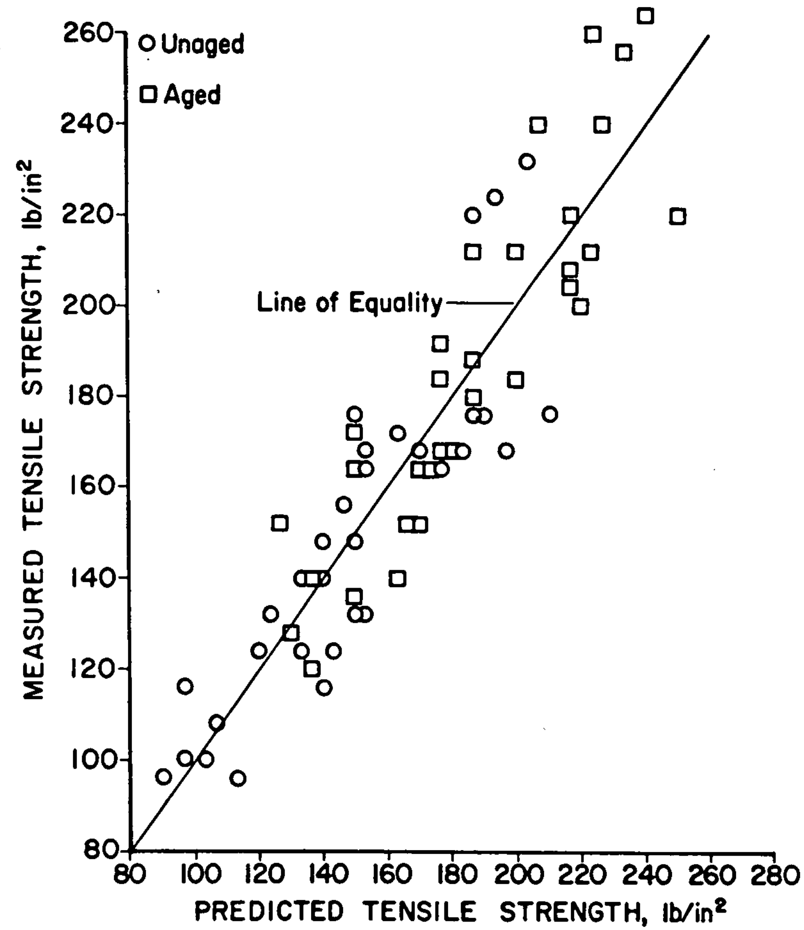

Figure G-3. Measured tensile strength vs. predicted by regression.

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