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NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **291**

DEVELOPMENT OF PAVEMENT STRUCTURAL SUBSYSTEMS

TRANSPORTATION RESEARCH BOARD
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F. FINN, C. L. SARAF, R. KULKARNI, K. NAIR, W.SMITH, and A. ABDULLAH
Woodward-Clyde Consultants
Walnut Creek, California

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AREAS OF INTEREST:

- PAVEMENT DESIGN AND PERFORMANCE
- BITUMINOUS MATERIALS AND MIXES
- CONSTRUCTION
- MINERAL AGGREGATES
- (HIGHWAY TRANSPORTATION)
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TRANSPORTATION RESEARCH BOARD
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WASHINGTON, D.C. DECEMBER 1986

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

The programs (PDMAP and COLD) described in this report are capable of predicting the amount of distress in terms of cracking and rutting that should be expected in a given asphaltic concrete pavement structural section built of materials, and located over subgrade soils, of measurable characteristics, and exposed to specific traffic loadings and environmental conditions. The report will be of interest and value to materials engineers, pavement designers, researchers, and others interested in improving the performance of asphaltic concrete pavements. On the basis of the limited field verification efforts, it is recommended that the predictive models be used on a trial basis and that local calibration activities be undertaken.

Pavements are extremely complex physical systems involving the interaction of numerous variables. Their performance is influenced by such factors as material properties, environment, traffic loading, and construction practices. Pavement design procedures currently in use depend heavily on "empirical" (described as relying on experience and observations rather than theoretical knowledge) relationships based, to a large extent, on long-term experience and field tests such as the AASHO Road Test. It is generally recognized that such relationships between traffic loading and pavement performance apply only to the conditions under which they were developed. Application of these relationships to other sets of conditions is quite difficult. Considerable research has been conducted over the past 15 to 20 years with the objective of applying "mechanistic" (described as dealing with the laws of mechanics) technology to the structural analysis and design of pavements.

The objectives of this research were to evaluate previous research and experience in the field of mechanistic technology as it applies to asphaltic concrete pavements; formulate models for the prediction of fatigue cracking, thermal cracking, and wheel path rutting in asphaltic concrete pavements; and demonstrate the calibration and application of these models to pavement analysis and design. The research team assembled by Woodward-Clyde Consultants selected linear elastic characterization of the pavement materials and a multi-layered system for accomplishment of the project objectives primarily because of the large amount of previous research utilizing these concepts and the prospect for early implementation. The primary project activity was to "package" available information into computer programs. This was accomplished with the development of the computer programs Probabilistic Distress Models for Asphalt Pavements (PDMAP) for predicting fatigue cracking and rutting and Computation of Low-Temperature Damage (COLD) for predicting low-temperature cracking. These computer programs were checked for implementability by their review by several state highway agencies, and local calibration was demonstrated with the cooperation of the Florida Department of Transportation and the Utah Department of Transportation.

Further verification and calibration of mechanistic models to reasonably predict the amount of distress likely to occur in a given pavement for a known set of conditions will certainly be a useful tool to agencies involved in the development of pavement design methodology. This technology will eventually be used to (a) accommodate changing wheel loads, (b) better use existing and new materials, (c) improve reliability of performance estimates, and (d) increase ability to consider influence of environmental factors. Initial use of the models will likely be for special pavement design problems for which field experience is not available to support empirical design procedures.

Appendixes B through G, containing computer program documentation and user guides for PDMAP and COLD, are of primary value to persons directly involved in operation of the programs. These appendixes are not included in this report but are contained in a separate Volume 2. Copies of Volume 2 have been distributed to the program sponsors and are available, on a loan basis, to other interested persons on written request to the Cooperative Research Programs, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, D.C. 20418.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 1-10B by Materials Research & Development, Inc. (MR&D), a divison of Woodward-Clyde Consultants. The principal investigator for MR&D was Fred N. Finn. Key staff members assigned to the investigation were Dr. C. L. Saraf, Dr. W. Smith, Dr. R. Kulkarni, and Dr. A. Abdullah. Overall administration and technical supervision of the project for Woodward-Clyde Consultants was assigned to Dr. K. Nair.

Consultants to the project who provided technical guidance for the investigation were Prof. M. Witczak, University of Maryland; Prof. J. Deacon, University of Kentucky; Prof. K. O. Anderson, University of Alberta; Prof. C. L. Monismith, University of California, Berkeley; Mr. B.. F. Kallas, The Asphalt Institute; and Mr. R. Schmidt, Chevron Research Corporation. Dr. R. Shiffman and his associates at the University of Colorado Computer Center provided assistance in reviewing the PDMAP computer program. Dr. J. T. Christison consented to the extensive use of information from his Ph.D. dissertation in the low-temperature subsystem. Dr. H. Shah of Stanford University has provided valued guidance regarding analysis procedures used by the staff.

Grateful acknowledgment is also extended to the five state agencies that participated in the Phase I evaluation of the investigation, with particular emphasis on the implementability of the results: the state highway departments in Florida, Kansas, Minnesota, Arizona, and California. The cooperation of the Illinois Department of Transportation in obtaining samples of AASHO Road Test materials was also appreciated.

Special assistance in the Phase II field investigation was provided by Mr. Douglas Anderson of the Utah DOT and Messrs. Larry Smith and Gale Page of the Florida DOT.

Special acknowledgment is made to The Asphalt Institute for the material testing accomplished in its College Park laboratories under the supervision of Mr. B. F. Kallas.

During the course of the study, frequent contacts were made with Mr. W. Kenis of the Federal Highway Administration in order to coordinate this investigation with the ongoing VESYS project. The FHWA also consented to the use of computer programs developed by Prof. W. Hufford at the University of Utah relating to probabilistic aspects of the computer programs.

DEVELOPMENT OF PAVEMENT STRUCTURAL SUBSYSTEMS

SUMMARY

The primary objective of the investigation described in this report was to develop a series of models designed to predict physical distress in asphalt-type pavements. Specifically, methods have been developed that attempt to predict when and how much fatigue cracking and rutting will occur and when low-temperature cracking can be expected

The working hypothesis for the development of the prediction subsystems was the assumption that the forms of distress predicted are related to some combination of stress, strain, or deformation within the pavement and that these responses can be calculated on the basis of estimates of the elastic constants of the pavement and subgrade materials. This approach is generally described as mechanistic, as compared with the more conventional empirical approach that depends exclusively on observations and correlations involving a large number of variables. The mechanistic approach has the advantage of combining variables into a manageable framework for application by the user agency.

Background information for the investigation was developed under previous NCHRP projects, specifically NCHRP Project 1–10 (*NCHRP Report 139 and NCHRP Report 140) and* NCHRP Project 9–4 (*NCHRP Report 195*). Results of research efforts from a number of sources in the United States, Canada, and Europe were also used. In effect, this investigation was designed to "package" the results of research reported during the past 15 years into two implementable subsystems for the prediction of three types of distress.

The computer programs have been developed through the investigation and are referred to as PDMAP (Probabilistic Distress Models for Asphalt Pavements) and COLD (Computation of Low-Temperature Damage).

The PDMAP program is designed to predict fatigue cracking and rutting in asphalt-type pavements and includes the following features:

1. Seasonal variations in material properties.
2. Long-term changes in material properties.
3. Daily variations in traffic (e.g., difference between daytime and nighttime truck traffic).
4. Traffic growth on a yearly basis if required.
5. Uncertainty in measuring material properties.
6. Conversion, by simple iteration, of the prediction models to design models for the selection of material and thicknesses.
7. Distress predictions considering untreated or treated materials, including asphaltic concrete, asphalt emulsion mixes, and soil–cement.

The COLD program is designed to predict if and when low-temperature cracking is likely to occur, given that a specific layer of asphaltic concrete and asphalt binder is to be placed in a known location for which reasonable temperature data are available. The pertinent features of this program are as follows:

1. Computes temperature in the asphaltic concrete.
2. Computes the thermally induced stress in asphaltic concrete at 2-hour intervals.
3. Provivdes a comparison of tensile strength with thermal stress at various levels of reliability in order to predict the probability of low-temperature cracking.

Both programs are believed to have a number of highly functional applications. Some of the more significant attributes are as follows:

1. PDMAP can be used to simulate field conditions incorporating climate, materials, traffic and loads, layer combinations, and variability in traffic and material properties. In this way a great deal of information can be developed without the need for a large number of case studies. This is not to say that field trials will not be necessary; however, the number of such trials can be greatly reduced through the use of PDMAP.
2. PDMAP can be used in pavement management systems for predicting physical distress if so required by the system.
3. PDMAP can be used as a basis for the development of structural design procedures tailored to materials, environment, and traffic.
4. PDMAP can be used to diagnose structural failures and to assist in defining those factors most likely to cause premature distress.
5. COLD can identify the potential for low-temperature cracking for pavements and materials to be constructed in a particular location.
6. COLD can be used to select asphalt binders that should be used to minimize the potential for low-temperature cracking.
7. COLD can be used to prepare asphalt specifications designed to reduce the susceptibility for low-temperature cracking.

It is pertinent to note that the current (1981) pavement design manual produced by the Asphalt Institute has made use of the concepts and models developed in this project as the basis for preparing design charts to be used with both asphalt-stabilized and untreated aggregate base designs for asphalt-surfaced pavements.

The results of the investigation have been summarized in two reports. The reports contained herein, includes information pertinent to the research approach, findings, appraisal and implementation, conclusions and recommendations, and one appendix (A) that provides background information for techniques used in the development of the structural subsystems. A second volume, containing Appendixes B through G, has been prepared as a User's Guide for those agencies interested in implementing the PDMAP and COLD programs. Included in that volume are appendixes identified as: User's Manual for the Computer Program PDMAP (Appen. B), User's Manual for the Computer Program COLD (Appen. C), Documentation of Computer Program PDMAP (Appen. D), Dictionary of Terms Used in Computer Program COLD (Appen. E), Probabilistic Analysis in the Program PDMAP (Appen. F), and Probabilistic Analysis in the Program COLD (Appen. G). Copies of Volume 2 are available on written request from the Cooperative Research Programs, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, D.C. 20418.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

INTRODUCTION

It would be difficult to say when engineers first became interested in developing a mechanistic (stress, strain, deformation) approach to the design and analysis of flexible (asphalt) pavements. Probably, the first formalized effort of this kind was made by Burmister (*1*) in 1943. Love (1929) and Hogg (1938) had provided some of the mathematical solutions; however, it remained for Burmister to provide solutions to boundary value problems for layered systems loaded at the surface with loads representing the tires of an airplane, a truck, a car, and so on.

Only limited use of Burmister's classical efforts followed his 1943 publication. Limited use was primarily the result of three factors: (1) solutions were limited to three layers, (2) there was difficulty in measuring material properties, and (3) design and evaluation criteria were lacking.

For the most part, engineers have relied on so-called empirical information to develop design procedures for pavements. Such procedures include the determination of thicknesses, materials, and construction requirements. Road tests have been the major source of empirical information. Some of the better known projects are the Bates Road Test in Illinois, the Brighton Road Test in California, the WASHO Road Test in Idaho, and the most recent, and by far the largest, the AASHO Road Test (1956–1960) in Illinois. None of the official analyses of these projects attempted to relate mechanistic responses of the pavement system to performance.

It is not strictly correct to indicate that engineers ignored mechanistic considerations completely. For example, the Corps of Engineers, in extrapolating the CBR method from single wheels to a heavy multiple-gear configuration (*2*), used calculations of shear stress to develop necessary thickness design curves. Hveem and Carmany (*3*) related their thickness design procedures to the case of a strip load on a soil foundation. In each case, however, the mechanistic approach was lost in the transformation to empirical models or formulas.

During the period of traffic testing at the AASHO Road Test, a number of investigators in the United States, notably Seed and his colleagues at the University of California (*4*), began to initiate research efforts directed toward the development of pavement evaluations and design procedures using material characterization methods appropriate to the use of the theory of elasticity as used by Burmister. At the same time, Jones (*5*) and Peattie (*6*) had made progress in developing tabular and graphic solutions to three-layer systems.

By August of 1962, when the First International Conference on the Structural Design of Asphalt Pavements was held at the University of Michigan, a wide range of investigations involving mechanistic considerations of stress, strain, and deformation for pavement design and evaluation were proposed. From that time to the present (1985), a continuing interest has been shown for the implementation of the mechanistic approach.

The first comprehensive design method using a mechanistic appraoch (based on using criteria for stress, strain, or deformation) in the United States was published by Dorman and Metcalf in 1964 (*7*). Several methods have been produced since that time, including one by the Kentucky Department of Highways (*8*) and a method for Air Carrier airports by Witczak (*9*). To date, two agencies have developed pavement decision procedures for highways using distress prediction models: specifically, Shell International Petroleum Co. and the Asphalt Institute.

In 1966, the problem of low-temperature cracking in asphalt pavements was extensively described by Canadian engineers (*10, 11*). NCHRP Project 9-4 reviews the many efforts dealing with the occurrence of low-temperature cracking and summarizes procedures for minimizing such cracking through material specifications. From 1966 through 1975, there was a continuing effort to develop predictive models for low-temperature cracking. Haas (*12*), Burgess et al. (*13*), Christison and Anderson (*14*), and Monismith et al. (*15*) have contributed to a mechanistic appraoch for the prediction of low-temperature cracking.

In 1968, NCHRP initiated the first in a series of projects designed to investigate basic properties of pavement components for the purpose of extending the findings from the AASHO Road Test. These investigations have been identified as the NCHRP 1-10 series and have resulted in three reports to date (*16, 17, 18*). The results have helped to identify the appropriate constitutive relationships for pavement components and have provided a systems engineering approach to the development of comprehensive pavement design procedures.

The investigation described herein includes the development and verification of structural subsystems designed to predict three important types of pavement distress for asphaltic pavements: specifically, (1) fatigue cracking, (2) permanent deformation (rutting), and (3) low-temperature cracking.

In addition to work by the research agency's staff, some effort was made to obtain the advice of a team of consultants familiar with the elements of the damage models. Also, consultations with six state agencies were found useful in planning program needs and in evaluating the potential implementability of the results.

The consultants for the project were Dr. J. Deacon, Dr. M. Witczak, Prof. C. L. Monismith, Prof. K. O. Anderson, Mr. B. Kallas, Mr. R. L. Schmidt, and Dr. H. Shah. Their major input came in the early planning stages. Because of the unique problems related to low-temperature cracking, Prof. Anderson provided detailed input throughout the term of the investigation.

Five states assisted in evaluating the subsystems during the development stages: Florida, California, Arizona, Kansas, and Minnesota. Personnel from the Illinois Department of Transportation provided samples of materials used in the construction of test sections at the AASHO Road Test. Chevron Research Corporation of Richmond, California, provided asphalts retained during construction at the AASHO Road Test. The Asphalt Institute and the University of Maryland participated in cooperative test programs during the verification phase. The states of Florida and Utah cooperated in the verification phase by providing field data and laboratory information for the test

No endorsement is expressed or implied by any of the individuals of organizations referred to; nevertheless, overall effectiveness of the investigation was enhanced by the opportunity to interact with this group of states, advisors, and consultants.

OBJECTIVES AND SCOPE

The primary objective of this investigation is to develop, modularize, and verify flexible-type pavement structural subsystems utilizing implementable mechanistic techniques to analyze specific distress modes in pavement structures for various environmental, traffic, and construction conditions, and which have the capability of being used to evaluate both new pavement structures and overlays. The analysis techniques are to be based on available information from previous and current research, including the FHWA structural subsystems.

RESEARCH APPROACH

The general approach used in this investigation was to find appropriate mechanistic models for the prediction of distress and to calibrate (adjust) the models based on field observations. Thus, each model contains a degree of verification through the empirical procedures used for the prediction of distress. Also, the procedures are oriented to simplicity; that is, given a choice between questionable improvement in the prediction models and a less complicated procedure, the less complicated procedure has been selected.

The project was divided into two distinct sets of activities: first, to develop the subsystems, and second, to verify their application to field conditions.

Activities associated with the development phase are broadly described as follows:

1. Develop damage prediction models using mechanistic responses as the determinants of distress.
2. Select and modify structural analysis computer programs that can reliably provide mechanistic responses appropriate to the damage models.
3. Develop probabilistic procedures that will account for variations in material characteristics, traffic, and material properties related to distress (e.g., fatigue properties).
4. Develop subsystems that can reflect seasonal variations due to environmental changes.
5. Select tests procedures for characterization of materials.
6. Develop a methodology (computerized) that will combine all of the previous five activities into prediction models for (a) fatigue cracking, (b) low-temperature cracking, and (c) permanent deformation.

In order to include some indication as to the uncertainty of the predictions, it was necessary to incorporate concepts that account for variations in material characteristics, fatigue properties, tensile strength, and traffic. It is pertinent to note that such estimates of variations should reflect the field conditions rather than laboratory variations.

The basic mathematical model for the probabilistic portion of the subsystems was developed by Dr. W. Hufferd of the University of Utah under FHWA sponsorship and was adopted for use for this investigation by the project staff.

In general, the purpose of the project was to package available, yet fragmented, research into a series of implementable subsystems. As will be noted, there were a number of "gaps" in the research which had to be filled as part of this investigation.

Evaluation was achieved by working with two states known to be interested in mechanistic applications for distress predictors. Both Florida and Utah had active programs and test sites under observation which could be used for evaluation and as examples for application of the subsystems.

Figures 1 and 2 illustrate the basic elements of each of the damage prediction subsystems. As can be noted from these figures, the basic input information includes climate, loading conditions (either traffic or temperature), and material characteristics. The structural analyses provide the stress on strain information required to predict distress.

BACKGROUND

The development of a mechanistic procedure for predicting pavement distress includes three major elements: (1) methods for characterization of material properties; (2) structural analysis for multilayered systems; and (3) methods of relating mechanistic responses to distress.

Materials Characterization

Three constitutive equations have been proposed for paving materials: linear elastic, nonlinear elastic, and linear viscoelastic. The linear elastic characterization has been modified to include such factors as stress sensitivity of most untreated materials and the influence of time and temperature on asphalt-bound systems.

A complete description of the various constitutive relationships has been provided in *NCHRP Report 140* by Nair and Chang (*17*).

For purposes of this investigation, it was decided to use linear elasticity as the basic constitutive relationship for paving materials. This decision was based primarily on the large amount of research reported in this area, including the design models by Dorman and Metcalf (*7*), Witczak (*9*), and Deen et al. (*8*). The relative simplicity of linear elastic procedures was also a consideration in this selection.

Structural Analysis

A variety of computer programs is currently available for the solution of a boundary value problem for a multilayered (pavement) system. These programs can be divided into two categories: finite-element and layered systems.

The desired attributes of the computer programs are as follows:

- They should accommodate multiple layers, with a minimum of five layers.

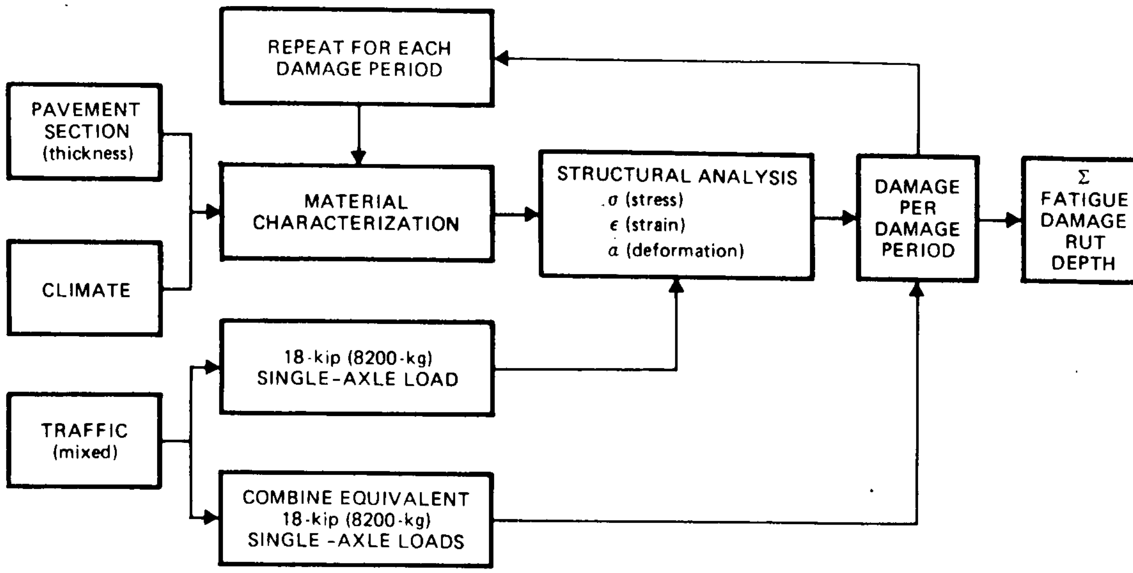

Figure 1. Fatigue and rut depth, subsystem PDMAP.

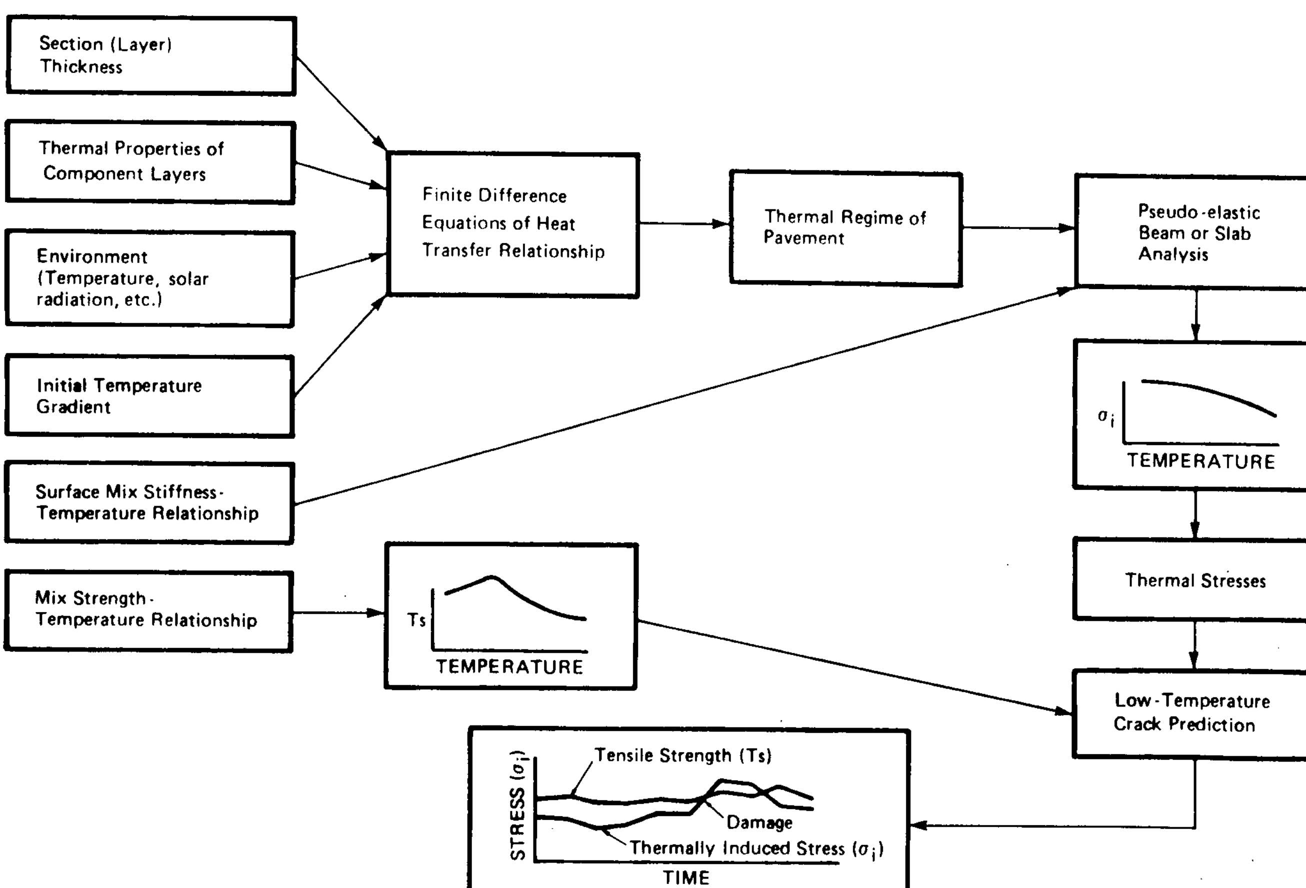

Figure 2. Low-temperature cracking.

- They should include provison for superposition of dual tire loads.
- They should include provision for stress sensitivity of unbound materials such as aggregate base and subbase or natural subgrade soil, either granular or fine-grained.

The finite-element programs are considered more powerful (accommodate greater number of variables) and more reliable (mathematically precise), but they are more expensive to run. (Note: Recent hardware and software developments make finite-element programs increasingly accessible to users; however, the PDMAP programs should continue to use the PSAD program, unless calibration is provided for alternative structural analysis packages.)

For this investigation, the SAPIV finite-element program developed at the University of California, Berkeley, has been used to establish the reliability of the alternative layered-system solutions and to determine whether such programs are interchangeable. Solution of identical problems with four computational programs had indicated that reasonably comparable primary response outputs will be obtained for each procedure. The layered-system programs included in the study were:

- CHEV5L—5 layers, without superposition or stress sensitivity (Chevron Research Corporation).
- NLAYER—an extension of CHEV5L to 14 layers, with superposition but without stress sensitivity (Dr. R. Shiffman).
- CRALAY—a computer program developed by Australian engineers, similar to CHEV5L.
- PSAD—an extension of CHEV5L for 5 layers, but with provision for both superposition and stress sensitivity (University of California).

The outputs from the four elastic-layered systems and the SAPIV finite-element programs were compared on the basis of estimated damage. In other words, how the fatigue life was affected by outputs of the various structural analysis methods. It was judged that differences were not significant considering the variances associated with inputs to the structural analysis programs.

The program used for this investigation is the PSAD solution with certain modifications made by the project staff for use with the fatigue cracking and rut depth prediction models.

Low-temperature stress analysis was comprehensively reviewed by Christison (*19*); he compared elastic beam, viscoelastic beam, and viscoelastic slab theory. It was concluded from this investigation that the methods proposed by Hills and Brien (*11*) could be used to calculate thermally induced stress with sufficient accuracy for the prediction of low-temperature cracking. Hills and Brien reported that the longitudinal stress in a pavement surface, assuming a beam of infinite length and neglecting lateral restrain, could be approximated by:

$$T^x = \alpha \sum_{T = T_o}^{T = T_f} S\,(\Delta T) \tag{1}$$

in which:

T_x = thermally induced tensile stress;
α = coefficient of thermal contraction;
S = asphaltic concrete stiffness modulus; and
ΔT = temperature interval, of which a finite number are taken between $T = T_o$ and $T = T_f$.

Distress Prediction

The working hypothesis for each prediction subsystem assumes that the observable physical distress (cracking or rutting) is a function of the state of stress, strain, or deformation induced by traffic loads or low temperature.

As will be shown, a number of investigations of fatigue and low-temperature cracking had been completed, but very little information was available for the prediction of rutting.

The terms *distress* and *damage* are defined for purposes of this investigation as follows:

- *Damage.* The effect of a single load application in contributing to cracking or rutting.
- *Distress.* The cumulative effect of damage resulting in an observable and measurable amount of cracking or rutting.

Fatigue Cracking

For the three materials considered in this investigation—asphaltic concrete, asphalt emulsion mixes, and cement-treated bases—it was desirable to obtain a single damage hypothesis. Sufficient knowledge exists regarding the fatigue of asphalt and soil-cement systems that implementation of a fatigue subsystem in practice seems possible. On the basis of a phenomenological approach to the behavior of these materials systems under repeated flexure, it has been determined that the fatigue life of a mixture can be expressed as a function of the repeatedly applied tensile strain as follows:

$$N_f = K \left(\frac{1}{\epsilon}\right)^{\eta} \tag{2}$$

in which: N_f = number of load repetitions to failure; ϵ = initial tensile strain determined for treated materials in constant stress-type loading; and K, η = experimentally determined constants.

It is pertinent to note that fatigue cracking damage models have been based on laboratory testing, with very little attempt to correlate results with the occurrence of distress in actual pavements. Monismith et al. (*20*) and Bergen (*21*) have shown in a limited number of cases that crack initiation observed in the field correlates with laboratory test results.

Investigations by Deen et al. (*8*), Kingham (*22*), and Hicks and Finn (*23*) have developed distress criteria from field information. In these studies, strain in the asphaltic concrete was used as the indicator of change in riding quality as measured by serviceability, or as a predictor of fatigue cracking.

McCullough et al. (*24*) and McCullough (*25*) have suggested a statistical approach to estimating the percent of cracking, based on the variations in fatigue properties obtained from laboratory testing. This methodology turns out to be very close in concept to the approach finally selected for this investigation.

Thus, there is a considerable amount of research available to provide an indication of the determinant of fatigue cracking but no significant amount of knowledge regarding the prediction of areal cracking in actual pavements.

For purposes of this investigation, fatigue cracking is defined as the amount of cracking in the wheel path area, which is estimated to be approximately 6 ft wide (about 2 m) for both wheel paths. Thus, 10 percent cracking in the wheel paths would be approximately 5 percent of the total area.

Permanent Deformation

Dorman and Metcalf (*7*) have proposed criteria that must be satisfied if excessive rutting is to be avoided; however, no prediction of the amount of rutting is available from these criteria. Similar applications have been made by Witczak (*9*), Finn et al. (*28*), and Chou et al. (*29*).

An alternate approach proposed by Romain (*30*), Barksdale, (*31*), and Monismith et al. (*32*) is based on combining the state of stress, using linear elastic constitutive relationships, with appropriate fitting coefficients obtained in repetitive triaxial tests. This procedure provides for a prediction of permanent deformation in each layer of the pavement and in the subgrade which, when summed, gives an indication of the cumulative surface deformation as a function of traffic. This procedure offers promise for the future; however, it was not considered implementable for the full pavement section at the present state of development.

Hills et al. (*33*) have proposed a procedure for the prediction of rutting in full-depth asphalt construction based on creep testing of asphaltic concrete. The method appears to have good application to all types of asphalt construction but has not been fully developed for conventional pavement construction.

Morris et al. (*34*) have proposed a method to predict rutting based on laboratory tests designed to simulate field conditions. The procedure requires a comprehensive series of triaxial tests with repetitive loading under stress states expected in the field. Statistical techniques are then used to estimate permanent deformation as a function of vertical and horizontal stresses, temperature, and the number of load repetitions. The procedure offers promise but is currently limited to full-depth asphaltic concrete constructions. The method also requires a considerable amount of testing in order to identify all of the factors contributing to rutting.

Low-Temperature Cracking

Extensive research has been completed on the subject of low-temperature cracking in asphalt pavements. It has been determined that such cracking will occur when the thermally induced tensile stress exceeds the tensile strength when determined or measured at corresponding temperatures and when evaluated over relatively long loading times.

The general methodology for predicting low-temperature cracking is represented in Figure 3. In this figure, it will be noted that as the temperature in the asphaltic concrete decreases, the thermally induced stress increases, and the tensile strength increases to a peak and then decreases. Thermally induced stress and tensile strength versus temperature are then plotted on the same figure to illustrate low-temperature cracking at the points of intersection or when tensile stress exceeds tensile strength.

The procedures referred to above have been evaluated and verified by Canadian engineers for the Canadian environment. The Ste. Anne Test Road in Canada (*26*) has provided some of the more impressive results, although Anderson and his colleagues (*10*) have also reported verification in Alberta.

Shahin and McCullough (*27*) have proposed a method for the consideration of low-temperature fatigue cracking related to the number of low-temperature cycles to which a pavement has been subjected.

CALIBRATION AND EVALUATION

For purposes of this investigation, calibration and evaluation are accomplished in two steps.

1. Calibration is the adjustment required to establish correlations between laboratory models and a set of field performance data, e.g., AASHO Road Test data.
2. Evaluation or verification of the calibrated model is obtained by comparing predictions to observations of pavement sections not included in the calibration exercise and for which adequate information is available.

The fatigue prediction subsystem was initially calibrated by using AASHO Road Test data. This effort produced a prediction model that was reasonably well correlated with field observation. A similar approach was used with data from Florida. As discussed in Chapter Two, the Florida calibration and evaluation started with laboratory fatigue data developed in Florida DOT laboratories; the AASHO correlations were based on laboratory fatigue data developed in the University of California (Berkeley) laboratories. Evaluation of the Florida fatigue model was obtained by comparisons with two additional sets of performance data from test sections located in Florida.

The rut depth prediction subsystem was developed by correlations with the mechanistic models to AASHO Road Test data. These models were subsequently calibrated for Florida conditions using one set of test section data and then evaluated by comparisons with data from the same two sets of test sections used for the fatigue models.

The cold-temperature cracking prediction subsystem was developed using procedures described by Christison and Anderson (*14*). The procedures used by Christison and Anderson had been correlated with observations in Canada. Low-temperature cracking was not reported at the AASHO Road Test; hence, calibration to that project was not possible. The purpose of this investigation was to adapt the Christison-Anderson approach for use in the United States. Information from the Utah DOT was used to evaluate the proposed COLD subsystem for conditions in that state.

SUMMARY

The preceding portions of this chapter have identified the objectives and research approach used to develop and evaluate the distress prediction subsystems included in this investigation. It is also pertinent to discuss the significance of being able to predict cracking and rutting in asphaltic pavements.

Following the publication of findings from the AASHO Road Test in 1962, it was generally thought that riding quality as expressed by the present serviceability index would suffice as an indicator of pavement performance. However, the objective

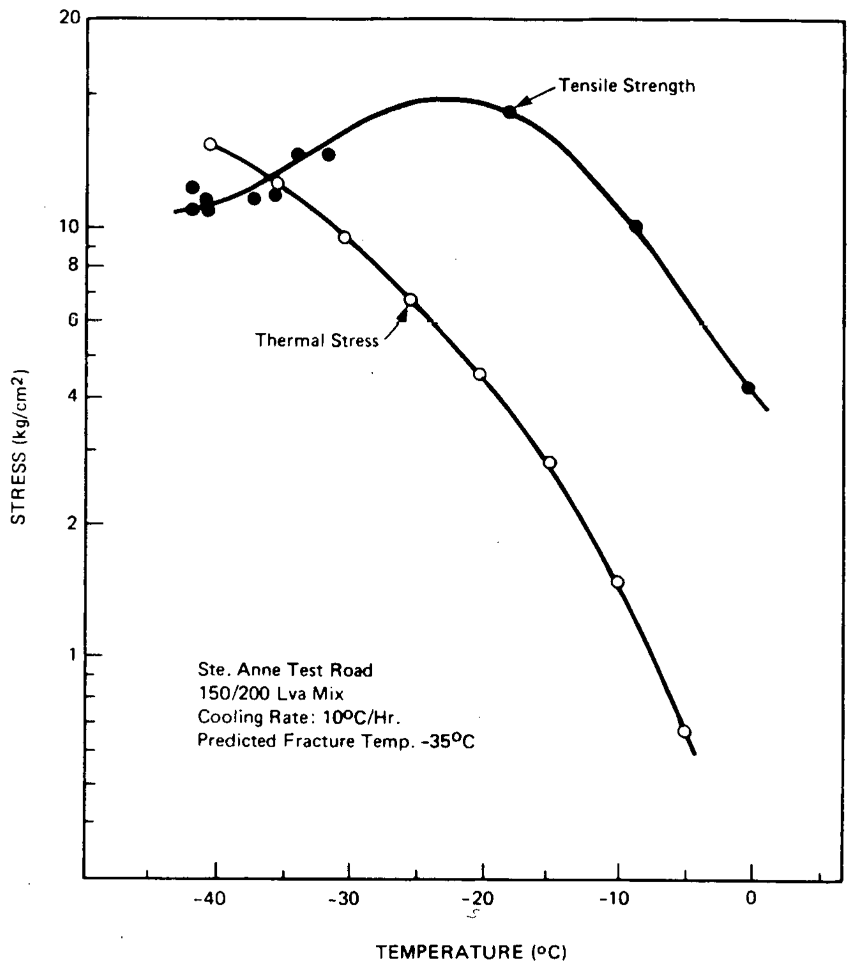

Figure 3. Example of thermal stress calculation, and prediction of fracture temperature, for a restrained strip of asphaltic concrete. (After Ref. 12)

measurements used to estimate riding quality assigned very little significance to the occurrence of fatigue cracking and rutting and nothing to low-temperature cracking.

In time, an increasing number of engineers have become dissatisfied with the concept that serviceability alone could adequately identify performance requirements since physical distress related to preservation of investment was not uniquely related to riding quality or pavement roughness. Also, in the period after 1970, pavement management systems were being investigated by a number of highway departments in the United States. When evaluating performance requirements for these systems, it became apparent that it would be necessary to predict physical distress as well as pavement roughness.

In summary, the need for prediction models of the type included in the investigation will have merit with regard to the initial design and construction of pavements and with regard to the need for maintenance and rehabilitation.

The text of Report 5 for the AASHO Road Tests (*37*) summarizes the dilemma as follows:

> An important element of the serviceability and the performance of the flexible pavements was the cracking of the surfacing materials. Although cracks do not, in themselves, have much effect on the ability of the pavement to service traffic, they serve as indications that something about the pavement design is inadequate and that failure of the pavement is likely to occur at an earlier date than would be the case if no cracking appeared.

The purpose of this investigation is to provide design agencies with the ability to predict distress and avoid premature failure.

FINDINGS

This chapter is subdivided into four sections as follows: (1) development of subsystems, (2) summary of PDMAP (Probabilistic Distress Models for Asphalt Pavements) for predicting fatigue cracking and rutting, (3) summary of program COLD (Computation of Low-Temperature Damage) for predicting low-temperature cracking, and (4) evaluation of both PDMAP and COLD by field trials. Based on experiences obtained from this project, procedures for implementation of both subsystems are provided in Chapter Three.

DEVELOPMENT OF SUBSYSTEMS

The methodology used in the development of the two subsystems is based on the application of linear elasticity relationships to a multilayered system as represented by an asphalt-type pavement.

Fatigue cracking predicted by PDMAP is a result of the cumulative traffic loadings. The basic hypothesis used in PDMAP is referred to as the linear summation of cycle ratios frequently referred to as the Miner criterion. According to Miner:

$$N_c = \frac{1}{\sum_{i=1}^{i=m} P_i/N_i} \tag{3}$$

in which: N_c = service life under simple loading; N_i = service life in simple loading under specific load (stress) condition i; and P_i = applied percentage of load (stress) condition i.

The purpose of the permanent deformation model is to predict wheel path rutting as a function of traffic and the state of stress in the pavement structure. The method is similar in concept to that reported by Dorman and Metcalf (*7*) with further development by others (*8, 9, 23, 52*). Dorman and Metcalf proposed a limiting strain at the subgrade as a means of controlling the amount of rutting, as shown in Figure 4. PDMAP attempts to predict the rate of rutting as a function of traffic, as well as deflection and vertical stress. The total amount of rutting is obtained by combining rate of rutting and the cumulative number of equivalent 18-kip single-axle loads in each season (see Fig. 5).

Low-temperature transverse cracks are generally categorized as nontraffic-associated cracking. Such cracks are believed to

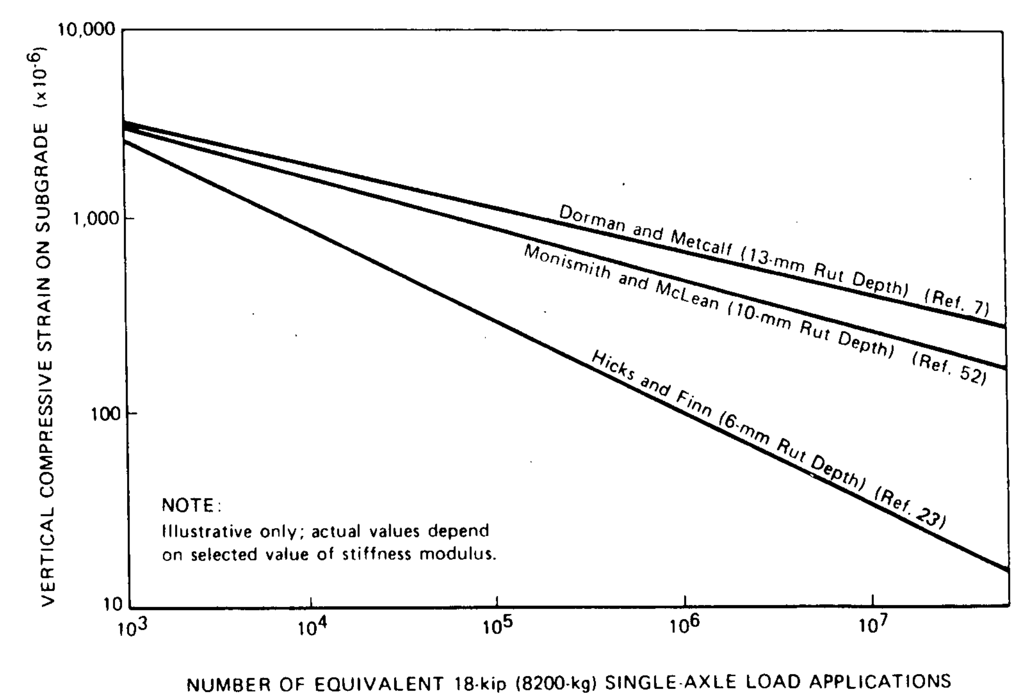

Figure 4. Rut depth prediction.

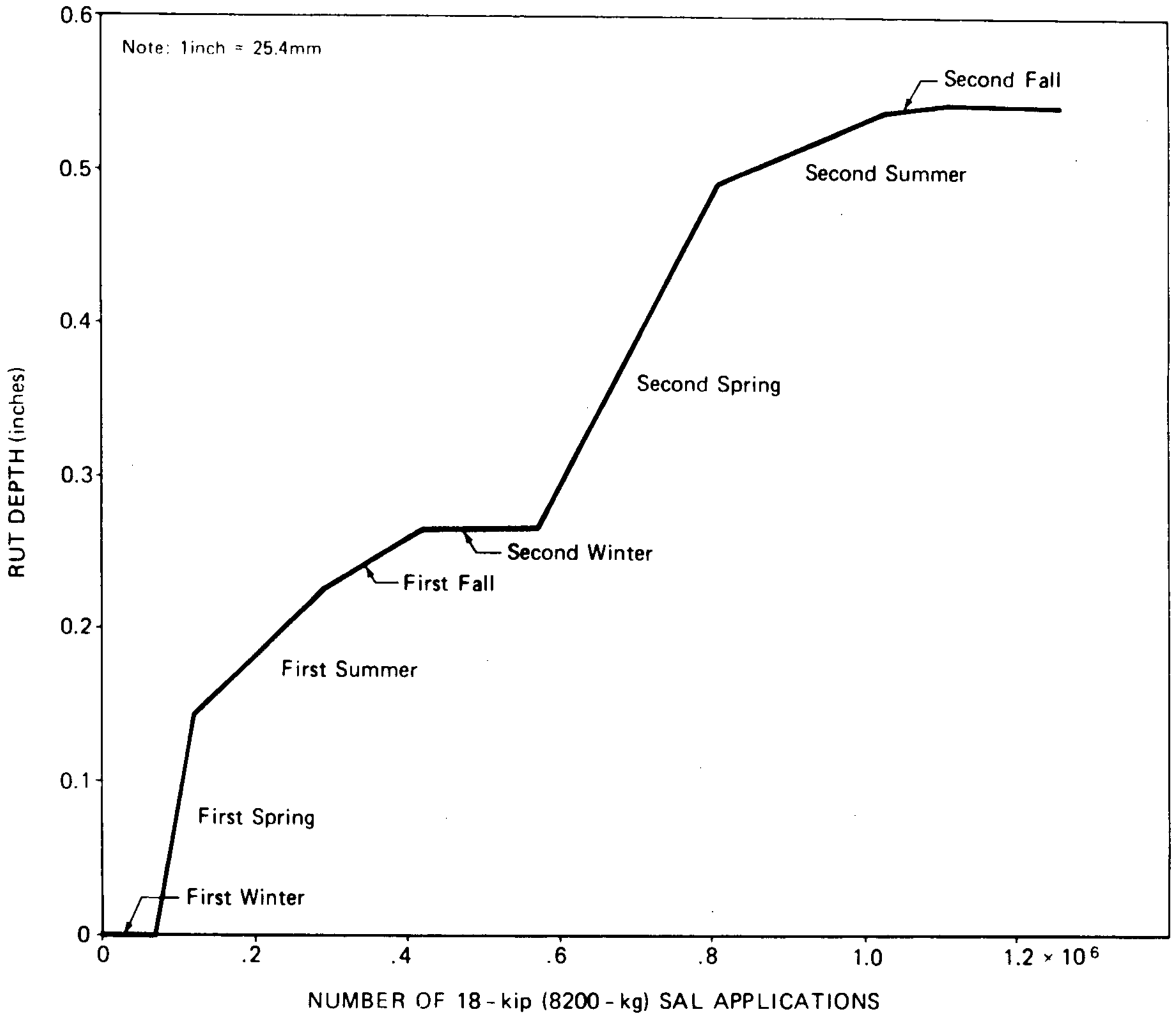

Figure 5. Typical rut-depth history for test sections on AASHO Road Test.

be caused by thermally induced tensile stresses in the asphaltic concrete. Factors that have been found to be contributing to thermal cracking include (1) asphalt stiffness, (2) asphalt grade, (3) asphalt temperature susceptibility, (4) asphalt source, (5) asphalt aging, (6) thickness of asphaltic concrete, and (7) subsurface materials.

The preponderance of research indicates that low-temperature cracking can be related to thermally induced tensile stress and the corresponding tensile strength. Thermally induced stress for this investigation was estimated using Eq. 1 in Chapter One.

Because of inherent limitations in the ability of mechanistic models to incorporate all of the factors influencing cracking and rutting, it is necessary to empirically adjust such models to reflect "real world" conditions.

The PDMAP and COLD subsystems can thus be characterized as being mechanistic-empirical procedures used to predict the occurrence of cracking and rutting in asphalt pavements. The subsystems have been designed, as much as possible, to simulate a pavement and to calculate the stress, strain, or deformation responses which cause distress. However, it is recognized that no model can perfectly simulate a pavement, considering the unpredictable effects of time and environment. Hence, the approach is to use the mechanistic formulations as a basis for correlation with field observations. It is generally believed that mechanistically based empirical models should be more reliable than the traditional-type models used for summarizing results from the AASHO Road Test.

PDMAP FOR PREDICTING FATIGUE CRACKS AND RUT DEPTH

The factors which have been incorporated in the PDMAP subsystem include (1) pavement section, (2) environment, (3) traffic, (4) materials characterization, (5) structural analysis, and (6) distress prediction models.

The *pavement section* refers to the thickness of the layers and material identification required for the structural analysis.

Environmental effects provide for temperature of the asphaltic concrete and the expected condition of supporting materials, i.e., water content and density of the zones influenced by traffic loadings. Environment is not a direct input to PDMAP; rather, it is indirectly included by appropriate techniques used for materials characterization.

Traffic input requirements are based on the conversion of mixed traffic to the equivalent number of cumulative 18-kip (80 kN) single-axle loads. AASHO load equivalency factors are used for estimating traffic for the first year, and a growth factor is assigned in order to accumulate loads for each subsequent year up to 25 years.

It is recognized that AASHO load equivalency factors were not developed from mechanistically based models. It is also recognized that load equivalency factors may be different for fatigue cracking and for rutting because different damage models are used for each. Deacon (*39*) has shown that mechanistically based (theoretical) load-equivalency factors for fatigue damage are reasonably similar to those found from the AASHO Road Test.

In order to facilitate computations, some acceptable deviation from theory has been introduced by using AASHO equivalency factors; however, such deviations can be minimized by empirical correlations with real world conditions. A thorough discussion of the sensitivity and methods for converting mixed traffic to equivalent axle loads can be found in *NCHRP Report 128* (*40*).

The PDMAP program requires that each component of the pavement structure, including the foundation, be characterized by linear elastic constants, i.e., modulus of elasticity and Poisson's ratio. Thus, *material characterization* procedures may have been designed to measure these characteristics. AASHO Test Method T274 may be used for soils, and ASTM Method D 3497 describes the method to be used for asphaltic concrete. Recognizing that paving materials are not necessarily linear elastic, the test methods provide for stress sensitivity of cohesive and noncohesive soils and for temperature and time of loading for asphaltic concrete.

The methods used to characterize elastic constants rely on the use of triaxial sample configurations such as that developed by Monismith and his associates at the University of California.

It is important to recognize that the damage predictions obtained from PDMAP are uniquely related to the methods used to characterize the various pavement layers and foundation soil. If a different method is used to characterize materials, different elastic constants will be obtained and new damage models will be required. Theoretically, the elastic constants should not be dependent on the method of testing; however, at the present time, experience indicates that measured properties are a function of the test method.

One of the major findings of this investigation is the critical importance of materials characterization. The ability to reproduce long-term in-place material properties poses a major problem to the implementation of these mechanistic procedures. More work is required in this area. A great deal of care in sample preparation and testing is required and sufficient replicate testing is recommended in order to develop the best possible representation of the in-place condition of materials with time. Thus, it is necessary to estimate initial conditions and annual changes in order to accurately simulate material properties. Field studies of water content and density changes with time are required for soils. For asphaltic concrete, the effects of aging on modulus-temperature and fatigue relationships should be investigated further.

The *structural analysis* model selected for PDMAP is a modification of the original Chevron five-layer program (CHEV5L) using principles of linear elasticity; provision was made for dual-tires superposition of loads and for stress sensitivity of both cohesive and noncohesive materials.

The program requires the following input information:

- Thickness of at least four structural layers.
- A representative indication of the modulus of elasticity for the structural layers and foundation soil for each damage period.
- Dual-tired loads with a specified inflation pressure.

The pavement structure must be divided into four layers plus the subgrade; the upper layer is always asphaltic concrete, the underlaying layers may be asphaltic concrete, emulsified asphalt mixes, cement-treated base, or untreated aggregate. Each layer of untreated aggregate is represented by the following stress sensitive relationship:

$$M_R = K_1 \theta^{K_2} \tag{4}$$

in which: M_R = resilient modulus; θ = first stress invariant ($\sigma_1 + 2\sigma_3$ in triaxial test); and K_1,K_2 = constants obtained from laboratory tests.

For nonstress-sensitive materials, K_2 is assigned a value of 0, thereby reducing M_R to a constant value equal to K_1.

The resilient modulus of cohesive soils (P.I.$>$0) used as subgrade or foundation for pavements has also been found to be stress dependent. The structural analysis procedure used by PDMAP uses the following relationship for this class of materials:

$$M_R = A\sigma_d^B \tag{5}$$

in which: M_R = resilient modulus, in psi, at a loading time of 0.1 sec; σ_d = deviator stress, in psi; and A,B = fitting coefficients dependent on material properties.

The elastic modulus for asphaltic concrete is variously referred to as stiffness, dynamic modulus (used by ASTM), and complex modulus (used by PDMAP). PDMAP needs modulus information for asphaltic concrete in the temperature range from 40°F (4°C) to 100°F (38°C) at a time of loading corresponding to 10 Hz.

The form of the *distress prediction model for fatigue cracking* of asphaltic concrete in PDMAP is as follows:

$$N_f = K_1 \left(\frac{1}{\epsilon}\right)^{K_2} \left(\frac{1}{|E^*|}\right)^{K_3} \tag{6}$$

in which: N_f = load applications to first crack: ϵ = maximum calculated tensile strain in lower fibers of asphaltic concrete; $|E^*|$ = complex modulus of asphaltic concrete; and K_1,K_2,K_3 = constants depending on material properties.

The procedure used to develop a prediction model was: (1) select a set of laboratory fatigue curves as a base case for crack initiation, and (2) calibrate the laboratory curves by means of a "shift-factor" to correlate with different levels of cracking based on field observations.

The base case fatigue curves selected were reported by Monismith et al. (*20*) and are shown in Figure 6. In order to simplify the computations, the Monismith laboratory fatigue curves were first converted into a family of parallel lines, shown in Figure 6 and represented by the following equation:

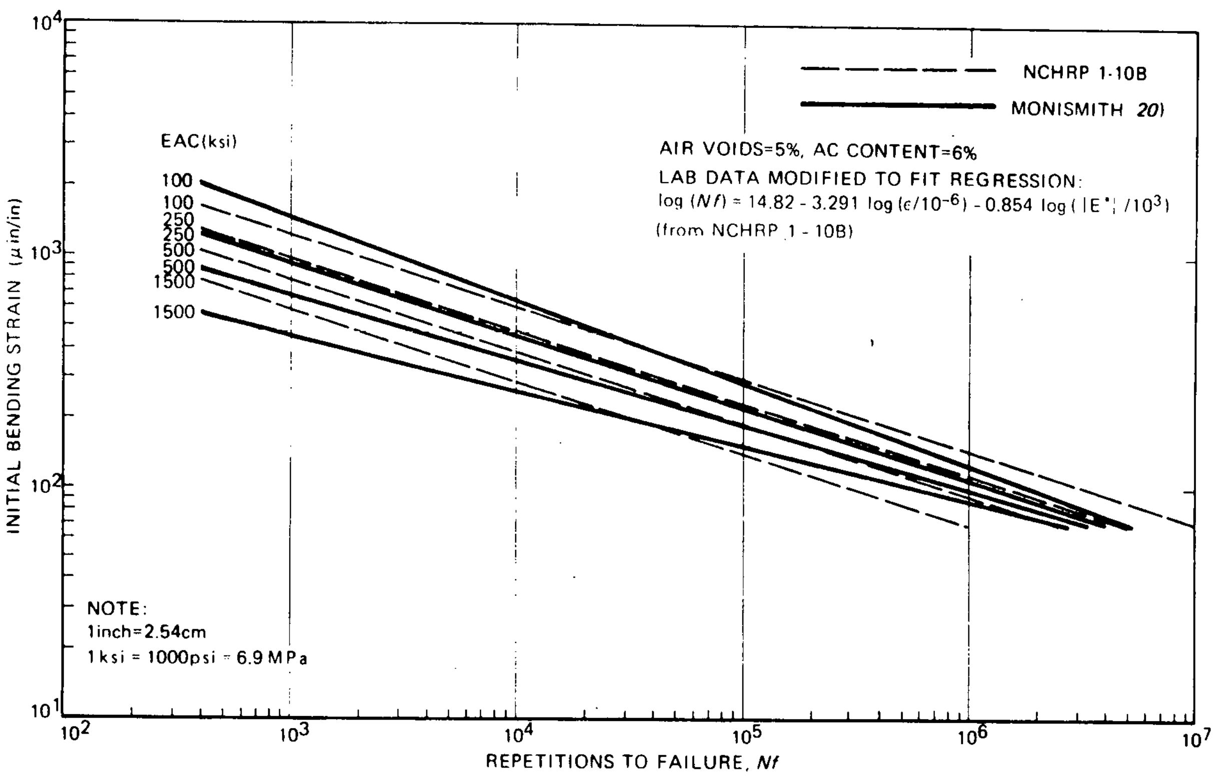

Figure 6. Fatigue lines for asphaltic concrete.

$$\log N_f = 14.82 - 3.291 \log (\epsilon/10^{-6}) - 0.854 \log (|E^*|/10^3) \quad (7)$$

in which: N_f = load applications of constant stress to cause fatigue failure; ϵ = initial strain on the underside of the asphalt concrete; and $|E^*|$ = complex modulus, in psi.

Initial efforts to calibrate the PDMAP prediction models for asphaltic concrete were based on observations of cracking and rutting at the AASHO Road Tests. The step-by-step procedure used to obtain a shift factor from observations at the AASHO Road Test is described in Appendix A. The shift factors obtained were 13.0 for 10 percent cracking and 18.4 for 45 percent cracking, where cracking is defined as the percent of the wheel path area exhibiting Class 2 cracking. Thus, 10 percent of the wheel path area would correspond to 5 percent of the total area.

The final prediction models for fatigue cracking, using materials similar to those used at the AASHO Road Test, are as follows:

$$\log N_f (10\%) = 15.947 - 3.291 \log (\epsilon/10^{-6}) - 0.854 \log (|E^*|/10^3) \quad (8)$$

$$\log N_f (45\%) = 16.086 - 3.291 \log (\epsilon/10^{-6}) - 0.854 \log (|E^*|/10^3) \quad (9)$$

No information comparable to the AASHO Road Test is available for the evaluation of *emulsified asphalt mixes.* Santucci (*48*) has presented relationships for fatigue distress developed from controlled stress and controlled strain laboratory tests on emulsified asphalt mixtures. These laboratory fatigue lines were shifted according to procedures recommended by Van Dijk (*49*) and Deacon (*50*). The final distress prediction model is given by the following equation:

$$N_f = 13.31 - 3.7058 \log (\epsilon/10^{-6}) - 0.6384 \log (|E^*|/10^3) \quad (10)$$

Mitchell and Monismith (*51*) have presented fatigue cracking distress criteria for *soil cement* that are appropriate to the models used in this investigation. Based on the results of the Mitchell and Monismith studies, a fatigue cracking model proposed for used with PDMAP is as follows:

$$N_f = 30.91 - 13.874 \log (\epsilon/10^{-6}) \quad (11)$$

Soil cement is defined as any mixture of soil and cement that will comply with wet-dry or freeze-thaw requirements of the Portland Cement Association when tested in accordance with ASTM procedures.

Data from the AASHO Road Test were used to develop the rut depth prediction model for thick asphaltic concrete pavements and for conventional sections, i.e., up to 6 in. (15 cm) of asphaltic concrete.

AASHO Road Test data consistently indicated that the rate of rutting, *RD/N,* was related to the season of the year (spring, summer, fall, winter) and that the rate of rutting for the second year decreased compared with the first year. Figure 5 illustrates the typical pattern of rut depth development recorded at the AASHO Road Test. Analysis indicated that the slope of the lines representing the rate of rut depth development per equivalent 18 kip (80 kN) could be related to surface deflection and to the vertical stress on the surface directly under the asphaltic concrete. The final equations proposed from AASHO Road Test data are:

Conventional construction—less than 6 in. of asphaltic concrete:

$$\log RR = -5.617 + 4.343 \log d - 0.167 \log (N_{18}) - 1.118 \log \sigma_c \quad (12)$$
$$(R^2 = 0.980 \quad \text{S.E.} = 0.316)$$

Full depth—equal to or greater than 6 in. of asphaltic concrete:

$$\log RR = -1.173 + 0.717 \log d - 0.658 \log (N_{18}) + 0.666 \log \sigma_c \quad (13)$$
$$(R^2 = 0.957 \quad \text{S.E.} = 0.174)$$

in which: RR = rate of rutting, micro-inches per axle load repetition; d = surface deflection $\times 10^3$ in. (S.I. notations not applicable for deflection and stress because regressions were made using inches for deflection and lb/sq in.); σ_c = vertical compressive stress at interface with asphaltic concrete, in psi; and N_{18} = equivalent 18-kip (80 kN) single-axle loads $\times 10^5$. For full-depth predictions, all traffic during frozen periods is neglected.

It is pertinent to note that the rut depth prediction model tends to be more empirically oriented than is fatigue cracking and hence can be expected to be more susceptible to load conditions. Equations 12 and 13 are limited to the test sections of the AASHO Road Test which exhibited relatively high levels of rut depths for rather normal levels of deflection.

In summary, PDMAP provides a subsystem for prediction of fatigue cracking and wheel path rutting for asphalt-type pavements. In order to be applicable to "real world" conditions, the prediction models were initially calibrated to results from the AASHO Road Test. It is expected that applications to areas using significantly different materials or subjected to different environments will require special calibration. This possibility will be discussed further in the verification section of this chapter.

One very important feature of the PDMAP system is the ability to incorporate the influence of the coefficient of variability for materials and traffic. Calculations summarized in Chapter Three indicate that the fatigue life could be reduced by approximately 30 percent, depending on levels of variability. An explanation of the procedure used to develop the influence of variability is given in Appendix F (see "Foreword" for availability).

COLD FOR PREDICTING LOW-TEMPERATURE CRACKING

The purpose of this subsystem is to estimate the potential for low-temperature cracking of asphaltic concrete for a particular asphalt–aggregate pavement structure when placed in a specific environment. The working hypothesis for this subsystem is that low-temperature cracking will occur when the thermally induced tensile stress exceeds the tensile strength. The procedures described herein as used for COLD were developed by Christison (*19*) and supported by efforts of Anderson et al. (*10*), Burgess et al. (*13*), and by Christison and Anderson (*14*).

The low-temperature prediction subsystem COLD is designed to compute thermally induced tensile stresses in the surface of the asphaltic concrete based on the thermal regime of the selected site. Tensile strength of the asphaltic concrete is obtained from laboratory tests.

Figure 7 illustrates the step-by-step procedure used in the COLD program. The trial thickness and mix design would evolve from the structural design and mix design procedures used by each agency.

Temperature in the asphaltic concrete is calculated by COLD based on the principles of heat transfer through a solid medium. The governing equation in this case is the one-dimensional heat condition equation describing the dissipation of heat in a homogeneous body and is of the following form:

$$\alpha \frac{\partial^2 T^2}{\partial x^2} = \frac{\partial T}{\partial t} \quad (14)$$

in which: α = diffusivity of the body; and T = the temperature of the body at a specified depth, x, and time, t.

The solution of Eq. 14 depends on the prescribed initial boundary conditions. In this case, temperature distribution throughout the pavement structure at a particular time (known) may be considered the initial condition. The boundary conditions describe the influence of the surroundings of the structure on its surface. At extreme depths in a pavement structure, changes in temperature with respect to depth and time become small in comparison to variations near the pavement surface; therefore, the assumption of a constant temperature at a specified depth for a given time period may be considered as a boundary condition for determining the temperature distribution in a pavement structure as a function of time. A relationship between various climatological factors and pavement surface temperature forms a second required boundary condition for the solution of Eq. 14.

Although amenable to a relatively simple description, the boundary conditions described above are very complex for an analytical solution of the equation. Therefore, a numerical solution technique (finite difference), with the use of digital computers, was adopted by Christison (*19*) to obtain a solution to the problem. Appendix C describes the computer program (COLD) that was developed to solve these equations and to determine the thermal regime of a given pavement structure.

Five different methods of analysis were studied by Christison (*19*) to estimate thermally induced stresses in any given pavement system. These methods are (1) pseudoelastic beam, (2) approximate pseudoelastic slab, (3) viscoelastic slab, (4) viscoelastic beam, and (5) approximate viscoelastic slab.

A comparison with the available data indicated that a suitable pseudoelastic beam analysis can yield reasonable results. Therefore, this method was adopted in the computer program (COLD) for estimation of thermally induced stresses in a given system. The stress equation for the pseudoelastic beam analysis is given as:

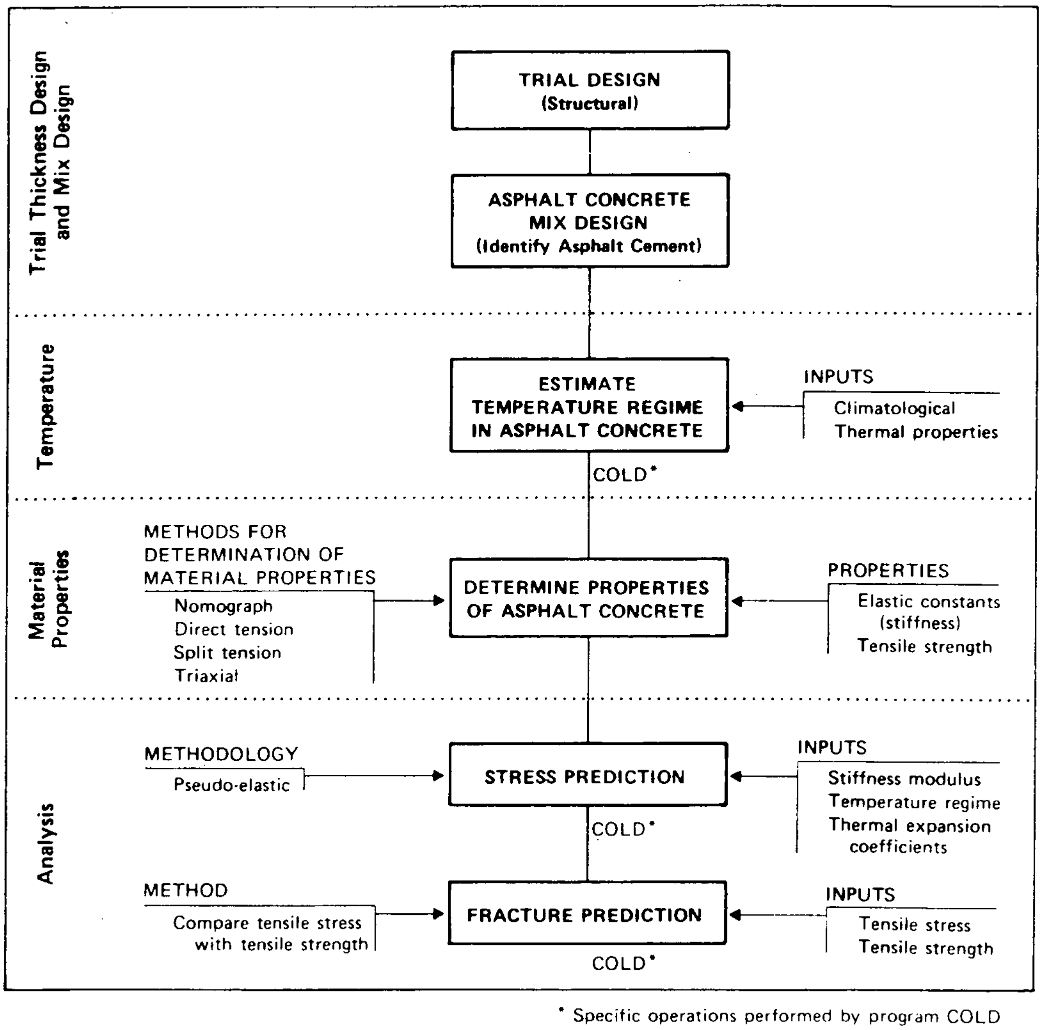

Figure 7. Low-temperature cracking subsystem.

$$\sigma_x(t) = -\int_t^t S(\Delta t, T)\alpha_o(T)dT(t) \tag{15}$$

where $S(\Delta t, T)$ is the time- and temperature-dependent stiffness modulus and $\alpha_o(T)$ is the coefficient of thermal expansion. Equation 16 is used to evaluate stress numerically:

$$\sigma_x(t_i) = \sigma_x(t_{i-1}) + (\Delta\sigma_x)_i \tag{16}$$

where:

$$(\Delta\sigma_x)_i = -\int_{t_{i-1}}^{t_i} S(\Delta t, T)\alpha_o(T)dT(t)$$

Using the trapezoidal rule, with stiffness evaluated at times t_i and t_{i-1}, and a loading time equal to the time step $t_i - t_{i-1}$, and assuming α_o temperature-independent, the stress increments in Eq. 16 are computed by the expression:

$$(\Delta\sigma_x)_i = \frac{-\sigma_o}{2}(S_i + S_{i-1})(T_i - T_{i-1}) \tag{17}$$

Computer program COLD solves the above equation and computes incremental stress for the specified time increments of one-eighth hour. These values are then accumulated and printed for every two hours of the day. Also, the program estimates the expected strength of the mix from the strength-temperature relationship, and the two values are compared to determine the possibility of thermal cracking at every two-hour time interval of the day for all the days included in the analysis.

In addition to deterministic estimates for cracking, provision for uncertainty based on variations in low-temperature creep modulus and tensile strength are incorporated in the program.

It should be noted that because low-temperature cracking was not observed at the AASHO Road Test, it was not possible to calibrate or evaluate the COLD subsystem with AASHO data.

Efforts to calibrate and evaluate the COLD program are discussed later in this chapter. Details concerning the development of the COLD subsystem are covered in Appendix A.

EVALUATION OF PDMAP AND COLD

In order to evaluate the applicability of PDMAP and COLD, an effort was made to extend the procedures to areas other than Illinois at the AASHO Road Test—specifically, to Florida and Utah.

The Florida Department of Transportation agreed to participate in a program designed to calibrate and evaluate PDMAP relative to fatigue cracking and rutting. The Utah Department of Transportation agreed to participate with evaluation of COLD. In both cases, the respective departments had been involved in research pertinent to the performance of asphalt pavements and specifically relative to the types of distress predicted by these two subsystems.

CALIBRATION AND EVALUATION OF PDMAP IN FLORIDA

The procedure used to calibrate and verify PDMAP in Florida involved two steps: (1) calibration at Chiefland test site, and (2) verification at Lake Wales and West Palm Beach test sites.

Calibration implies adjusting the base case prediction models to known field performance; verification amounts to testing the calibrated model with sections completely independent of those used for calibration.

CALIBRATION AT CHIEFLAND TEST SITE

In order to calibrate the PDMAP for the Chiefland test site, it was necessary to obtain information pertinent to (1) material properties, (2) climate, (3) in-situ conditions relative to density and water content, (4) traffic in terms of equivalent 18-kip single-axle loads during test period, (5) a history of the occurrence of cracking and rutting on each of the 24 test sections at the site, and (6) a damage prediction model for fatigue cracking.

All of the foregoing information was provided by the Florida DOT in accordance with specific requests made by the investigating team.

Pertinent information provided by the Florida DOT included the following:

- Layer thickness of each test section (Table 1).
- Materials used in test sections and their characteristics suitable for incorporation in the structural analysis program (Table 2).
- Traffic data (Table 3).
- Weather data (Table 3).
- Pavement condition data measured at different times (Table 1).
- Deflection data (Table 4). (Note: The ratio of the loads used for deflection measurements and for calculating deflections was 0.90, which corresponds almost exactly to the ratios of deflections shown in Table 4.)

Calibration of Fatigue Cracking Subsystem

The procedure used to calibrate the fatigue cracking subsystem was essentially the same as that used with data from the AASHO Road Test with one important exception—the base case for fatigue damage (crack initiation or approximately 1

Table 1. Chiefland test sections, layer thickness, materials and field survey data.

Section No.	Surface Layer	Lime Rock Base Thickness (in.)	12" Stabilized Sub-base LBR	Surface Deflection 10^{-3} in.	Fatigue Cracks per 1000 sq. ft.	Fatigue Cracks Date Observed	Rut Depth Observed on 03/28/74 (in.)
5A	2"	6	55	13.0	3.0	07/02/73	.24
10B	Thick	6		13.0	3.0	07/02/73	.20
9A	Type I	9		13.5	3.0	11/02/72	.19
2A	Mix	12		13.5	1.0	09/01/71	.17
3B		6	32	14.5	2.0	07/02/73	.36
1B		9		14.0	1.0	03/28/74	.19
6B		9		14.5	5.0	03/28/74	.27
12A		12		14.0	1.0	07/02/73	.13
8B		6	18	17.0	2.0	01/01/68	.43
11B		9		15.0	57.0	11/02/72	.23
4B		12		14.0	3.0	07/02/73	.23
7A		12		15.0	2.0	11/01/69	.27
5B	3"	6	55	14.0	1.0	09/05/68	.31
10A	Thick	6		14.0	125.0	11/02/72	.24
9B	Type I	9		14.0	1.0	09/01/71	.25
2B	Mix	12		13.0	0.0	03/28/74	.22
3A		6	32	15.0	1.0	07/02/73	.24
1A		9		11.0	0.0	03/28/74	.21
6A		9		13.5	5.0	04/01/69	.27
12B		12		14.5	3.0	11/01/69	.17
8A		6	18	15.0	2.0	04/01/69	.43
11A		9		14.0	117.0	11/02/72	.20
4A		12		14.0	3.0	10/17/73	.22
7B		12		14.5	1.0	11/01/69	.23

Table 2. Material properties used for PDMAP input.

Section Layer	Poisson's Ratio	Elastic Modulus, psi	
		Mix Temperature, °F	Dynamic Modulus, psi
Surface	.35	40	3,000,000
		50	1,900,00
		60	1,200,00
		70	720,000
		80	400,000
		90	185,000
		100	73,000
Base	.40	M_R = 8,200	θ .4173
Stabilized Sub-base			
50% LR	.40	M_R = 18,150	θ .3433
25% LR	.40	M_R = 14,400	θ .3433
0% LR	.40	M_R = 14,300	θ .3300
Subgrade (in situ)	.45	M_R = 12,450	$\alpha_d^{-.3300}$

Table 3. Summary of traffic and weather data used in PDMAP analysis of Chiefland test section 5.

Item	1st Season Oct. - April	2nd Season May - Sept.
Traffic		
6 am - 6 pm	24,000	30,000
6 pm - 6 am	12,000	14,000
Weather		
Mean Air Temperature, °F	61.0	79.0
Diurnal Range, °F	26.0	23.0
Mean Wind Velocity, mph	2.0	1.4
Solar Insolation, Langleys/day	375.0	500.0
Mean Sky Cover, tenths	5.3	6.5

Table 4. Measured and estimated surface deflections (10^{-3} in.), Chiefland project.

Section No.	Field Measurements (20K)	PDMAP Estimates (18K)	Ratio Est./Obs.
1A	11.0	12.5	1.136
1B	14.0	13.0	.929
2A	13.5	13.0	.963
2B	13.0	12.5	.962
3A	15.0	12.5	.833
3B	14.5	12.5	.862
4A	14.0	13.0	.929
4B	14.0	13.5	.964
5A	13.0	11.5	.885
5B	14.0	11.5	.821
6A	13.5	12.5	.926
6B	14.5	13.3	.917
7A	15.0	13.5	.900
7B	14.5	13.0	.897
8A	15.0	12.5	.833
8B	17.0	13.0	.765
9A	13.5	12.5	.926
9B	14.0	12.0	.857
10A	14.0	11.5	.821
10B	13.0	11.5	.885
11A	14.0	13.0	.929
11B	15.0	13.0	.876
12A	14.0	13.5	.964
12B	14.5	13.0	.897
		Mean	.9027

percent of area) was obtained directly from fatigue tests on undisturbed samples of asphaltic concrete from the test site. The relationship was reported by Sharma and Smith (*70*) and is given below for ready reference as follows:

$$\log N_f = 5.553 - 3.1805 \log \epsilon - 1.984 \log E^* \quad (18)$$

Using Eq. 18, the average cumulative damage associated with crack initiation was underestimated by factor of 1.68, i.e., on the average, the number of 18-kip single-axle loads associated with field observations of crack initiation was 68 percent greater than would be predicted from the laboratory fatigue curves. Thus, if the constant term in Eq. 18 was shifted by a factor of log 1.68, the model would be calibrated to average conditions. The prediction model obtained from the calibration is given as follows:

$$\log N_f = 5.778 - 3.1805 \log \epsilon - 1.984 \log E^* \quad (19)$$

Notations are identical to those used in Eq. 6.

The standard deviation for the shift factor was 0.62. The 95 percent confidence interval for the constant term in Eq. 18 would be from 5.26 to 5.64. One of the conditions which contribute to the large variation is the inherent variability in performance of test sections. To illustrate this, consider the performance of replicate sections at the Chiefland test site. Only two of six replicate sections performed alike; the variations in the number of months or amount of traffic to reach the same amount of cracking ranged from 1.8:1 to 2.8:1.

In addition to the variability in performance of replicate sections, it was also noted that, on occasion, thinner sections would out perform (greater number of load applications) thicker sections at the Chiefland test site. Analysis of the field data indicated a significant improvement in performance of sections surfaced with 2 in. of asphaltic concrete compared with 3 in. of asphaltic concrete, all other factors considered constant. Further analysis of field data indicated that the damage factor obtained from PDMAP could be enhanced by adjustment as a function of the surface and base thickness. To illustrate this possibility, a section-by-section correlation was developed for PDMAP shift factors as a function of the thickness of surface and base as follows:

$$\text{Shift factor (PDMAP)} = 2.442 - 0.575 (S_T) + 0.075 (B_T) \quad (20)$$

in which: S_T = thickness of asphaltic concrete, in.; B_T = thickness of aggregate base, in.; and $R^2 = 0.3167$.

To illustrate the effect of this adjustment, prediction models for a 3-in. (7.6 cm) asphaltic concrete and 12-in. (30.5 cm) limerock base are compared below with a 3-in. (7.6 cm) asphaltic concrete and 6-in. (7.6 cm) aggregate base section.

12-in. aggregate base:

$$S_F = 2.442 - 0.575(3) + 0.075(12) = 1.62 \quad (21)$$

6-in. aggregate base:

$$S_F = 2.442 - 0.575(3) + 0.075(6) = 1.17 \quad (22)$$

Converting into the mechanistic model according to Eq. 19 would produce the following relationships:

$$\log N_f(12) = 5.762 - 3.1805 \log \epsilon + 1.984 \log |E^*| \quad (23)$$

$$\log N_f(6) = 5.621 - 3.1805 \log + 1.984 \log |E^*| \quad (24)$$

By adjusting the Chiefland model for thickness, the standard deviation was reduced from 0.62 to 0.32 as summarized in Table 5. For 95 percent confidence interval, the constant term in Eq. 18 would range from 5.39 to 6.67, a significant improvement over the unadjusted shift factor.

In summary, if no adjustment for thickness is made, the shift factor should be increased by a factor of 1.68; if an adjustment is made, no shift is required. The need for the thickness adjustment suggests that the calculated tensile strain at the underside at the asphaltic concrete layer does not adequately model the fatigue performance of pavements at the Chiefland test site. Some investigators have suggested that additional computations involving shear strain should be incorporated in the damage models.

It is pertinent to note that there was no significant difference in the adjusted shift factor as a function of the limerock bearing ratio (LBR) value for this material. Apparently, the elastic constants did, in effect, adequately identify the influence of LBR values on fatigue cracking. The use of a shift factor is simply a technique to allow the mechanistic approach to be extended to field conditions. It does not, *per se,* provide a physical explanation for the lack of agreement between estimated and observed performance.

Table 5. Chiefland project, fatigue damage estimates.

Section No.	Surface Layer	Lime Rock Base Thickness (in.)	12" Stabilized Sub-base LBR	PDMAP Estimate of Fatigue Damage: Before Calibration	PDMAP Estimate of Fatigue Damage: After Calibration
5A	2"	6	55	1.8	1.03
10B	Thick	6		1.8	1.03
9A	Type I	9		2.0	1.02
2A	Mix	12		1.8	0.82
3B		6	32	2.1	1.21
1B		9		2.5	1.27
6B		9		2.5	1.27
12A		12		2.5	1.14
8B		6	18	0.7	0.40
11B		9		2.1	1.07
4B		12		2.5	1.14
7A		12		1.3	0.59
5B	3"	6	55	0.6	0.51
10A	Thick	6		1.4	1.20
9B	Type I	9		1.3	0.93
2B	Mix	12		2.3	1.42
3A		6	32	1.7	1.46
1A		9		2.0	1.44
6A		9		0.8	0.57
12B		12		1.0	0.62
8A		6	18	0.8	0.69
11A		9		1.7	1.22
4A		12		2.1	1.30
7B		12		1.0	0.62
			Mean	1.68	0.999
			SD	0.62	0.321

Calibration for Rut Depth Subsystem

The capability of the model represented by Eq. 12 to estimate rutting in the Chiefland test sections was determined by comparing the amount of rutting observed in the field with the PDMAP estimates as given in Table 6. As is evident from this table, the estimated values are very small compared to the observed values; therefore, calibration of this model was necessary to produce comparable estimates.

The ratio of observed to estimated values are given in Table 6. These values indicated that the estimated rate of rutting should be adjusted in a manner similar to fatigue damage shift factor in order to produce a rate of rutting that is comparable to field observations. This was accomplished by developing a relationship of the following type:

$$R_T = 302.2 - 26.33(S_T) - 14.12(B_T) \quad (25)$$

in which: R_T = ratio (observed rutting/estimated rutting); S_T = surface thickness, in.; and B_T = base thickness, in.

If the ratio for a given section is R_T, the rate of the rut depth model represented by Eq. 12 can be calibrated for this section by adding log R_T to the constant value of -5.617. The results of this calibration are reflected in the values of rut depth estimates as shown in the last column of Table 6. The average of the ratios is now 1.07 as compared to 1.09, when the model without calibration was used. The ratio estimated in this case represents the shift factor for the rut depth model.

To illustrate the adjustment, assume a section is constructed with 3 in. (7.6 cm) of asphaltic concrete and 9 in. (22.9 cm) of aggregate base; the constant in Eq. 12 would be increased from -5.617 to -3.634. After calibration the predicted rut depths would, on the average, be 7 percent greater than the observed rut depths as shown in Table 6.

In summary, the mechanistically based regression model developed from AASHO road test performance data required a large adjustment factor in order to provide reasonable predictions for sections at the Chiefland test site; however, once this adjustment was made, the predictions were satisfactory.

EVALUATION OF LAKE WALES AND WEST PALM BEACH TEST SITES

In order to evaluate the prediction models, an attempt was made to apply the damage equations from Chiefland to two

Table 6. Chiefland project, rut depths—observed and PDMAP estimated values.

Section No.	Surface Layer	Lime Rock Base Thickness (in.)	12" Stabilized Sub-base LBR	Observed on 03/28/74 (in.)	PDMAP Estimate Before Calibration (in.)	Ratio Observed / Estimated	PDMAP Estimate After Calibration (in.)	Ratio
5A	2"	6	55	.24	$.135 \times 10^{-2}$	178	.22	1.09
10B	Thick	6		.20	.135	148	.22	0.91
9A	Type I	9		.19	.175	109	.21	0.91
2A	Mix	12		.17	.208	82	.17	1.00
3B		6	32	.36	.191	188	.31	1.16
1B		9		.19	.224	85	.27	0.70
6B		9		.27	.224	121	.27	1.00
12A		12		.13	.251	52	.20	0.65
8B		6	18	.43	.203	212	.33	1.30
11B		9		.23	.230	100	.28	0.82
4B		12		.23	.257	89	.21	1.10
7A		12		.27	.257	105	.21	1.29
5A	3"	6	55	.31	$.192 \times 10^{-2}$	161	.27	1.15
10A	Thick	6		.24	.192	125	.27	0.89
9B	Type I	9		.25	.240	104	.23	1.09
2B	Mix	12		.22	.280	79	.15	1.47
3A		6	32	.24	.260	92	.36	0.67
1A		9		.21	.300	70	.29	0.72
6A		9		.27	.300	90	.29	0.93
12B		12		.17	.347	49	.19	0.89
8A		6	18	.43	.272	158	.38	1.13
11A		9		.20	.207	97	.20	1.00
4A		12		.22	.352	63	.12	1.83
7B		12		.23	.352	65	.12	1.92
					Mean	109		1.07
					SD	44		.32

different locations in the State of Florida: specifically, with the Lake Wales and West Palm Beach test sites. The following discussion summarizes this activity for both fatigue cracking and rutting at these locations.

Evaluation of Fatigue Cracking

Test sections at the Lake Wales site included ten thickness combinations with two different base materials. One set of test sections was constructed using limerock base with a replicate set of sections using sand asphalt hot mix (SAHM) produced in accordance with Florida DOT specifications. No replicate sections within base type were included at the site.

Construction of the test sections was completed in February 1971, and observations of distress were continued through December 1978. The asphaltic concrete thicknesses were 1.5 in. (3.8 cm) and 3 in. (97.6 cm); base thicknesses were 3, 4, 6, 8, and 10 in. (7.8, 10.2, 15.2, 20.3, and 25.4 cm).

The West Palm Beach test site constructed in February of 1970 included 11 test sections with three base types: (1) sand asphalt hot mix with three levels of Marshall stability, six sections; (2) shell base, three sections; and (3) limerock base, two sections. The asphaltic concrete thicknesses was limited to 1.5 in. (3.8 cm); SAHM base thicknesses were 3, 4.5, and 6 in. (7.6, 11.4, and 15.2 cm) replicated at two levels of Marshall stability, 800 and 1200 lb (3.6 and 5.3 kN); shell base thicknesses were 4, 6, and 8 in. (10.2, 15.2, and 20.4 cm); and limerock base thicknesses were 4 and 8 in. (10.2 and 20.4 cm).

Evaluation of the Chiefland model was primarily concerned with the limerock base sections. Attempts to use the Chiefland models for the sand asphalt sections indicated the model would not apply to this type of construction.

Equation 19, adjusted for thickness, was used for evaluation of the fatigue cracking model.

The calibrated Chiefland fatigue model underpredicted the fatigue performance of the Lake Wales limerock base sections by a factor of 1.56, as shown in Table 7. Thus, on the average the number of equivalent 18-kip (80 kN) single-axle loads required to produce initial cracking was 1.56 times the number of axle loads predicted by the calibrated PDMAP. One of the problems in comparing predictions and observations of fatigue cracking at Lake Wales was the absence of systematic trends between performance and pavement thicknesses as shown in Table 8.

The adjusted (calibrated) PDMAP fatigue models based on Chiefland performance data overpredicted the fatigue cracking on all of the test sections at the West Palm Beach test site; in other words, the number of load repetitions predicted was larger than those observed to first class 2 cracking. The calculated strains were extremely small, and yet, cracking was reported in all of the test sections after only 200,000 equivalent 18-kip single-axle loads. The majority of sections at the West Palm Beach site were constructed using materials unique to this site, i.e., sand-asphalt and shell base, with a thin layer of asphaltic concrete wearing surface. Because of these differences in construction and because the sections exhibited distress prematurely, it was considered inappropriate to use the West Palm Beach data for verification of the model.

In order to evaluate the effectiveness of the PDMAP predictions, a comparison can also be made with results obtained from the AASHO Road Test (*37*). Empirical equations developed to predict cracking observed on the Road Test had a mean residual of 0.18 for the log of axle load applications required to produce first class 2 cracks. Thus, observations whose residuals are less than the mean residual will range from 64 percent to 151 percent of the corresponding predictions for load applications. Based on these comparisons, the ability of PDMAP to predict cracking base on calibrated models projects, i.e., Chiefland to Lake Wales, is comparable to the reliability of predictions within the AASHO Road Test using empirical models. It is pertinent to note that the calibrated Chiefland model had a standard deviation of 0.33, or approximately two-thirds of the observations would be between 67 and 133 percent of the predictions; this would be considered a slight improvement over the AASHO Road Test correlations. It is also important to recognize that the AASHO predictions apply only to AASHO data, whereas the PDMAP predictions were made from one set of observations to a second set in two different locations.

EVALUATION OF RUT DEPTH SUBSYSTEM

The method used to verify the rut depth prediction model was similar to that for fatigue cracking, i.e., use the calibrated

Table 7. Summary of PDMAP analysis of Lake Wales project data.

Section No.	Layer Thickness (in.) Surface	Base	Subbase	Surface Deflection (in.) Observed*	PDMAP**	Fatigue Cracking[a] Observed[b]	PDMAP[c]	Rutting (in.)[a] Observed[d]	PDMAP
1A	3.0	10	12	.0090	.0104	--	--	.20	.26
1B	1.5	10	12	.0095	.0104	8/10/77	1.29	.22	.18
2A	3.0	4	12	.0110	.0100	8/10/77	3.15	.34	.35
2B	1.5	4	12	.0090	.0099	12/10/74	0.59	.26	.22
3A	1.5	3	12	.0100	.0098	4/18/78	1.16	.24	.23
3B	3.0	3	12	.0090	.0099	12/10/74	1.80	.30	.36
4A	3.0	6	12	.0110	.0101	9/2/75	1.95	.30	.33
4B	1.5	6	12	.0110	.0102	4/18/78	1.31	.24	.22
5A	1.5	8	12	.0110	.0103	8/10/77	1.24	.20	.20
5B	3.0	8	12	.0110	.0102	12/10/74	1.51	.34	.30
Average							1.56	.264	.265

Note: 1 inch = 2.54 cm

* 20K Single axle load was used in field.
** 18K Single axle load was used for PDMAP input.

[a] Calibrated model (Chiefland Project)
[b] Date associated when first fatigue crack was observed on the pavement.
[c] Ratio of observed number of months to first class 2 fatigue cracking to months predicted by PDMAP.
[d] End of 1978.

Table 8. **Months to initial cracking for Lake Wales, Florida, test sections.**

Thickness of Base in Inches (cm)	Thickness of Asphalt Concrete 1.5 Inches (3.8 cm)	3 Inches (7.6 cm)	Average
3 (7.6)	86	46	66
4 (10.1)	46	78	62
6 (15.2)	86	54	70
8 (20.2)	78	46	62
10 (25.4)	78	94	86
Average	74.8	63.6	

models from Chiefland to predict rutting at the Lake Wales test site.

The comparison of observed to predicted rut depth is shown in Table 7. These comparisons would appear to be acceptable; however, in this case, this conclusion could be misleading. The calibrated model at Chiefland was based on rut depths ranging from approximately 0.2 in. (0.5 cm) to 0.4 in. (1.0 cm), and rut depths at Lake Wales were of the same magnitude. Deflections at the two sites were also similar. Thus, the model was verified only for low deflections and relatively small amounts of rutting; it is not known how well the correlation would have been for high deflections and relatively large or small rut depths.

CALIBRATION AND EVALUATION OF LOW-TEMPERATURE CRACKING SUBSYSTEM IN UTAH

The data required for the calibration and evaluation of COLD were provided by the Utah Department of Transportation (UDOT). Ten pavement sections were selected for this purpose. Table 9 shows the structural properties of the test sections along with the year of construction and field observations of transverse cracking. Additional data collected for this part of the study are described in the following paragraphs.

The indirect tensile test method was used to obtain the tensile strength of the surface mix. Four-inch-diameter cores were obtained from each test section in the Utah DOT laboratory according to the procedure outlined in Refs. *14* and *19*. Typical temperature-stiffness-tensile strength information obtained from these tests is shown in Figure 8.

In the original developments of the COLD program, as described in Chapter One, the mixture stiffness was to be measured at a loading time of 7,200 sec. Using such values for Utah would not, in any case, predict cracking. The need for reduced loading times is believed to be related to the rapid rate at which the temperature changed (decreased) compared to experience in Canada. Asphalt aging and mixture design could also contribute to this problem.

The purpose of this phase of the investigation was to determine how well the COLD subsystem would perform for conditions in Utah and to modify the inputs to the model in such a way as to predict observations on selected projects in that state. Thus, in this case an effort was made to verify the model and then to calibrate to determine if some adjustments could be made which would correlate with field performance.

Table 9. Utah Department of Transportation test sections.

Section No.	Layer Thicknesses, inches: Surface & Seal	Base	Borrow Material	Year Constructed	Transverse Cracks No./1000'[c]
1[a]	7.75	4.0	-	72	16
2	7.75	4.0	-	72	11
3	7.75	4.0	-	72	0
4	4.75	4.0	-	61	0
5	4.75	4.5	-	62	24
6	5.75	4	6	73	23
7	8.75	4	-	70	0
8	4.75	4	-	70	20
9	4.75	-	-	75	0
10[b]	5.75	-	-	75	16

[a]1% hydrated lime.
[b]1% hydrated lime and Latex rubber.
[c]Observed in summer 1978.

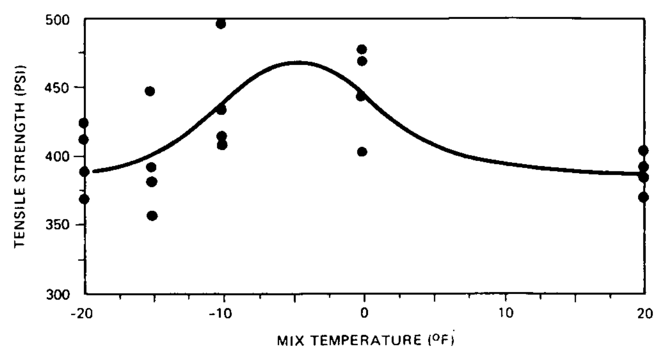

Figure 8a. Tensile strength–temperature relationship of mixture used in pavement number 2 (core).

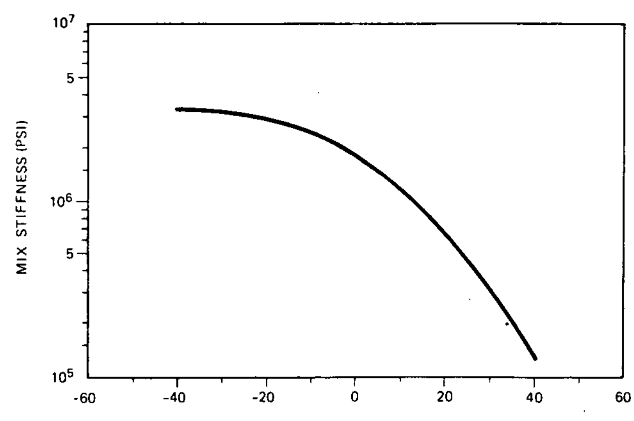

Figure 8b. Stiffness–temperature relationship of mixture used in pavement number 2 (10-sec loading).

Table 10. Typical input data for COLD (partial input data)—thermal properties of pavement layers.

Thermal Properties of Pavement Layers

Surface Layer

Absorptivity of surface	0.90
Emissivity of surface	0.95
Convection coefficient	2.70 Btu/sft/hr/°F

Layer No.	Description	Thermal Conductivity Btu/hr/°F Unfrozen	Frozen	Heat Capacity Btu/lb/°F Unfrozen	Frozen
1.	Surface layer, asphalt mix	.84	.84	.22	.22
2.	Untreated base and granular borrow	.75	1.10	.40	.30
3.	Subgrade	.65	1.25	.54	.37

Temperature Records and Solar Radiation

1st Extreme Temp at 2 hrs.
2nd Extreme Temp at 14 hrs.
Sunrise Time 8 hrs.
Sunset Time 16 hrs.

	1st Extreme Temp.	2nd Extreme Temp.	Solar Radiation
Dec. 27, 1972	10.0	30.0	500.0
Dec. 28, 1972	14.0	32.0	500.0
Dec. 29, 1972	12.0	30.0	500.0
Dec. 30, 1972	10.0	28.0	500.0
Dec. 31, 1972	8.0	28.0	500.0
Jan. 1, 1973	3.0	28.0	190.0
Jan. 2, 1973	0.0	28.0	190.0
Jan. 3, 1973	14.0	30.0	190.0
Jan. 4, 1973	-11.0	24.0	190.0
Jan. 5, 1973	-16.0	23.0	190.0
Jan. 6, 1973	- 8.0	22.0	190.0
Jan. 7, 1973	-14.0	20.0	190.0
Jan. 8, 1973	-13.0	20.0	190.0
Jan. 9, 1973	7.0	23.0	190.0
Jan. 10, 1973	12.0	27.0	190.0
Jan. 11, 1973	6.0	28.0	190.0
Jan. 12, 1973	18.0	40.0	190.0
Jan. 13, 1973	23.0	43.0	190.0

Thermal properties used to calculate induced stresses are summarized in Table 10. Temperature records were obtained from the monthly summaries of temperature for the State of Utah published by the U.S. Department of Commerce, National Oceanic and Atmospheric Administration.

A weather station close to each pavement was selected and temperature records were checked to find a period of about 15 to 18 days during which daily records of minimum temperature (after the construction of the pavement) indicated lowest minimum temperature. Records of minimum and maximum temperatures for each day were then used in the analysis.

Daily solar radiation data were not available from weather stations close to the test sites. Therefore, this information was obtained from the Climatic Atlas of the United States. Only monthly averages were available. Therefore, a constant average value of solar radiation was assumed for each month as shown in the example included in Table 10.

A trial analysis of data using mixture stiffness values estimated at 60-sec loading time indicated that the model used in the program will not predict cold-temperature cracking in most of the pavement sections included in the Utah evaluation; therefore, the mixture stiffness values were estimated at 10-sec loading in the analysis. This provided a means of calibrating the model to the field conditions observed in the State of Utah and represented by the data used in this study.

The results obtained from the calibrated model were further analyzed by estimating the probability of cracking assuming the

Table 11. Summary of COLD program outputs.

Pavement #	Observations of Field Cracks (No./1000 ft.)	COLD Program Predictions* Using 60-Sec. Stiffness	Using 10-Sec. Stiffness	Probability of Transverse Cracking (%)**	Density of Mix Cores PCF
1	16	No	Yes	54	147
2	11	No	No	41	151
3	0	No	No	45	153
4	0	Yes	Yes	83	137
5	24	Yes	Yes	88	135
6	23	No.	Yes	75	139
7	0	No	Yes	53	144
8	20	No	Yes	66	144
9	0	No	No	10	141
10	16	No	No	27	139

*No = No transverse cracking predicted with a probability of 50%.

Yes = Possibility of transverse cracking predicted with a probability of 50%.

**Based on coefficient of variation (C_v) of 25% on tensile strength and 50% on mixture stiffness estimated for 10-sec time of loading.

following values for the coefficients of variations, C_v, in the material properties:

- Tensile strength, C_v = 25 percent.
- Mixture stiffness, C_v = 50 percent.

The probabilities (%) thus calculated are given in Column 5 of Table 11.

Stiffness values of surface mixes used in pavement sections were estimated according to procedures of McLeod (*71*). Construction records were used to obtain information on the following items related to the mixture used in the construction of the test sections:

- Penetration of asphalt at 77°F.
- Viscosity of asphalt at 275°F (centistokes).
- Asphalt cement in the mixture (%).
- Air voids in the mixture (%).
- Voids in mineral aggregate (VMA).

The results shown in Table 11 indicate that if mix stiffness values are estimated at 60-sec loading, the predictions of thermal cracking will not be accurate. On the other hand, if the stiffness is estimated at 10-sec loading, the predictions of thermal cracking can be improved significantly.

Probability of cracking and number of cracks in the pavements are plotted in Figure 9a. This shows that the probability of cracking correlates to the frequency of cracking observed in the field for pavements having thermal cracks only. For those pavements that do not have thermal cracks, the predicted probability of cracking varied from 10 to 83 percent.

The symbols used in Figure 9a indicate that two different source asphalts were used in the Utah test sections. It can be concluded from these plots that the source of the asphalt will significantly influence the performance of the asphaltic concrete as measured by the occurrence of low-temperature transverse cracking.

The dependence of prediction of thermal cracking on a specific grade of asphalt was further substantiated from the plot shown in Figure 9b. Whereas all the pavements using source no. 1 could be clearly included in the plot of core density, versus number of cracks, pavements using different grades of asphalt obtained from other sources showed no relationship between density and cracking frequency.

SUMMARY

Based on the results of this investigation, the ability to predict fatigue cracking from laboratory data and using the PDMAP subsystem would be considered fair providing proper adjustments are made. In making such an evaluation, it is important to recognize the inherent variability of performance of test sections used in the evaluation phase.

A general summary of the results is as follows:

1. The base case fatigue damage model from laboratory tests can be calibrated for site-specific locations with an acceptable degree of precision.
2. The calibrated PDMAP prediction model underpredicted the load cycles to initial fatigue cracking at Lake Wales.
3. The predictions for West Palm Beach test sections overpredicted load cycles to first crack and no specific explanation can be made at this time. West Palm Beach test sections were subjected to a relatively small amount of loadings (171,000 EALs); had more traffic been applied, it is possible that more distress would have occurred and improved the comparison.
4. More research is required in characterizing the elastic properties of granular materials. The stress sensitivity relationship used for granular materials in PDMAP penalizes granular bases. Efforts to subdivide the base layer into two thinner layers did not correct this tendency. The difficulty arises because the effective modulus of the aggregate base decreases as the thickness of the base increases.
5. The use of deflection matching as a means of calibrating materials characteristics, at least for limerock aggregates and stabilized sections typical of materials used in Florida, does not appear to adequately adjust material properties for local conditions.
6. The inherent variability of the fatigue performance of asphalt pavements makes it extremely difficult to predict fatigue cracking for site-specific cases. Some improvement is possible as the sample size increases; however, the data from this investigation would indicate that the PDMAP subsystem will require some further refinement before it can be broadly applied as a precise prediction model.
7. The rut depth prediction model is considered to have some merit once it was calibrated to limerock base materials in Florida. The relatively small amount of rutting and low deflections reported for the Florida sections has not provided a real test of the model. However, the concept that the rate of rutting is a function of surface deflection, vertical stress, and cumulative traffic is considered reasonable and justifies further evaluation.
8. The low-temperature crack prediction model, as originally proposed—i.e., 7,200-sec loading time—requires further evaluation. One consideration not specifically included in the evaluation of the COLD model is the effect of aging on stiffness. For this study, stiffness was based on the properties of the original unaged asphalts. However, the pavement performance

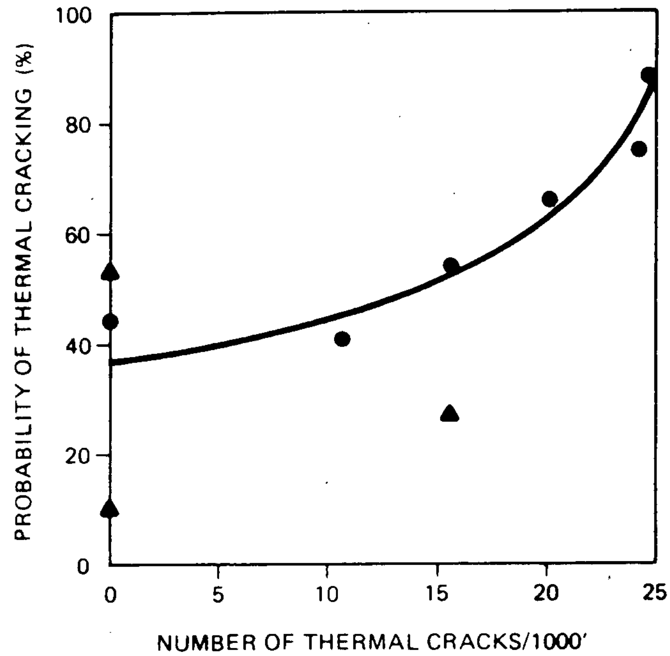

Figure 9a. Probability of thermal cracking—number of cracks/ 1,000 ft.

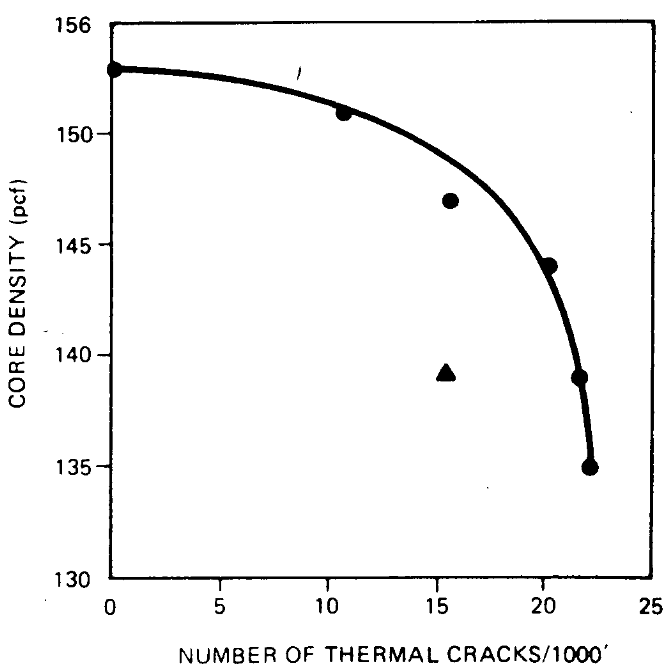

Figure 9b. Core density—number of cracks/1,000 ft.

is based on aged asphalts. In order to obtain a better estimate of the aged asphalts, the time of loading was decreased rather than estimating changes in asphalt properties. Thus, the most likely change required to improve the reliability of the model will be to use aged properties of the asphalt.

Based on the findings summarized in this chapter, it can be concluded that mechanistic-empirical models, while still in need of improvement, are as reliable as, or more reliable than, empirical models, and with some additional research effort will prove to be a more versatile and dynamic method to predict pavement distress (performance).

CHAPTER THREE

APPRAISAL AND IMPLEMENTATION

The PDMAP and COLD programs developed in this project use mechanistic response factors as primary determinants for predicting distress. It is considered highly unlikely, however, that any such prediction models can ever be expected to be developed without some consideration of field experience. Thus, the development of both programs has incorporated information from field studies; unfortunately, the sources of adequate field data to calibrate and evaluate mechanistic models are very limited.

Chapter Two of this report describes the calibration and evaluation of PDMAP and COLD for a limited number of cases in Florida and Utah, respectively. In this chapter the two programs will be appraised by examples of the type of information required to implement the programs and will provide information indicating the output and use of output from the two subsystems.

It is pertinent to note that the initial development of both models did incorporate information from field trials. Specifically, as noted in Chapter Two, PDMAP was calibrated from performance data obtained at the AASHO Road Test; the COLD program was initially calibrated to results obtained from tests in Canada (Edmonton and Ste. Anne Test Road) and reported by Christison and Anderson (*14, 19*).

Based on the methodology used in the development of the two subsystems, an appraisal of the various elements is provided in this chapter. Because the data base was relatively small, it is emphasized that appraisal of the subsystems will require a more intensive effort than is included in this report. A description of procedures to be used for implementation will also be provided. Because both PDMAP and COLD programs are designed to be modular, it is possible to modify the distress prediction model without rewriting the whole program.

APPRAISAL OF SUBSYSTEM PDMAP

There are three basic sets of input information required for PDMAP: climate, materials characterization, and traffic.

In order to facilitate an appraisal of PDMAP, a number of simulated pavement sections were analyzed and distress predictions made and discussed. Two hypothetical examples will be used for the appraisal of PDMAP (fatigue cracking and rutting). The setting for these examples is Minneapolis, Minnesota, and Concord, California—two widely different climatological areas of the United States. Climatological information was obtained from Ref. *54*. Weather data are summarized in Table 12. Assumed traffic information for two time periods during each day in the initial year of the example is given in Table 13. A range of designs is included in the examples, including sections of conventional base and subbase designs, no base with thick asphaltic concrete, and full-depth asphaltic concrete.

Assumed values of resilient modulus relationships for the roadbed soil (subgrade material) are given in Table 14. Modular relationships for asphaltic concrete, aggregate base, and subbase are given in Tables 15, 16, and 17, respectively.

Table 18 is a summary of output information obtained from PDMAP using conditions and materials summarized in Tables 12 through 17.

Climate

Climatological information is used in PDMAP to calculate the temperature regime in the asphaltic concrete.

The Barber program used for this calculation has been verified by a number of investigators (*15, 55*). Pavement temperature predictions are considered acceptably reliable for use in PDMAP, and no further work should be required. Solar radiation information is difficult to obtain from weather stations in the United States at present; therefore, it is necessary to consult a climatological atlas such as Ref. *54*.

Predictions of distress, in Table 18, for environmental conditions for Minnesota and California suggest that the differences in the occurrence of fatigue cracking are not as great as would be expected and that, in fact, there is a greater potential for cracking in California. This prediction tends to be contradictory to what might be expected. The prediction is a function of the distress model developed from AASHO Road Test data. The results also reflect the fact that no damage occurs to the pavements in Minnesota when the ground is frozen; thus, some damage-free traffic is assigned to these sections.

Table 12. Climatological information for PDMAP examples.

Area	Time Period	Mean Air Temp.	Daily Range	Wind Velocity (mph)	Solar Radiation (Langleys)	Sky Cover (Percent)
Minnesota	Nov - Feb	20°F (-7°C)	20°F (±11°C)	11	150	7.0
	Mar, Apr	40°F (4°C)	20°F (±11°C)	13	400	7.0
	May - Aug	70°F (21°C)	25°F (±14°C)	10	500	5.0
	Sept, Oct	55°F (13°C)	25°F (±14°C)	10	300	5.5
California	Nov - Apr	45°F (7°C)	20°F (±11°C)	7	300	5.5
	May - Oct	70°F (21°C)	30°F (±17°C)	11	600	3.0

Table 13. Traffic data for PDMAP examples.

Time Period	EAL[a] per Month	Growth Rate
6 a.m. - 6 p.m.	4500	1.05
6 p.m. - 6 a.m.	3000	1.05

[a]Equivalent 18-kip (8200-kg) single-axle loads.

Table 14. Seasonal moduli for subgrade materials used in PDMAP examples.

Area	Time Period	Resilient Modulus (psi)[a]
Minnesota	Nov - Feb	50,000
	Mar - Apr	$4{,}000\ \sigma_d^{-0.7}$
	May - Aug	$12{,}000\ \sigma_d^{-0.7}$
	Sept - Oct	$8{,}000\ \sigma_d^{-0.7}$
California	Nov - Apr	$6{,}000\ \sigma_d^{-0.7}$
	May - Oct	$12{,}000\ \sigma_d^{-0.7}$

[a] 1 psi = 6.9 kPa.

Table 15. Complex moduli of asphaltic concrete.

Asphalt No.	Complex Modulus (psi)[a] 40°F (4.4°C)	70°F (21.1°C)	100°F (37.8°C)
1	1.3×10^6	4.5×10^5	1.0×10^5
2	1.8×10^6	7.0×10^5	2.2×10^5

[a]1 psi = 6.9 kPa.

Table 16. Resilient modulus of aggregate base.

Area	Time Period	Resilient Modulus (psi)[a]
Minnesota	Nov - Feb	50,000
	Mar - Apr	$5{,}000\ \theta^{0.6}$
	May - Aug	$10{,}000\ \theta^{0.6}$
	Sept - Oct	$8{,}000\ \theta^{0.6}$
California	Nov - Apr	$6{,}000\ \theta^{0.6}$
	May - Oct	$12{,}000\ \theta^{0.6}$

[a]1 psi = 6.9 kPa.

Table 17. Resilient modulus of aggregate subbase.

Area	Time Period	Resilient Modulus (psi)[a] 1[b]	2
Minnesota	Nov - Feb	50,000	50,000
	Mar - Apr	$5{,}000\ \theta^{0.6}$	$1{,}500\ \theta^{0.6}$
	May - Aug	$10{,}000\ \theta^{0.6}$	$5{,}000\ \theta^{0.6}$
	Sept - Oct	$8{,}000\ \theta^{0.6}$	$3{,}000\ \theta^{0.6}$
California	Nov - Apr	$6{,}000\ \theta^{0.6}$	$3{,}000\ \theta^{0.6}$
	May - Oct	$12{,}000\ \theta^{0.6}$	$5{,}000\ \theta^{0.6}$

[a] 1 psi = 6.9 kPa.
[b] Subbase properties considered identical to base.

Table 18. Summary of PDMAP outputs.

Number	Section	Minnesota				California			
		Fatigue Damage		Rut Depth		Fatigue Damage		Rut Depth	
		Years	D.F.[a]	Years	Inches[b]	Years	D.F.	Years	Inches
				Using Subbase Type No. 1 from Table 17					
1	4-8-20	12	1.0	25	0.3	10	1.1	25	0.3
2	4-8-25	13	1.0	25	0.2	11	1.1	25	0.2
3	4-8-30	14	1.0	25	0.1	11	1.0	25	0.1
4	4-8-35	15	1.1	25	0.1	11	1.0	25	0.1
				Using Subbase Type No. 2 from Table 17					
5	4-8-20	7	1.0	14	0.8	6	1.0	25	0.7
6	4-8-25	7	1.0	14	0.7	6	1.0	25	0.5
7	4-8-30	7	1.0	21	0.7	6	1.0	25	0.4
				No Base, Subbase Type No. 2 from Table 17					
8	6-0-12	8	1.0	9	0.7	6	1.1	9	0.8
9	9-0-12	22	1.0	25	0.4	21	1.1	25	0.5
10	12-0-12	25	0.2	25	0.1	25	0.4	25	0.1
				Thick Asphalt Concrete with Subbase Type No. 2 from Table 17					
11	6-6-20	15	1.0	24	0.7	12	1.1	25	0.5
12	12-6-6	25	0.2	25	0.1	--	---	--	---
				Full-Depth Asphalt Concrete with Asphalt No. 1 from Table 15					
13	8-0-0	14	1.0	18	0.7	8	1.0	12	0.8
14	10-0-0	25	0.8	25	0.4	25	0.5	25	0.3
15	12-0-0	--	---	--	---	25	0.5	25	0.3
				Full-Depth Asphalt Concrete with Asphalt No. 2 from Table 15					
16	8-0-0	25	1.0	25	0.5	18	1.0	25	0.7
17	10-0-0	25	0.3	25	0.2	25	0.5	25	0.2

[a] D.F. = Damage Factor equal to 1.0 indicates up to 10 percent cracking.
[b] 1 inch = 25.4 cm.

In obtaining predictions of distress for the two geographical areas, it was estimated, on the basis of AASHO Road Test data, that the spring thaw modulus properties for the same subgrade and stress conditions in Minnesota would be 50 percent lower than the fall modulus and 67 percent lower than the summer modulus. Similar estimates, although less severe, were made for base and subbase (Tables 16 and 17). It was also assumed that the pavement was frozen for 4 months of the year in Minnesota, possibly an excessively long period. It is pertinent that the spring damage rate in fatigue is approximately four times that found for assumed summer conditions.

The distress prediction is also a function of Eq. 8, developed to predict fatigue damage. More will be said of the influence of Eq. 8 in a subsequent section of this chapter.

Material Characterization

Material characterization is extremely important to the output obtained from PDMAP. To illustrate the influence of material properties, reference can be made to Table 18.

Sections 1 through 4 simulate a pavement constructed with base-quality material for subbase, whereas sections 5 through 7 use number 2 subbase materials (Table 17). A significant difference in the occurrence of both rut depth and fatigue cracking can be noted. The fatigue life to 10 percent cracking is increased by approximately 100 percent and the rut depth is less than 50 percent by using aggregate base throughout the granular layers.

Sections 13 through 15 simulate an asphaltic concrete using an AC 10 asphalt, similar to that used on the AASHO Road Test, whereas sections 16 and 17 reflect the use of a somewhat stiffer asphalt. In these comparisons, a full-depth asphaltic concrete section was used to illustrate the influence of asphalt stiffness. The asphalt with the higher stiffness is shown to provide significantly improved fatigue life for the two environmental conditions used in this example.

It is pertinent to note that the differences in material properties could be real or could be due to errors in measuring or estimating modular values. Thus, it can be concluded that material characteristics used in the model will influence the output and should be carefully evaluated if predictions are to be reliable.

A key factor in the evaluation of material properties is the ability to (1) reproduce in the laboratory specimens for conditions that exist in the field, and (2) know what field conditions really are for a particular area.

Some of the more important conditions to be evaluated include (1) the influence of saturation and whether or not to test under conditions that can develop pore pressure due to traffic loadings, (2) simulation of spring thaw, and (3) seasonal and long-term changes in material properties. Not much is known in detail regarding the foregoing factors, and more field information is needed.

For purposes of this project, testing of granular materials was accomplished by drained triaxial dynamic testing, thus eliminating the possibility of pore pressure. This may have been an incorrect assumption, but it was based on the best judgment of the staff and consultants as well as studies reported by Monismith et al. (*56*). Variations in seasonal subgrade properties were based on fall and spring deflections at the AASHO Road Test.

Provision for hardening of the asphalt cement has been made by assigning properties by groups of years, i.e., 0 to 3, 4 to 10, greater than 10. The ability to predict aging is somewhat limited at the present time; however, hypothetical cases can be simulated, as desired, in order to estimate the role of asphalt aging on fatigue cracking and rutting.

One of the major problems in developing a distress prediction model for overlays is the inability to characterize the in-place treated layers. Arbitrary assignments of values for damaged areas have been made by the project staff; however, the ability to reliably measure such values remains to be developed. Diametral testing equipment, such as that proposed by Schmidt (*43*) or Kennedy (*42*), is the most convenient way to measure in-place properties of undamaged asphaltic concrete; however, such measurements do not provide information as to the effective modulus of damaged asphaltic concrete. Also, the diametral modulus does not reproduce values obtained from triaxial testing; therefore, new distress models would be required, depending on the method of testing.

Traffic

Traffic input using equivalent 18-kip (80.1 kN) single-axle loads provides a significant simplification for the prediction of distress. If unit damage factors for incremental loads were added to the computational requirements of PDMAP, the costs would become excessive except for application to special case studies.

The use of AASHO equivalencies for both fatigue-cracking and rut-depth predictions has posed a problem because these equivalencies were influenced primarily by riding quality. If separate equivalencies were to be used, a significant computational effort would be required. If at some future time such refinements are considered necessary, appropriate subroutines can be formulated.

In order to overcome some of the deficiencies related to traffic equivalencies, a considerable amount of reliance is placed on the use of empirical correlations to obtain fitting coefficients necessary to satisfy field observations.

Structural Analysis

The program used to predict the primary response of a pavement is a development of the CHEV5L layered-systems programs. This program, called PSAD, includes both superposition and stress sensitivity. PSAD is the only layered-systems program with this dual capability. There are two weaknesses in the PSAD program: (1) the limitation to five layers, and (2) the use of default factors when computations indicate tension in the granular layers.

Comparison of the PSAD program with the more powerful finite-element program (SAPIV) indicates comparable results for horizontal strain in the asphaltic concrete and pertinent stress predictions. Some differences do occur in computations of surface deflection in specific situations.

The PDMAP program is modular as regards possible substitutions of future developments in structural design procedures. When making such substitutions, care should be exercised in recalibrating the prediction models used to predict distress.

Distress Prediction Models

The development of distress prediction models for fatigue cracking and rut depth for PDMAP has depended on results obtained at the AASHO Road Test.

Future users of this program will need to calibrate both prediction models, using local experience as the basis of such calibrations. Descriptive procedures for such calibrations will be provided later in this chapter. Procedures for calibration to conditions other than the AASHO Road Test are given in Chapter Two.

One of the probable adjustments for the fatigue distress model is the sensitivity for damage as a function of the stiffness of the asphaltic concrete. Adjustments in the K_2 and K_3 constants can be used to obtain a greater (or smaller) spread in the distress prediction lines (Fig. 3) and in the slope of those lines. Hence, the basic model has considerable potential for adjustments to fit local experience.

There are a number of applications for which PDMAP can be used:

- Simulation
  - Climate
  - Material properties
  - Wheel loads
  - Layer combinations
  - Influence of variability in material properties and distress prediction model
- Management systems
- Structural design guides
- Diagnosis of structural failures

Table 18 illustrates the ability of the PDMAP program to predict distress for different climates (Minnesota versus California), different materials (untreated aggregates of two different qualities), and asphaltic concrete (two levels of stiffness). All of these could have some influence on design policies within a given user organization. For example, sections 1 through 7 tend to indicate the increased efficiency of higher quality (increased modulus) aggregates and asphalt stabilized base.

PDMAP may be used for design determinations on specific projects, particularly if new materials or unique conditions (construction, climate) are anticipated. However, PDMAP will have a greater utility in developing design guides or standards. For example, in Table 18 the design requirements for three construction sections (i.e., thin, thick, and full-depth asphaltic concrete) can be used to establish a general relationship for alternate designs of these types. By means of expanded parametric solutions, simplified tabular or graphic solutions can be obtained that will eliminate repetitious solutions for individual projects. The Asphalt Institute used the concepts established in PDMAP to update their design manuals in 1981.

The use of the PDMAP damage models for overlays will require evaluation of the material properties of the damaged asphaltic concrete in the original pavement. The use of deflection measurements and "back calculation" methods to estimate in-place material properties provides at least an estimate or "seed" value which can be adjusted with experience. Reference *53* describes at least one method for using mechanistic procedures for overlay design.

APPRAISAL OF SUBSYSTEM COLD

The ability to predict low-temperature cracking in asphaltic concrete from the COLD program depends on the ability to reliably (1) predict the thermal regime in a pavement subjected to low ambient temperatures, (2) predict thermally induced tensile stress, and (3) measure tensile strength at low temperatures. Three factors need discussion in the appraisal of this system: the ability to recognize low-temperature cycling, the influence of the rate of change in temperature, and the possible interaction of the supporting layers.

To date there is a very limited amount of information to show the influence of low-temperature cycling on the occurrence of low-temperature cracking. It has been concluded in this investigation that asphalt hardening with time may be just as appropriate as temperature cycling; hence, such procedures have not been incorporated in this subsystem.

The rate of change in temperature could play an important role in predicting temperature-induced tensile stresses. It is apparent that there could be an infinite number of possibilities. Christison and Anderson (*14*) and Christison (*19*) used 7,200 sec for the time of loading in developing the procedures used in COLD. This time of loading is equivalent to a 5°F (2.7°C) change in temperature per hour. This rate of change is somewhat faster than the 10,000 and 20,000 sec suggested by other Canadian investigators.

The program is not tied to any specific rate of loading. If faster rates of loading are considered appropriate, such as discussed in Chapter Two, it is only necessary to obtain stiffness values for such rates as seem most applicable. If higher stiffness values are obtained, the program will compute higher thermally induced stresses and cracking predictions at higher temperatures.

The COLD subsystem predicts the potential for low-temperature cracking and does not attempt to predict the frequency of cracking. Haas (*12*) has proposed an empirical nomograph for predicting cracking frequency. Since there is no mechanistic base for such prediction, it is not included in this subsystem.

The utility of the COLD subsystem can be illustrated for the example used for PDMAP. In the PDMAP example, two levels of asphaltic concrete stiffness were assumed as a function of the grade of asphalt used. Table 19 indicates that low-temperature cracking is likely to occur if asphalt number 2 is used; however, by using asphalt number 1, this problem can be avoided. Table 18 indicates that either mixture would be satisfactory for a 10-in. (25.4 cm) full-depth asphaltic concrete design; however, excessive rut depths could develop for the 8-in. (20.3 cm) section with asphalt no. 1. Modification in mixture design or structural design could be required in some situations in order to avoid excessive cracking or rutting.

On the basis of this investigation, some further studies are indicated to be necessary in order to produce a reliable prediction model applicable to a variety of environmental conditions. Canadian engineers report some satisfaction with the working hypothesis used in the COLD program. However, as indicated in Chapter Two, it was necessary to reduce loading times by orders of magnitude before stiffness was sufficiently increased to produce thermally induced stresses that would exceed tensile strength. If properties of an aged asphalt were known, it is possible that the aging alone would have increased the stiffness comparable to that obtained by reducing the time of loading. To be useful, the model should work with unaged materials because that is what the designer knows best and can analyze with some degree of certainty. It would appear, however, that some provision for aging will be necessary in order to improve the prediction capability of the model.

Table 19. COLD program outputs.

1. Section — 10" Full Depth.
2. Site — Minneapolis, Minnesota.
3. Weather Conditions — Typical month of January.
4. Asphalt Mix Stiffness — Typical values shown below. These values were calculated for a loading time of 7200 seconds.

Mix Number	Typical Mix Stiffness, psi 20°F	0°F	-20°F	-40°F	Low Temperature Cracking Prediction
1	48,000	280,000	930,000	2,000,000	No
2	370,000	1,600,000	3,700,000	5,700,000	Yes

In addition to aging, there are a number of additional factors to consider with regard to low-temperature cracking:

1. Low-temperature cycles or the possible effect of fatigue, as suggested by Shahin and McCullough (*27*).
2. Rate of change in temperature, particularly in the low-temperature zone.
3. Possible effect of wind on surface temperature, i.e., wind-chill factor.
4. Ability to adequately measure stiffness and tensile strength at low temperatures.

Nevertheless, the results of this study do indicate that a mechanistic procedure for predicting low-temperature cracking is available and can be useful, providing the right input parameters are used. The problem, of course, is to be able to identify those parameters during the design stage of a specific construction project.

The COLD subsystem should be very useful in the following applications:

- Developing asphalt specifications.
- Selecting asphalt binders to minimize low-temperature cracking.
- Diagnostic estimates of potential for low-temperature cracking for a given asphalt and known environment.

One very important feature of PDMAP and COLD is the ability to incorporate the influence of the coefficient of variation on material properties and traffic. Table 20 illustrates this capability for PDMAP. The probability of low-temperature cracking was illustrated in Chapter Two in connection with the Utah evaluation phase of the project.

From information such as that provided in Table 20, it is possible to evaluate the effect of uncertainty or variability on the performance of a pavement.

IMPLEMENTATION

The subsystems represented by PDMAP and COLD are a combination of procedures designed to predict distress. For PDMAP, a degree of evaluation has been obtained by using field data to obtain the necessary fitting coefficients from the AASHO Road Test and from Florida test sections. This is primarily the case for fatigue-cracking and rut-depth predictions. Low-temperature cracking tends to be more mechanistic and less dependent on field correlations and has been evaluated with prediction comparisons in Edmonton and Ontario (Ste. Anne), Canada, and for pavements in Utah.

Implementation can be subdivided into three phases: (1) familiarization with programs, (2) parametric evaluation of program outputs, and (3) calibration of fitting coefficients to conform with local experience.

Familiarity with Computer Programs

This step will involve a review of the User's Manual for each of the subsystems and the use of these programs on available computer hardware to assure that they are locally operational.

In all probability, this will be relatively straightforward and should not be the source of any difficulty. The NCHRP offices have access to copies of the programs which can be provided at a nominal cost.

Parametric Evaluations

The objective of this step is to obtain an indication of the type of predictions that could be obtained for typical designs, traffic, and environmental conditions within the experience of the user agency. Such trials would help establish the need for adjustments in the damage prediction relationships. This is particularly applicable to PDMAP.

Every effort has been made to develop damage prediction models that are amenable to modification on the basis of field observations. For example, Eq. 6 in Chapter Two presents the general equation for fatigue distress as follows:

$$N_f = K_1 \left(\frac{1}{\epsilon}\right)^{K_2} \left(\frac{1}{|E^*|}\right)^{K_3}$$

in which K_1, K_2, and K_3 are fitting coefficients obtained from laboratory and field correlations. Equation 7 shows these coefficients to be 14.82, 3.291, and 0.854, respectively. The position, slope, and spread of this damage prediction relationship can be adjusted by modifying these fitting coefficients. For example, if K_1, K_2, and K_3 are assigned values of 13.95, 1.5, and 2.5, respectively, the slope will be significantly steeper, the spread in stiffness effect will be similar to that proposed by Kingham (*22*) and Witczak (*9*), but the position will remain essentially constant.

One of the most significant factors to consider is the shift factor as generally represented by K_1. It may be that K_1 will be found to be a function of the thickness of the surface and treated base. If such is the case, a relatively simple addition to the program could adjust the shift factor as a function of the thickness of treated layers.

For the COLD subsystem, adjustments may be necessary in terms of the rate of which temperature changes occur, specifically, the time of loading or as a function of the aging index for the asphalt.

Table 20. Influence of material properties and traffic variability on PDMAP predictions[a].

Section	Coefficient of Variability (percent)		Fatigue Cracking (year) Reliability			Rut Depth[b] (inches) Reliability		
	Materials	Traffic	50%	75%	99%	50%	75%	99%
4-8-20[c]	50	10	11	10	8	0.1	0.2	0.4
		25	11	10	8	0.1	0.2	0.4
	100	10	9	8	7	0.2	0.6	1.4
		25	9	8	7	0.2	0.6	1.4
8-0-0	50	10	14	12	9	0.7	0.8	1.0
		25	--	--	--	--	--	--
	100	10	12	11	9	0.6	0.8	1.0
		25	12	11	9	0.7	0.8	1.1

[a] Based on 250 daily equivalent single axle loads and 5 percent growth in traffic.

[b] Estimated for corresponding number of years indicated in respective fatigue columns.

[c] 4-8-20 represents 4 inches (10.2 cm) of asphalt concrete, 8 inches (20.3 cm) of aggregate base, and 20 inches (50.8 cm) of aggregate subbase material.

Field Trials

The final stage for implementation will involve the selection of field trials; that is, test sections for which the following information is or can be made available: (1) traffic, (2) climate, including freeze and thaw periods, (3) material properties, and (4) condition surveys made with sufficient frequency to identify when and how much rutting or cracking occurs. For the COLD subsystem, it will be necessary to know almost daily during the winter season when cracking occurred and the temperature history some 5 or 10 days preceding the occurrence of cracking.

The major effort in the field trials will be to calibrate the subsystems further in a manner similar to that described in Chapter Two and Appendix A.

A major consideration will be the determination of the seasonal moduli of the subgrade materials. This can best be accomplished from deflection data in conjunction with knowledge of the moduli of the structural layers. Additional information pertinent to obtaining modular values is given in Appendix A.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

This investigation has developed two structural subsystems designed to (1) predict fatigue cracking and rutting (PDMAP) and (2) predict low-temperature cracking (COLD) in pavements constructed with asphaltic concrete surface courses. The working hypothesis for each of the subsystems is based on the assumption that an asphalt pavement can be modeled as an elastic layered system, providing certain adjustments can be made to adequately characterize materials of construction. The prediction models within each subsystem are described as mechanistic—i.e., they incorporate principles of physical science which treat the action of forces on bodies. In the models used for this investigation, the response variables are stress, strain, and deflection resulting from traffic loads or thermal conditions.

The models used to predict responses are considered representative of paving materials and in-service conditions, as described in Chapters One and Two. Provision has been made for (1) environment, (2) in-place conditions (density, water content, etc.), (3) variance of traffic and material properties, (4) changes in traffic and material properties with seasons, (5) traffic growth, and (6) aging of materials. However, inclusion of these factors in the models does not assure that they, in fact, incorporate or represent all of the factors that could influence the types of distress being considered. Also, some simplifying assumptions have been made in order for the models to be useful for practical applications. For example, mixed traffic has been combined into equivalent 18-kip single-axle loads using the AASHTO equivalency factors. Also, in order to incorporate seasonal effects or effects of aging, it has been necessary to make assumptions with regard to the amount of time to assign to seasons or the rate of change in properties after years of service. The models themselves are not limited with regard to the number of seasons to use within a year; theoretically, predictions could be made for each day of the year and for succeeding years up to a total of 25 years. Practically, it would be impossible to measure or realistically assign changes in properties on such a time scale.

For reasons cited above, a considerable amount of importance must be given to the use of empirical information to calibrate the models; thus, rather than refer to the models as being purely mechanistic, a more descriptive reference would be mechanistic-empirical models.

As part of the development package, an effort was made to evaluate the subsystems in stages: (1) installation on agency computers, and (2) trial calculations and field trials.

The PDMAP and COLD programs were furnished to the Minnesota, Arizona, and Florida Departments of Transportation for installation on mainframe computers. In each case the computer programs were made workable with a minimum of adjustments for different operating requirements of the computers. On the basis of this experience, both PDMAP and COLD are judged to be compatible with most hardware available to user agencies. The CPU times for PDMAP will depend on two factors: (1) use of mainframe or mini-computer, and (2) run in deterministic or probabilistic mode. On a mainframe, the deterministic mode will require only a few CPU seconds; a probabilistic mode could require 1 to 2 min, depending on the type of hardware used. The COLD program can be run on a PC in approximately 15 sec.

Trial calculations with both PDMAP and COLD have been described in Chapter Three. These calculations demonstrate the versatility of the mechanistic-empirical approach. The ability to computer-simulate pavement performance should prove to be a very useful tool in developing new design procedures, diagnosing premature pavement distress, or evaluating the potential benefits of new materials or concepts pertinent to pavement design or construction.

Field trials in Utah and Florida have been used to evaluate the ability of PDMAP and COLD to be adapted to local conditions. While these efforts were not, in all respects, as satisfactory as would be desired, they indicated that (1) the subsystems can be calibrated to local conditions, and (2) mechanistic-empirical methods should be superior to pure empirical or pure mechanistic methods.

The investigation is believed to be sufficiently encouraging to justify further research efforts in order to develop reliable, useful, and usable mechanistic-empirical prediction models.

RECOMMENDED RESEARCH

Research is recommended in six areas: (1) sample preparation and testing, (2) measuring properties of in-situ materials, (3) structural analysis, (4) prediction model for permanent deformation, (5) calibration of distress prediction models for both fatigue and permanent deformation, and (6) trial implementation and evaluation.

Sample Preparation and Testing

Improved procedures for sample preparation are needed in order to reliability represent field conditions. Specifically, in order to be able to simulate field conditions in the laboratory, it will be necessary to research pavement conditions throughout the year, as well as from year to year. Specific examples are seasonal variations in water content, pore pressure in various structural layers, conditions appropriate to the spring thaw period, and long-term changes in the properties of asphaltic concrete systems as they occur in the field.

Measurement of Material Properties In Situ

In order to be able to monitor material properties in situ, it will be necessary to develop (1) laboratory methods to prepare and test materials in their natural environment, or (2) nondestructive methods of testing material in place to provide properties similar to those obtained in the laboratory. The latter approach is preferred because it is faster and likely to be much less expensive. The use of deflection measurements to estimate in-place modular values are available; however, only limited information is available regarding correlations between laboratory results and nondestructive procedures. The properties need to be comparable in order to be useful in the damage model used to predict distress.

Methods to predict stiffness of aged asphaltic concrete at low temperatures at slow rates of loading or different rates of change in temperature are needed to enhance the COLD subsystem.

Structural Analysis

The structural analysis program used in PDMAP represents what is believed to be a reasonable and relatively simple method to estimate a pavement's response to traffic and environment. There are three areas that need to be improved: (1) provision for at least nine layers, including the semiinfinite layer of foundation material, (2) a more rational method for including stress sensitivity for untreated materials, and (3) greater flexibility in providing for mixed traffic, axle configurations, and contact pressure. Any improved procedure should continue to include provision for superposition of loads.

It is considered that provision should be made for nine layers as follows: (1) asphaltic concrete overlay, one layer; (2) original asphaltic concrete surface, two layers; (3) aggregate base, two layers; (4) aggregate subbase, two layers; and (5) subgrade materials, two layers.

The present method for evaluating stress sensitivity involves averaging the first stress invariant ($\sigma_1 + \sigma_2 + \sigma_3$) at 27 points within each layer of the untreated granular layer; when these sums equal zero or less, a dummy variable of zero is assigned to the combined stress at that point and averaged with the remaining states of stress. A better way of accounting for stress sensitivity is needed. Finite-element solutions are theoretically the appropriate alternative; however, they were considered too expensive at the time PDMAP was developed. Recent advances could make such techniques a viable alternative.

Permanent Deformation Model

The models for predicting permanent deformation as used in PDMAP must be classified as "expedient" at the present time. The elastoplastic techniques are preferred, because plastic deformation can be traced to individual elements and conditions of the pavement. However, the proposed approach is somewhat less complicated and requires no additional input beyond what is required for the fatigue-cracking subroutines.

Surface deflection plus compressive stress tends to yield the best correlations with AASHO Road Test observations and was reasonably indicative of the rate and magnitude of rutting on the Florida test sections, provided the models were calibrated to field observations. The correlation to rutting is predicated on the assumption that increased pavement deflection creates a physical state with the greatest potential for permanent deformation; i.e., the rate of rutting is proportional to deflection and cumulative amount of truck traffic.

The methodology employed in PDMAP is not unreasonable and could prove to be extremely useful. User agencies should carefully evaluate the model employed in PDMAP and seek adjustments through field observations and local experience. Considerably more care in extrapolation to new materials and traffic conditions should be exercised in the application of the rut-depth model when compared to fatigue or low-temperature cracking.

Calibration of Damage Models

A key to the use of mechanistic-empirical models is the calibration of the damage model and the technique to combine damage into a cumulative distress model. Figures 4, 5, and 6 represent damage models of the type used in PDMAP. The correctness of such models depends on their ability to model the pavement response and to allow the program to predict when distress will occur. The damage model and method for accumulating damage is the "bottom line," and requires much more attention than it has been given in the past.

Calibration of the model is the most expensive part of the development of a prediction model; it requires extensive testing to establish estimated properties of in-place materials with time, as well as careful, frequent, and systematic measurements of performance.

Calibration of the COLD subsystem also requires further research. There are several models for predicting low-temperature cracking; each should be given some opportunity for evaluation based on field trials. The method proposed in COLD is relatively simple and believed to have merit; however, more evaluation is required.

Trial Implementation

Field trials to verify both PDMAP and COLD represent the single most important research need. Some important aspects of field trials for PDMAP are included in the previously described research needs, such as seasonal changes in material properties.

The COLD subsystem needs more field verification and closer examination of the influence of (1) rate of change in air temperature, (2) number of low-temperature cycles, and (3) effects of aging on the prediction model.

SUMMARY

The efforts described under this investigation represent a compilation and combination of research made possible by many competent investigators. The results are encouraging and should be sufficient to justify additional research.

The PDMAP models for fatigue cracking and rutting provide a procedure for relating pavement design to distress. The design criteria to be used will depend on the limiting amount of cracking or rutting to be associated with a particular section. Thus, the procedure for design will require the following steps: (1) establish the level of cracking or rutting, (2) estimate thickness requirements based on experience or alternate design procedures, (3) obtain information pertinent to material properties, and (4) obtain, from PDMAP, the associated ESAL's corresponding to assigned levels of distress. It will be necessary to run the program several times in order to bracket the estimated traffic for the site. Designs at different levels of reliability can be determined from PDMAP output. The COLD program will estimate the probability of low temperature cracking, for a particular mixture and asphalt in a specific environment. If the results indicate a medium-to-high probability of cracking, it may be necessary to respecify the grade of asphalt to be used or to change the source to obtain a less temperature-susceptible asphalt.

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Appendix A

DEVELOPMENT AND USE OF PDMAP AND COLD SUBSYSTEMS

One of the principal developments in the PDMAP programs was the development of the damage models for fatigue cracking and rate of rutting for the rut prediction model. For the COLD subsystem a preliminary evaluation using simplified procedures is suggested prior to initiating a full computer and laboratory analysis. This appendix describes these developments for users who are interested in implementing the structural subsystems.

An important aspect of PDMAP and COLD is the capability to predict damage at various levels of reliability. Some confusion arises as to how the coefficient of variability influences the predictions. It might be expected that a 100 percent coefficient of variability on material properties would decrease the life cycle in fatigue by more than 30 percent as is suggested by PDMAP. An explanation of how probabilities influence predictions is provided in this appendix.

Included in this appendix is information that will be useful in estimating material characteristics (elastic constants) in the absence of laboratory data. It is emphasized that this information is provided only as a guide for trial evaluations or qualitative studies. Direct measurements should be considered essential for implementation.

DAMAGE MODEL FOR FATIGUE CRACKING

The procedure for obtaining a fatigue model was dependent on two analyses: (1) representative fatigue lives for laboratory data, and (2) development of a shift factor to provide results compatible with field observations.

These developments are summarized in Chapter Two of this investigation. Nothing further is needed as regards the laboratory curves; however, a detailed description of the procedures used to obtain the shift factor will be given. These techniques should prove useful to users for future calibrations based on field trials.

As has been mentioned, the shift factor was obtained from information related to the AASHO Road Test, and discussions and references to data related exclusively to that project.

The maximum horizontal tensile strain in the treated layer was determined with the PSAD program that is described in the report on this investigation. There were 19 sections (7 from loop 4, 12 from loop 6) involved in obtaining the shift factor; damage was calculated for each month and with two time periods for a typical day in the month. It was necessary to develop a regression equation to predict tensile strain from PSAD in order to reduce the number of individual structural analysis computations. These equations were developed to include the expected range of layer thicknesses and material properties. The equations used were as follows:

$$\log \varepsilon = 3.355 - 0.72219 \log T_1 - 0.089108 \log T_2 \qquad \text{(A-1)}$$
$$- 0.065293 \log T_3 - 0.53784 \log E_1$$
$$- 0.26563 \log E_2 - 0.12667 \log E_3$$
$$- 0.07358 \log E_4 + 0.45913 \log L$$
$$R = 0.9933 \quad S.E. = 0.0236$$

$$\log d = 1.746 - 0.49644 \log T_1 - 0.089549 \log T_2 \qquad \text{(A-2)}$$
$$- 0.090574 \log T_3 - 0.16402 \log E_1$$
$$- 0.11523 \log E_2 - 0.13222 \log E_3$$
$$- 0.57113 \log E_4 + 0.83490 \log L$$
$$R = 0.9985 \quad S.E. = 0.01401$$

in which:

ε = maximum horizontal tensile strain in asphalt layer, microinches/inch;*

d = vertical surface deflection between dual wheels, 10^{-3} inches;

E_1 = complex modulus $|E^*|$ of surface layer, psi x 10^5 (30,000 to 2.75 x 10^6 psi);

E_2 = modulus of base layer, ksi (15 to 50 ksi);

E_3 = modulus of subbase layer, ksi (7 to 50 ksi);

E_4 = modulus of subgrade, ksi (3 to 50 ksi);

T_1, T_2, T_3 = thickness of surface (3 to 6 inches), base (3 to 9 inches), and subbase (0 to 12 inches);

L = axle load, kips (18 or 30);

*Note: 1 inch = 2.54 cm; 1 psi = 6.9 kPa; 1 kip = 454 kg.

R = multiple correlation coefficient; and

S.E. = standard error.

Material properties used to calculate the primary responses for the various test sections were determined by the Asphalt Institute as a substantial contribution to the project. In all cases, tests were performed using materials from the AASHO Road Test. Special acknowledgment is made to the Illinois Department of Transportation for samples of the aggregates and to Chevron Research (Richmond, California) for the asphalt.

Tables A-1 and A-2 summarize the results of dynamic modulus values for the various asphalt concrete systems used on the AASHO Road Test. These data are representative of density and void content values after traffic had been applied to the pavements on the project. For all practical purposes, and within the ability to estimate the mean values, the moduli of the surface and binder courses can be considered equal. It is worth noting that the asphalt contents of these two systems differed by 0.9 percent (surface course, 5.4 percent; binder course, 4.5 percent); however, the void contents were approximately the same (3.8 and 4.1 respectively).

The resilient modulus for the untreated base and subbase was expressed in terms of the first stress invariant as indicated in Equation 4.

TABLE A-1

COMPLEX MODULUS OF AASHO ROAD TEST SURFACE COURSE

TEMPERATURE	DYNAMIC MODULUS (PSI x 10^5) LOADING FREQUENCY: 1 Hz	4 Hz	16 Hz
40°F (4.4°C)	9.98	12.3	14.0
70°F (21.1°C)	2.32	3.53	5.30
100°F (37.8°C)	0.62	0.89	1.29

Note: 1 psi = 6.9 kPa.

TABLE A-2

COMPLEX MODULUS OF AASHO ROAD TEST BINDER COURSE

TEMPERATURE	DYNAMIC MODULUS (PSI x 10^5) LOADING FREQUENCY: 1 Hz	4 Hz	16 Hz
40°F (4.4°C)	9.25	11.7	14.3
70°F (21.1°C)	2.09	3.25	5.10
100°F (37.8°C)	0.66	0.88	1.21

Note: 1 psi = 6.9 kPa.

Tests on base aggregate were made at density values representative of the average construction values as reported in Reference 57, Tables 32 and 33. Samples were tested at the optimum reported water content (4.2 percent) after attempting to saturate the specimens (5.6 percent) and after air-drying to a constant water content (1.5 percent). The M_R relationships were as follows:

$$M_R(1.5\%) = 8000\ \theta^{0.6} \quad \text{(A-3)}$$

$$M_R(4.2\%) = 4000\ \theta^{0.6} \quad \text{(A-4)}$$

$$M_R(5.6\%) = 3200\ \theta^{0.6} \quad \text{(A-5)}$$

in which M_R = resilient modulus, in psi.

Tests with the subbase aggregate were performed at the reported optimum water content of 3.8 percent and at a condition approximating saturation (6.4 percent). The M_R relationships were as follows:

$$M_R(3.8\%) = 5400\ \theta^{0.6} \quad \text{(A-6)}$$

$$M_R(6.4\%) = 4600\ \theta^{0.6} \quad \text{(A-7)}$$

As with untreated granular soils, it has been found that the resilient moduli of fine-grained silts and clays are also stress-sensitive. The most convenient and representative relationship of this dependency is given in equations of the following form:

$$M_R = A\sigma_d^B \quad \text{(A-8)}$$

in which

M_R = resilient modulus, psi, at a loading time of 0.1 sec;

σ_d = deviator stress, psi; and

A, B = fitting coefficients.

A detailed description of this test can be found in Appendix H.

Water content and dry density values of laboratory specimens were based on average field construction control data. Specimens were prepared by kneading, impact, and static compaction procedures. No significant differences in moduli were reported for the three compaction procedures. This similarity would be expected when compaction is accomplished at or near the optimum water content.

The best fit to an equation of the form suggested by Equation A-8 was as follows:

$$M_R = 25{,}000\ \sigma_d^{-1.06} \qquad \text{(A-9)}$$

Austin Research Engineers reported data (53) obtained from undisturbed cores taken from the AASHO Road Test subgrade in 1974 that could easily be pooled with the laboratory data reported by the Asphalt Institute, which provides good verification for the resilient modulus relationship of the subgrade.

In order to estimate the seasonal variations in the modulus of the subgrade materials, a procedure suggested by McCullough (58), and detailed in Reference 53, was used. In this method a trial-and-error procedure (iterative) is used to adjust the subgrade modulus in order to obtain a reasonable match to measured deflections on the surface of the pavement.

For the investigation reported herein, the calculations were made using seasonal modulus values of the structural layers as shown in Table A-3. To a degree, the determination of the adjusted seasonal values was

TABLE A-3

ELASTIC MODULI OF AASHO ROAD TEST MATERIALS

MATERIAL	SEASONAL MODULI (PSI)			
	OCT-NOV (FALL)	MAR-APR (SPRING)	MAY-AUG (SUMMER)	DEC-FEB (WINTER)
Asphalt Concrete $\lvert E^*\rvert$	0.45×10^6	0.71×10^6	0.23×10^6	1.7×10^6
Temperature (^oF)	70	59	85	30
Base, M_R	$4000\ \theta^{0.6}$	$3200\ \theta^{0.6}$	$3600\ \theta^{0.6}$	50,000[a]
Subbase, M_R	$5400\ \theta^{0.6}$	$4600\ \theta^{0.6}$	$5000\ \theta^{0.6}$	50,000[a]

Note: 1 psi = 6.9 kPa.

[a]Assigned values assuming frozen conditions.

based on a certain amount of judgment. However, insofar as possible, data from Reference 37 (e.g., seasonal deflections, water content, and CBR values) were used in selecting moduli. The moduli of the asphalt concrete were based on calculated temperatures, using (insofar as possible) information related to the Road Test location during the period of traffic testing.

Deflection values used for adjusting the subgrade moduli were the fall and spring rebound equations given in Reference 37.

Fifteen sections were included in this part of the investigation in order to find appropriate seasonal subgrade modular relationships. Accordingly, the specific values were as follows:

$$M_R \text{ (spring)} = 8000\ \sigma_d^{-1.06} \tag{A-10}$$

$$M_R \text{ (summer)} = 18{,}000\ \sigma_d^{-1.06} \tag{A-11}$$

$$M_R \text{ (fall)} = 27{,}000\ \sigma_d^{-1.06} \tag{A-12}$$

A resilient modulus of 50,000 psi (3.45×10^5 kPa) was assigned to the winter period when the untreated layers were substantially frozen. Higher values, up to 10^6 psi (6.9×10^6 kPa), could have been assigned, assuming the materials were completely saturated; however, since some air was expected to be present, it was considered possible that a lower value would not be unreasonable. In any event, the calculated damage during these periods was very low, since the tensile strain in the asphalt concrete was extremely low.

Table A-4 shows the comparison obtained in the fall and spring periods based on use of the PSAD program or regression Equations A-1 and A-2.

TABLE A-4

SURFACE DEFLECTIONS CALCULATED WITH SEASONAL MODULI FOR BASE, SUBBASE, AND SUBGRADE

	DEFLECTION (INCHES x 10^{-3})					
	SPRING			FALL		
SECTION NUMBER	AASHO EQUATION	PSAD	REGRESSION EQUATION	AASHO EQUATION	PSAD	REGRESSION EQUATION
Loop 4, L_1 = 18K (SAL)						
623(3-6-8)	66.9	66.5	63.2	37.2	37.9	36.7
597(4-3-8)	55.2	58.5	55.6	36.6	--	32.9
625(4-6-12)	35.6	42.5	42.8	26.4	25.1	25.7
579(5-3-4)	53.0	--	53.7	40.8	32.1	33.1
593(5-3-12)	30.1	--	39.4	26.0	23.3	23.9
629(5-6-4)	45.6	45.8	47.7	36.6	--	29.9
591(5-6-8)	34.3	--	40.3	29.2	25.0	25.0
581(5-6-12)	26.3	34.0	35.4	23.6	--	21.8
Loop 6, L_1 = 30K (SAL)						
253(4-6-16)	59.7	--		37.6	36.8	
267(4-9-12)	68.0	72.3		41.9	--	
315(5-3-16)	50.9	62.5		37.0	35.9	
259(5-6-8)	75.4	77.6		50.9	--	
331(5-9-12)	50.7	59.8		37.6	35.0	
271(6-9-8)	49.1	57.6		41.2	35.0	
333(6-9-16)	30.5	44.1		28.0	27.6	

The procedure used to calculate the shift factor from the laboratory curves given by Equation 7 can best be described by an example related to section 315 on loop 6. For these calculations the tensile strain in the asphalt concrete is obtained by using the material properties given in Table A-3 and by Equations A-10, A-11, and A-12 for the subgrade.

Section 315 was first observed to have 35 square feet of class 2 cracking per 1000 square feet of area (3.5%) on September 23, 1959, and 700 square feet (70%) on April 6, 1960.

In order to calculate damage in this section for each 12-hour period, the maximum tensile strain in the bottom of the asphalt concrete was determined by means of the regression Equation A-1 for a single-axle load of 30 kips (13,620 kg). The damage for each 12-hour period was then calculated according to the relationship

$$D_i = \frac{n_i}{N_i} \tag{A-13}$$

in which

D_i = damage during period i;

n_i = number of load applications applied during period i;

N_i = total allowable load applications for strain value calculated during period i, as obtained from Equation 7.

The accumulated damage thus obtained is shown in Table A-5.

Table A-5 suggests that if the laboratory data are correct the initial fatigue crack would have occurred in March or April of 1959. The

TABLE A-5

CUMULATIVE FATIGUE DAMAGE FOR SECTION 315 USING LABORATORY DATA

MONTH	YEAR	MONTHLY DAMAGE	CUMULATIVE DAMAGE
November	1958	0.03631	0.03631
December	1958	0.03580	0.07211
January	1959	0.03240	0.10451
February	1959	0.05584	0.16035
March	1959	0.35125	0.51160
April	1959	1.00411	1.51571
May	1959	2.12563	3.64134
June	1959	2.50896	6.15031
July	1959	2.74972	8.90003
August	1959	2.43565	11.33568
September	1959	1.69281	13.02849[a]
October	1959	1.18476	14.21325
November	1959	0.07842	14.29166
December	1959	0.08505	14.37671
January	1960	0.13833	14.51505
February	1960	0.14044	14.65548
March	1960	2.42192	17.07740[b]

[a]Less than 10 percent cracking.

[b]More than 45 percent cracking.

delay until September of 1959 is accounted for in terms of the amount of traffic required to propagate the cracking through the surfacing and to develop 3.5 percent areal cracking. Thus, the required shift factor would be 13.03 for this section in order to adjust the laboratory curves to correspond with field observations. A shift factor of 18.4 was obtained for cracking in excess of 45 percent.

Table A-6 summarizes similar results for 17 sections in loops 4 and 6. There was no indication from this analysis that a change in the thickness of the asphalt concrete from 4 inches to 6 inches would change the shift factor.

Because of a lack of sufficient information regarding the occurrence of cracking in the asphalt-treated base sections, the same equations are used for both conventional and full-depth sections. Future calibrations, as suggested in Chapter Three, may indicate a need to revise this conclusion.

The final distress prediction equations for fatigue cracking on the AASHO Road Test are as follows:

$$\log N_f\ (\leq 10\%) = 15.947 - 3.291 \log (\varepsilon/10^{-6}) - 0.854 \log (E^*/10^3) \quad \text{(A-14)}$$

$$\log N_f\ (\geq 45\%) = 16.086 - 3.291 \log (\varepsilon/10^{-6}) - 0.854 \log (\varepsilon/10^3) \quad \text{(A-15)}$$

in which the terms represent load applications, strain, and complex modulus as previously described.

TABLE A-6

SECTIONS USED IN FATIGUE DAMAGE ANALYSIS: EQUAL TO OR LESS THAN 10 PERCENT AREAL CRACKING

SECTION	CRACKING IN PERCENT OF AREA	DATE SECTION FAILED	NO. OF MONTHS IN SERVICE	PREDICTED DAMAGE	PREDICTED DAMAGE AFTER SHIFT OF 13.4
Loop 6					
315 (5-3-16)	3.5	9/23/59	11	13.03	0.97
307 (5-6-12)	1.8	12/02/59	13	14.94	1.11
327 (5-6-16)	9.8	4/06/60	17	13.83	1.02
313 (5-9-8)	4.5	10/07/59	12	17.08	1.27
331 (5-9-12)	0.3	3/23/60	17	14.91	1.11
265 (5-9-16)	0.7	4/20/60	18	13.54	1.00
255 (6-13-16)	0.3	11/04/59	12	9.48	0.70
271 (6-9-8)	0.5	4/06/60	17	14.17	1.05
311 (6-9-12)	10.7	5/18/60	19	13.62	1.01
333 (6-9-16)	3.3	5/04/60	18	9.87	0.73
Loop 4					
575 (4-3-12)	1.7	4/20/60	18	17.91	1.33
577 (4-6-8)	10.0	4/06/60	17	17.35	1.29
625 (4-6-12)	0.3	10/07/59	12	10.38	0.77
631 (5-3-8)	3.0	10/07/59	12	10.59	0.79
593 (5-3-12)	4.0	1/13/60	15	8.38	0.62
629 (5-6-4)	0.2	1/27/60	15	13.88	1.03
591 (5-6-8)	2.0	6/01/60	19	14.78	1.10
			Average	13.40	0.994

A simple procedure for estimating the load applications to 45 percent cracking, or more, is to multiply the value obtained at 10 percent by 1.38, the average ratio of the number of load applications at 10 percent cracking to those at 45 percent cracking as obtained from Equations A-14 and A-15. In theory, these relationships could apply to areas other than the AASHO Road Test, with or without spring thaw, providing the fatigue properties of the asphalt concrete do not vary significantly from those reported by Monismith and summarized in Reference 46. It is clear that an alternate set of reference fatigue curves would produce a different shift factor by yielding different monthly damage values.

DAMAGE MODEL FOR RATE OF RUTTING

Figure 5 illustrates the general trends for the occurrence of rutting at the AASHO Road Test. In order to develop a prediction model it was necessary to relate the seasonal rate of rutting to a primary response for the test sections from which data were available.

Thirty-two sections from lane 1 of loops 4 and 6 were selected to obtain the seasonal rate of rutting. Since traffic on loop-6 sections was applied with a 30-kip single-axle load, the number of load applications used to determine the rate of rutting on this loop was converted into 18-kip single-axle loads by multiplying with 7.94, the AASHO load equivalency factor.

Table A-7 illustrates typical values of the rate of rutting obtained from selected test sections at the AASHO Road Test. It is

TABLE A-7

SEASONAL RATE OF RUTTING (RR) FOR SELECTED SECTIONS ON THE AASHO ROAD TEST

LOOP NO.	THICKNESS (INCHES)	RATE OF RUTTING (INCHES/MILLION REPETITIONS OF 18-KIP SINGLE-AXLE LOAD)					
		SPRING*		SUMMER*		FALL*	
		1st YEAR	2nd YEAR	1st YEAR	2nd YEAR	1st YEAR	2nd YEAR
4	3-6-12	3.4	0.9	0.6	0.4	0.6	0.2
	4-6-8	4.4	0.5	0.8	0.4	0.3	0.2
	5-6-8	3.4	0.4	0.6	0.1	0.2	0.3
	5-6-8	3.0	0.4	0.7	0.1	0.3	(a)
6	4-9-16	0.7	0.04	0.1	0.04	-	-
	5-9-8	1.4	0.2	0.2	-	0.04	-
	6-9-8	0.9	0.2	0.2	(a)	0.04	(a)
	6-9-16	0.7	0.1	0.2	(a)	0.04	(a)

Note: 1 inch = 2.54 cm.

*Spring: March 25 to May 20, 1959, and March 20 to May 1, 1960.
Summer: May 20 to August 12, 1959, and May 1 to August 7, 1960.
Fall: August 12 to November 1, 1959, and August 7 to October 2, 1960.

(a) = No measurable change in rut depth.

apparent from the information shown that some inconsistencies in the observed rate of rutting occurred during testing. These inconsistencies could be due to field measurements, data analysis, use of load equivalency on loop 6, or true variations which must be expected in this type of data. In any case, such inconsistencies will influence the reliability of the prediction model.

A stepwise regression analysis procedure was used to correlate the rate of rutting with various combinations of primary response factors. The following independent variables were selected for this purpose:

1. Vertical surface deflection between dual tires.

2. Vertical subgrade strain under the center line of one wheel.

3. Vertical compressive stress at the bottom of the asphalt concrete layer under one wheel.

4. Horizontal tensile stress at the bottom of the asphalt concrete under one wheel.

5. Ratio of vertical and horizontal stresses from Items 3 and 4 above.

6. Cumulative traffic, expressed as equivalent 18-kip single-axle loads.

The analysis has indicated that the most significant correlations were obtained with vertical deflection at the surface of the pavement, followed by vertical compressive stress in the asphalt concrete, cumulative traffic, and vertical strain in the subgrade. The stress ratio factor was not considered sufficiently significant to be included in

the final prediction model. Since vertical strain in the subgrade was found to be highly correlated with surface deflection, it contributed little to the correlation when both factors were used in the model.

Equations 13, 14, and 15 in Chapter Two of this report summarize the prediction models obtained from this analysis.

LOW-TEMPERATURE CRACKING SUBSYSTEM

The objective of this subsystem is to estimate whether or not a given asphalt-aggregate combination is likely to develop transverse cracks when subjected to low temperatures. The major factors to consider are the thermally induced stresses in the asphalt concrete and the tensile strength of the same asphalt concrete at low temperatures and at slow rates of deformation.

In order to minimize the amount of testing required, it is recommended that a three-step approach be used by the analyst, as follows:

1. Estimate thermally induced stresses and tensile strength with no testing.

2. Estimate thermally induced stresses by nomographs and measure tensile strength by laboratory testing.

3. Test for both creep modulus (stiffness) and tensile strength of asphalt concrete and use computer program to compute and compare thermally induced stresses with tensile strength.

<u>Preliminary Estimate for Low-Temperature Cracking</u>

The purpose of this preliminary step is to estimate the thermally induced stress without the need for testing or calculations. The basic procedures are as follows:

1. Estimate critical air temperature.
2. Estimate temperature in surface of asphalt concrete.
3. Estimate stiffness properties of asphalt and asphalt concrete.
4. Estimate thermally induced stress.
5. Compare to limiting value.

No specific criteria have as yet been selected to estimate the critical temperature. A tentative procedure is to obtain the minimum 5-percentile temperature for the past 20 years; that is, from climatological records estimate the air temperature that would be exceeded 95 percent of the time.

An estimate of the temperature on the surface of the asphalt concrete can be made on the basis of information in References 13 and 19. For this estimate the temperature in the mat could be estimated by adding 10°F to the critical air temperature.

The stiffness of the asphalt cement and asphalt concrete can be estimated by the use of nomographs, as in References 12 and 46. For the use of these nomographs, a time of loading of 7200 seconds is recommended and the Heukelom bitumen test data chart should be used to obtain an adjusted ring-and-ball temperature based on a series of penetration tests on the unaged asphalt. Testing on unaged asphalt is recommended for this estimate because stress-induced estimates have been based on measurements made by this technique (14). The inputs required to estimate the creep modulus are (a) time of loading, 7200 seconds, (b) temperature difference between ring-and-ball temperature and critical

temperature, and (c) temperature susceptibility of the asphalt (penetration index). Procedures for obtaining this information are described in References 12 and 46.

The thermally induced stress can be estimated from Figure A-1, as developed by Christison (19). This is only a preliminary estimate of the thermally induced stress and is proposed for use only in this initial estimate.

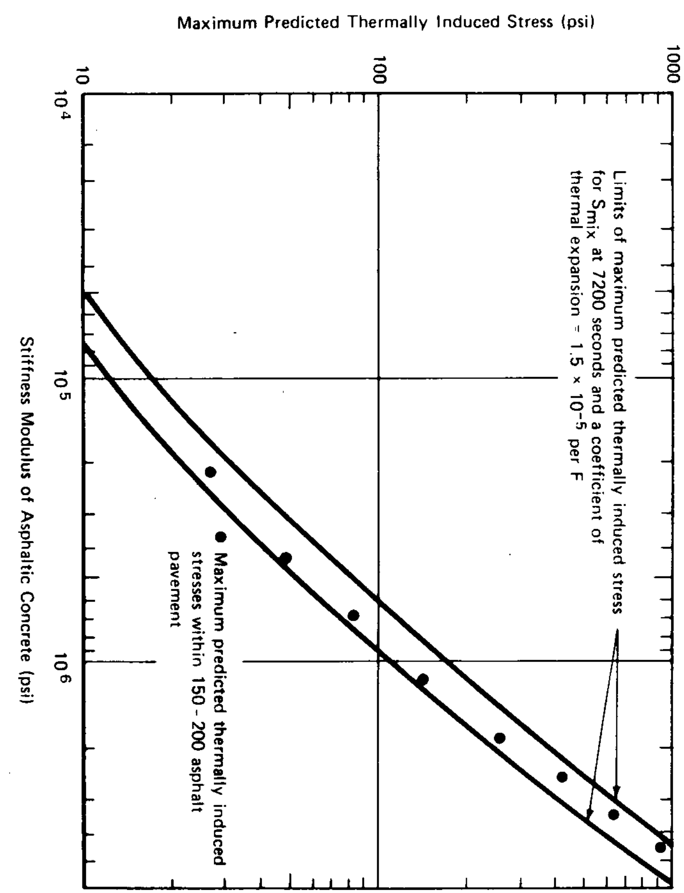
If the thermally induced stress exceeds 200 psi it can be concluded that there would be a possibility of low-temperature cracking. If the thermally induced stress is less than 200 psi it is unlikely that cracking would be predicted by the subsystem, and no further analysis is recommended.

Intermediate Estimate for Low-Temperature Cracking

The intermediate step will call for a measurement of the tensile strength at the critical temperature; e.g., a low temperature that would not be exceeded more than 5 percent of the time.

The tensile strength can be obtained in either of two ways: (a) split tension test, or (b) uniaxial tension test.

The split tension test, developed in Alberta, is described by Anderson and Hahn (59) and is included in Appendix H to this report. The uniaxial tension test was developed at the University of Waterloo and is described in Reference 12. A minimum of three specimens would be tested at each temperature.



(after Christison (19))

Figure A-1. STIFFNESS MODULUS OF ASPHALTIC CONCRETE VERSUS MAXIMUM PREDICTED THERMALLY INDUCED STRESS

If the tensile strength exceeds the estimated thermally induced tensile stress by a factor of 2.0, no further analysis is recommended.

Program COLD for Low-Temperature Cracking

Program COLD follows the flow diagram shown in Figure 2. Program COLD has been designed to (a) compute the thermal regime of the asphalt concrete, (b) compute the thermally induced tensile stress in the asphalt concrete, and (c) compare in tabular and graphic output the computed stress with measured tensile strength and indicate those conditions that could induce fracture.

The information required for program COLD is as follows:

1. Section thickness.
2. Thermal properties of component layers.
3. Ambient temperatures and solar radiation.
4. Initial ambient temperature gradient.
5. Creep modulus (stiffness) of asphalt concrete versus temperature of the mix.
6. Tensile strength of asphalt concrete at slow rates of loading, 0.01 inch (0.004 cm) per minute, versus temperature of asphalt concrete.

Program COLD is based on the work of Christison (19), with modifications made by the present project staff (specifically, to accommodate limited climatological data from weather reports and to provide for probability estimates based on variance associated with mix modulus and tensile strength).

Computer program COLD solves for thermally induced stresses for the specified time increments of one-eighth hour. These values are then accumulated and printed for every 2 hours of the day. Also, the program estimates the expected strength of the mix from strength-versus-temperature relationships, and the two values are compared to determine the possibility of thermal cracking at every 2-hour time interval for all of the days included in the analysis.

It is recommended that "real" temperature data be used in implementing the program. That is, obtain climatological information from weather reports for the location being studied, pick 10 or 15 days for each time period being studied, and use that information as input to the program. "Synthetic" information can be misleading, since the temperature history during the entire test period will influence results. The program is designed to use only maximum and minimum temperature values as reported in standard weather data.

Thermal Properties of Component Layers. The following thermal properties of component layer materials are required in this analysis: (a) thermal conductivity, and (b) heat capacity. Thermal conductivity, K, expresses the rate of heat flow through a unit area under a unit temperature gradient. The most commonly used units of K are Btu/hr/ft/^{o}F or Cal/sec/cm/^{o}C. Btu units are used in this program.

The heat capacity, C, is the amount of thermal energy required to cause a 1-degree change in a unit mass of the material. The units used in this analysis are Btu/lb/^{o}F.

Thermal properties of pavement component layers can be grouped as follows: (a) thermal properties of surface mixes, and (b) thermal properties of granular materials and fine-grained soils.

Thermal Properties of Surface Mixes. Surface mixes are usually a mixture of aggregate and bituminous binders. Therefore it is reasonable to assume that the properties of these mixes will be governed by the thermal properties of their ingredients. However, from the studies reported (60, 61, 62, 63), thermal conductivity and heat capacity values of asphalt mixes have been found to vary within narrow limits. Also, considering the moisture content of such mixes to be negligible, the properties may be assumed to be temperature-independent. The following values are suggested for use in the absence of actual laboratory test results:

Thermal Conductivity = 0.82 - 0.86 Btu/hr/^{o}F

Heat Capacity = 0.20 - 0.22 Btu/lb/^{o}F

Thermal Properties of Granular Materials and Fine-Grained Soils. Thermal properties of these materials are primarily dependent on moisture content, density, and temperature. In temperature-prediction analyses of pavement structures in low-temperature climates, three different sets of these thermal properties must be recognized, depending upon whether the material is in an unfrozen, freezing, or frozen condition. The equations suggested by Kersten (61) are as follows:

Fine-Textured Soils:

Unfrozen: $$k_u = \frac{(0.9 \log w - 0.2)\ 10^{0.01\gamma d}}{12} \qquad \text{(A-16)}$$

Frozen: $$k_f = \frac{0.01(10)^{0.022\gamma d} + 0.085(10)^{0.008\gamma d}(w)}{12} \qquad \text{(A-17)}$$

Granular Soils:

Unfrozen: $$K_u = \frac{(0.07 \log w + 0.4)10^{0.01\gamma d}}{12} \qquad \text{(A-18)}$$

Frozen: $$K_f = \frac{0.076(10)^{0.013\gamma d} + 0.032(10)^{0.146\gamma d}(w)}{12} \qquad \text{(A-19)}$$

in which

K_u = thermal conductivity of unfrozen soil, Btu/hr/ft/^{o}F;

K_f = thermal conductivity of frozen soil, Btu/hr/ft/^{o}F;

γd = dry density, pcf; and

w = moisture content, percent by dry weight of soil.

Kersten (61) considered the above equations to be accurate within ±25 percent and suggested that soils with 50 percent or more silt and clay-sized particles less than 0.05 mm be defined as fine-textured soils and those with less than 50 percent silt and clay-sized particles be defined as granular soils. Kersten indicated that Equations A-16 and A-17 are valid for soils having a moisture content of 7 percent or more and that Equations A-18 and A-19 are valid for soils having a moisture content of 1 percent or greater.

The heat capacity of dry granular materials and fine-grained soils was also investigated by Kersten and was found to vary from approximately 0.15 to 0.19 Btu/lb/^{o}F at 0^{o}F and 140^{o}F, respectively. Since moisture is invariably present in these materials when they are used in pavement structures, the following relationships were suggested for estimating the specific heat of a soil-water system:

$$Cu = \frac{100\ Cs + 1.0W}{100 + w} \qquad \text{(A-20)}$$

$$Cf = \frac{100\ Cs + 0.5W}{100 + w} \qquad \text{(A-21)}$$

in which

Cu = heat capacity of unfrozen soil, Btu/lb/^{o}F;

Cf = heat capacity of frozen soil, Btu/lb/^{o}F;

Cs = heat capacity of dry soil, Btu/lb/^{o}F; and

w = moisture content of soil, percent by dry weight of soil.

The values 1.0 and 0.5 in the above equations are the heat capacities, expressed in units of Btu/lb/^{o}F, of water and ice, respectively.

Temperature and Solar Radiation. The present model uses daily temperature and solar radiation data along with surface absorption and emissivity properties to compute surface boundary and heat transfer to pavement structure. Since so many factors are involved in this process, a detailed discussion of the computational process of this phenomenon is not possible here; the thesis by Christison (19) presents further details on the topic.

The original model was developed and tested for hourly temperature and solar radiation data. Since it is not always possible to acquire such information for all locations or from standard climatological reports, it was decided to use minimum and maximum temperature data along with solar radiation data for the entire day. A comparison with the results of hourly data indicated that the extreme pavement temperatures computed by using this simplification did not differ significantly, so the method could be used without sacrificing the reliability of results.

Studies have indicated that asphalt-mix surface absorptivity varies from 0.85 to 1.00 and that emissivity values range from 0.93 to 1.00. Thus, suitable values can be adopted for practical purposes in the absence of actual laboratory tests on the given mix.

Solar radiation data of reasonable reliability may be obtained from the Climatic Atlas of the United States (54).

Initial Temperature Gradient. The initial temperature gradient in the pavement structure is assumed to provide the initial boundary conditions for the solution of the problem. Unless facilities for such measurements are available at the given site, an assumed gradient has to be provided for computer solution of the problem. Experience in this area indicates that any reasonable assumption of the initial (beginning of the first day) temperature gradient can provide a solution that can be reliably used for further analysis. However, it was observed that the thermal regime predicted for the first few days was dependent on the assumed values of the initial temperature gradient. This dependency declined each day until about the fifth or sixth day, when the predicted thermal regime was essentially independent of the initial temperature

gradient. It is therefore recommended that provisions should be made in the data to account for these possible variations by starting the analysis about six days in advance of when the critical temperature conditions are expected.

Creep Modulus Versus Temperature. Creep modulus of the surface mix, which is generally defined as the ratio of axial stress to strain, is a time- and temperature-dependent property. Therefore a suitable relationship between stiffness and temperature has to be established by either testing the given mix or using nomographs from the preliminary analyses herein. If the nomograph is to be used for obtaining stiffness values, a time of loading of 7200 seconds should be used with the van der Poel procedure (11, 46). To convert asphalt stiffness into mix stiffness, the following formula is used:

$$S_{mix} = S_{asphalt}\left(1 + \frac{2.5}{n} \cdot \frac{Cv}{1-Cv}\right)^n \qquad \text{(A-22)}$$

in which

S_{mix} = mix stiffness, kg/cm^2;

$S_{asphalt}$ = asphalt stiffness, kg/cm^2;

Cv = volume concentration of aggregate, defined as

$\dfrac{\text{volume of aggregate}}{\text{volume of (aggregate + asphalt)}}$; and

$$n = 0.83 \log \left(\frac{4 \times 10^5}{S_{asphalt}}\right).$$

Tensile Strength of Asphalt Concrete Versus Temperature. Tensile strength should be determined in accordance with procedures described by Anderson and Hahn (59) or Haas (12).

Data Analysis and Interpretation of Results

Computer program COLD, as described in Appendix C to this report, is used to analyze the data collected for this purpose. The input to this program is transformed into initial and boundary conditions of the heat transfer equations described earlier. A numerical solution of these equations is then obtained by use of this computer program. Pavement temperatures thus obtained are used in computing thermal stresses induced by the changing temperature regime of the pavement.

Results of this analysis are printed as 2-hour values of air temperature, pavement temperature, and expected values of tensile strength of mix and induced stresses. Also, strength and induced stresses are computed at 75, 90, 95, and 99 percent reliabilities and are printed along with the above-mentioned output.

Provisions are made within computer program COLD so that the computed stress is compared with the expected tensile strength at each 2-hour time interval. In case this stress exceeds the tensile strength of the mix, the possibility of thermal cracking is indicated by printing an asterisk on the right-hand side of the computed tensile stress value.

RELIABILITY CALCULATIONS IN PREDICTION OF FATIGUE CRACKING

Appendixes D and G describe the probabilistic aspects of PDMAP and COLD. In order to illustrate how variability influences predictions,

the following information is provided as it pertains to fatigue cracking.

Fatigue damage is calculated using Miner's hypothesis. Thus,

$$D = \sum d_i = \sum \frac{n_i}{N_{f_i}} \qquad \text{(A-23)}$$

in which

D = cumulative fatigue damage;

d_i = fatigue damage during i^{th} time period;

n_i = repetitions of 18-kip-equivalent single-axle load during i^{th} time period; and

N_{f_i} = repetitions of 18-kip-equivalent single-axle load to cause failure during the i^{th} time period.

The fatigue-life parameter N_{f_i} is related to the horizontal strain ε_i at the base of the asphalt concrete layer and the dynamic modulus $|E_i^*|$ of that by the following regression equation:

$$\log_{10} N_{f_i} = K_1 + K_2 \log_{10} \varepsilon_i + K_3 \log_{10} |E_i^*| + \lambda \qquad \text{(A-24)}$$

where K_i's are the regression coefficients and λ is the random error term with zero mean and variance of σ^2.

Using a first-order, second-moment approach, the mean and the variance of the cumulative fatigue damage, D, can be calculated from Equation A-23. Thus,

$$\bar{D} = \sum \bar{d}_i = \sum \frac{\bar{n}_i}{\bar{N}_{f_i}} + \sum \frac{\bar{n}_i}{\bar{N}^3_{f_i}} \sigma^2_{N_{f_i}} \qquad \text{(A-25)}$$

$$\sigma_D^2 = \sum \frac{1}{\bar{N}_{f_i}^2} \sigma_{N_{f_i}}^2 + \sum \left(\frac{\bar{n}_i}{\bar{N}_{f_i}^2} \right)^2 \sigma_{N_{f_i}}^2 \qquad \text{(A-26)}$$

To illustrate the calculations of mean and variance of fatigue damage, let us consider the following example in which fatigue damage during the first year in the analysis period is considered.

TRAFFIC DATA

TIME PERIOD	n_i
1	3,000
2	15,000
3	30,000
4	15,000

Coefficient of variation of traffic = 0.10.

HORIZONTAL STRAIN ε_i

TIME PERIOD	$\bar{\varepsilon}_i$	σ_{ε_i}
1	8.04×10^{-5}	5.39×10^{-6}
2	1.65×10^{-4}	1.09×10^{-5}
3	1.87×10^{-4}	2.15×10^{-5}
4	1.68×10^{-4}	1.46×10^{-5}

MODULUS $|E_i^*|$

TIME PERIOD	$|\bar{E}_i^*|$
1	1.95×10^6
2	1.19×10^6
3	0.33×10^6
4	0.70×10^6

Coefficient of variation of $|E^*| = 0.25$.

The data of ε_i and $|E_i^*|$ are taken from the PDMAP solution of a typical problem. As a simplification, constant properties are assumed over a day in a given time period (i.e., a day is not divided into two intervals).

Regression Data

$K_1 = -1.234$

$K_2 = -3.29$

$K_3 = -0.854$

$\sigma_\lambda = 0.3$

From Equation A-24 it is possible to calculate mean and variance of fatigue life N_{f_i}; these are shown below:

Time Period	$\bar{N}_{f_i}$	Var N_{f_i}
1	10.09×10^6	78.01×10^{12}
2	1.4×10^6	1.198×10^{12}
3	3.04×10^6	8.69×10^{12}
4	2.189×10^6	3.96×10^{12}

Using Equations A-25 and A-26, mean and variance of cumulative fatigue damage, D, during the first year can be calculated; these are:

$$\bar{D} = 0.0537$$

$$\sigma_D^2 = 0.0001$$

$$\sigma_D = 0.0133$$

Fatigue damage with a given reliability can be calculated from the equation of the following type:

$$D_R = \bar{D} + K_R \, \sigma_D \qquad \text{(A-27)}$$

where the factor K_R is dependent on the specified reliability. Assuming normal distribution for cumulative fatigue damage, K_R can be found from normal tables. For example, for a reliability level of 99 percent, K_R is 2.33; using Equation A-27.

$$D_{99} = \bar{D} + 2.33 \, \sigma_D = 0.0848 \qquad \text{(A-28)}$$

It may be noted that because of a large standard error of 0.3 in the regression Equation A-24, the uncertainty in the fatigue life N_{f_i} is high, as indicated by a coefficient of variation of the order 80 to 90 percent. However, the coefficient of variation of a typical fatigue-damage term $d_i = \frac{n_i}{N_{f_i}}$ is reduced to about 40 to 50 percent. There are two reasons for this reduction:

1. n_i and N_{f_i} are assumed to be uncorrelated.
2. The relationship $d_i = \frac{n_i}{N_{f_i}}$ is nonlinear; the mean $\frac{n_i}{N_{f_i}}$ is not equal to $\frac{\bar{n}_i}{\bar{N}_{f_i}}$; i.e., the mean of the ratio is not equal to the ratio of the means. The mean of $\frac{n_i}{N_{f_i}}$ is in fact equal to:

$$\frac{\bar{n}_i}{\bar{N}_{f_i}} + \left(\frac{\bar{n}_i}{\bar{N}_{f_i}^3}\right) \sigma_{N_{f_i}}^2$$

Thus the mean of the ratio d_i is increased without a proportionate increase in the standard deviation of d_i. This reduces the coefficient of variation.

There is a further reduction in the coefficient of variation of the cumulative fatigue damage $D = \sum d_i$. The cumulative damage D has a coefficient of variation of the order 20 to 30 percent. This reduction is due to the assumption that the individual damage terms d_i are uncorrelated. This is a linear filtering effect wherein the sum of uncorrelated random variables has a smaller coefficient of variation than that of an individual variable. Loosely speaking, the reduction in the coefficient of variation of the sum results from the fact that if some of the variables take high values, this may be compensated by low values of other variables in the sum. The chance of all the variables assuming extreme values — low or high — would be extremely small. The sum of uncorrelated variables therefore tends to fall close to the sum of the average values of the variables exhibiting significantly smaller dispersion than the individual variables.

METHODS FOR ESTIMATING OR MEASURING MATERIAL CONSTANTS

The single factor of greatest concern to state engineers was the ability to characterize materials for use in the structural subsystems. Such testing would require equipment that is not now available in most laboratories.

The test methods described in Appendix H to this report illustrate test equipment; however, machine drawings are not included in these descriptions. Some drawings can be obtained from the University of California, Berkeley, or from the Asphalt Institute, College Park, Maryland.

Materials are to be characterized in accordance with procedures applicable to linear elasticity designed to recognize time and temperature, asphalt systems, and stress; unbound soils including base, subbase, and subgrade materials.

Each of the materials in the structural section, plus the subgrade, should be tested in order to develop appropriate input information for each analysis period.

It is difficult, if not impossible, to evaluate all possible conditions to be expected in the field, including both age and climate; however, every effort should be made to obtain representative test data.

Asphalt Concrete and Emulsified Asphalt Mixes. Methods for characterizing materials should be compatible with methods used in developing damage prediction models. For asphalt concrete, characterization was accomplished at 10 hertz at 100°F, 70°F, and 40°F. Estimates of modulus were obtained by plotting a smooth curve on a semilog plot. The maximum modulus at temperatures below 32°F (frozen foundation) can be estimated to be 3.5×10^{6} psi (24.1×10^{6} kPa). At elevated temperatures the modulus should level off at approximately 50,000 psi (3.45×10^{5} kPa).

For emulsified asphalt mixes (EAM) the criteria reported by Chevron Research Corporation (48) should be used for characterizing materials. The modulus of such mixes was obtained by pulse loads (0.1 sec loaded and 0.9 sec unloaded) using a diametral configuration described by Schmidt (43). Tests were normally conducted at temperatures of 100°F and 73°F and were plotted as a straight line on a modified log-log scale. Figure A-2 illustrates a typical plot for EAM showing the influence of curing time on modulus.

The time for an EAM to reach its final moduli is critical in predicting the traffic or time required to cause fatigue cracking. Reference 48 describes such procedures in detail. In order to use PDMAP it is necessary to subdivide the first year or two years into damage periods according to the rate of curing predicted for the EAM. Thus, the first year could be divided into four time periods, each with a temperature modulus relationship as shown on Figure A-2.

Although it is recommended that materials should always be characterized by laboratory testing, it is recognized that simplified and somewhat approximate methods are useful for preliminary estimates and trials with the distress prediction subsystems.

For asphalt concrete, two procedures for estimating the complex modulus are available: (a) indirect estimations, as originally developed by van der Poel and expanded for use with asphalt concrete by Huekelom and Klomp, and (b) regression equations by Shook and Kallas (64).

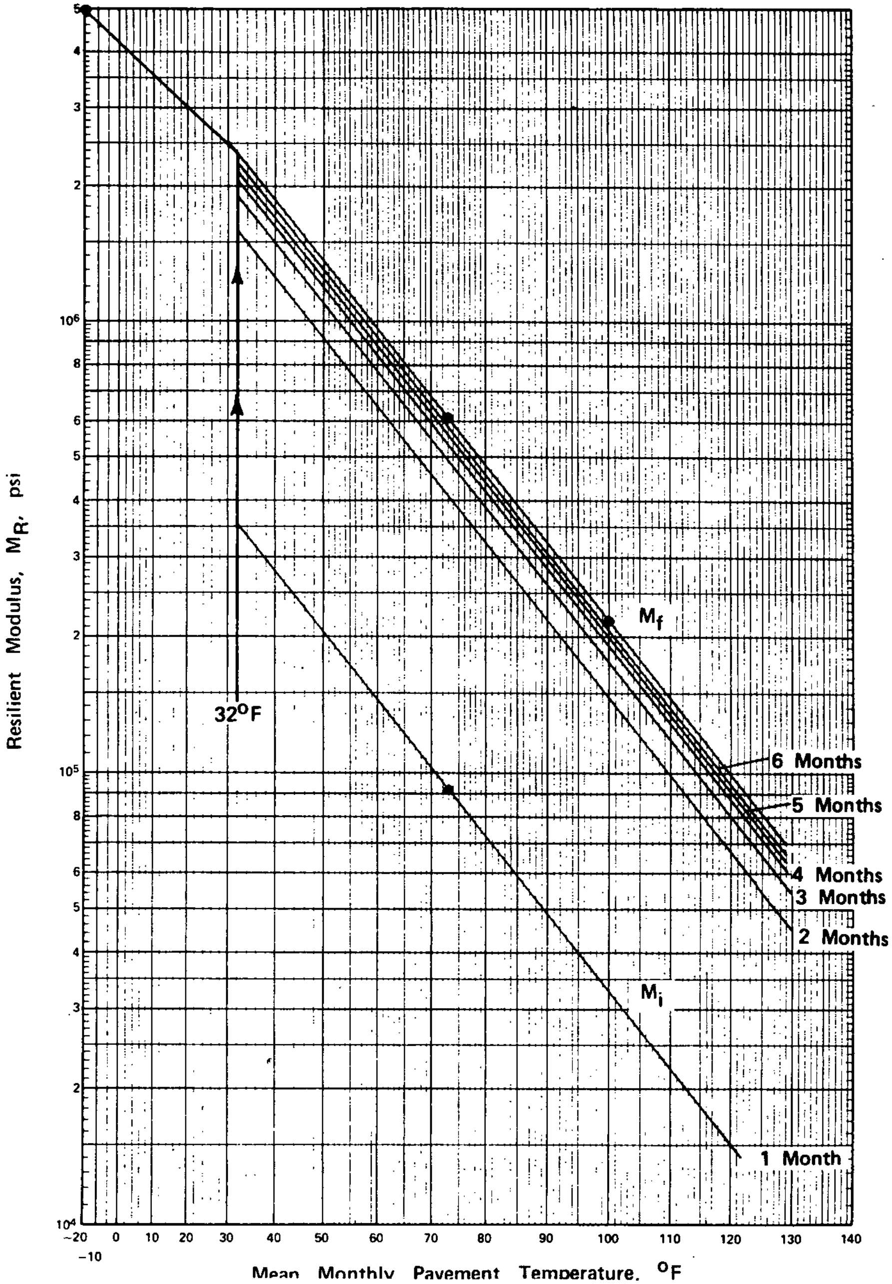


Figure A-2. MODULUS - PAVEMENT TEMPERATURE RELATIONSHIP FOR EMULSIFIED ASPHALT MIXES

The van der Poel procedure provides a convenient method for representing a large range in asphalt properties, time of loading, and temperature. All that is required is data from the ring and ball softening point, TR&B (ASTM D36), and the penetration at 77°F. The accuracy of such procedures is said to be of the order of a factor of 2. Nomographs developed by Huekelom and Klomp were formulated to convert asphalt modulus (referred to as stiffness by van der Poel et al.) to mixture modulus based on the volume concentration of the aggregate in the mixture.

The Heukelom and Klomp nomographs are based on an air-voids content of 3 percent. For air voids larger than 3 percent, a correction in the volume concentration factor has been proposed by Van Draat and Sommer.

A complete description of the procedures of van der Poel and Huekelom and Klomp can be found in the final report for NCHRP Project 9-4 or in the Asphalt Institute Research Report RR 73-1, "A Working Method for Designing Asphalt Pavements to Minimize Low-Temperature Shrinkage Cracking" by R.C.G. Haas.

Shook and Kallas (64) have published equations developed from test data on a variety of mixes using multiple regression techniques. These regressions were based on data obtained at 4 hertz rather than 10 hertz as required for PDMAP. A 10 to 20 percent increase in the predicted modulus at 4 hertz (increasing with temperature) should provide a reasonable first approximation of the modulus at 10 hertz.

The equations by Shook and Kallas (64) are:

$$\log |E^*| = 1.54536 + 0.020108(x_1) - 0.0318606(x_2) + 0.068142(x_3) - 0.00127003(x_4)^{0.4}(x_5)^{1.4}$$

$$R^2 = 0.968 \quad \text{S.E.} = 0.0888904 \qquad \text{(A-29)}$$

$$\log |E^*| = 3.12197 + 0.0248722(x_1) - 0.0345875(x_2) \quad 9.02594 \frac{(x_4)^{0.19}}{(x_6)^{0.9}} \qquad \text{(A-30)}$$

$$R^2 = 0.971 \quad \text{S.E.} = 0.0849$$

in which

$|E^*|$ = dynamic modulus, 10^5 psi (4 hertz);

x_1 = percent minus No. 200 material in the aggregate;

x_2 = percent air voids in the mix;

x_3 = asphalt viscosity at 70°F, 10^6 poises;

x_4 = percent asphalt by weight of mix;

x_5 = test temperature, °F; and

x_6 = log viscosity of asphalt at test temperature, poises.

A less reliable correlation with Marshall stability has also been reported by Shook and Kallas (64) as follows:

$$\log |E^*| = 0.124262 + 1.25469(K) - 0.0616215(V) \qquad \text{(A-31)}$$

$$R^2 = 0.900 \quad \text{S.E.} = 0.151416$$

in which

$|E^*|$ = dynamic modulus, 10^5 psi (4 hertz);

K = log of Marshall stability (pounds) divided by 100 times Marshall flow value (0.01 inch); and

V = percent air voids for the modulus specimen minus percent air voids for the Marshall test specimen.

A first approximation of the modulus of an emulsion mix can be obtained by assuming the cured modulus to be represented by the M_f curve in Figure A-2. The rate of strength development can be established on Figure A-2 according to the following equation and tabular information:

$$M_t = M_f - (M_f - M_i)\ (RF) \qquad \text{(A-32)}$$

in which

M_t = total modulus for the specified time after construction, psi;*

M_f = final modulus (measured at 73°F after three-day air cure and four-day vacuum cure at room temperature), psi;

M_i = initial modulus (measured at 73°F after one-day air cure), psi; and

RF = early cure reduction factor, as shown below.

*Note: 1 psi = 6.9 kPa.

	REDUCTION FACTOR				REDUCTION FACTOR
MONTH	SIX-MONTH CURE	ONE-YEAR CURE	TWO-YEAR CURE	MONTH	TWO-YEAR CURE
1	1.0	1.0	1.0	13	0.198
2	0.37	0.62	0.78	14	0.175
3	0.225	0.48	0.69	15	0.154
4	0.136	0.37	0.62	16	0.136
5	0.082	0.29	0.545	17	0.120
6	0.05	0.225	0.48	18	0.105
7	-	0.175	0.42	19	0.093
8	-	0.136	0.37	20	0.082
9	-	0.105	0.33	21	0.073
10	-	0.082	0.29	22	0.064
11	-	0.064	0.255	23	0.057
12	-	0.05	0.255	24	0.05

This relationship assumes that the rate of cure of the mix is rapid at first and then levels off, reaching 95 percent of its final modulus at the end of the specified cure period. For the example in Figure A-2, a six-month field cure period was selected. For most practical purposes a reduction factor for the one-year cure can be used. The dry deserts of California and Arizona or the southern portions of Texas and Florida could be assigned a six-month cure. Places at high altitudes in the

Rocky Mountain states could have a two-year cure, as could the northern states.

A complete description of the procedures for characterizing emulsion mixes can be found in Reference 48. A comprehensive summary of information regarding dynamic testing of asphalt concrete can be found in Reference 65, published by ASTM. Reference 66, by R.J. Schmidt, also provides useful information about the influence of temperature and water content on the modulus of asphalt systems.

For predicting fatigue cracking in overlays, it is recommended that the in-situ surfacing be assumed to have achieved a damage ratio of 1.0 with an assigned modulus ranging from 50,000 psi (3.45×10^5 kPa) to 75,000 psi (5.18×10^5 kPa), depending on the severity of observed cracking in the old pavement. Thus, the model has limited capability as regards use in stage construction or preventive maintenance considerations. Reference 53 presents further recommendations for overlay design that are based on methods comparable to those used in this investigation.

Unbound Aggregates. Unbound aggregates should be tested in triaxial configuration, as described in Appendix H. Every effort should be made to represent the anticipated field conditions, particularly with regard to the water content and density. Deviator stress levels of 7.5, 15, 30, and 60 psi have been found adequate to cover most field conditions. At each deviator stress level, confining pressure varies to produce principal stress ratios (σ_1/σ_3) of 4, 2.5, and 2.0. One thousand load applications are applied to condition the specimens at the lowest deviator stress and the highest principal stress ratio. After conditioning, 200 load repetitions are applied at each deviator stress-confining pressure test condition.

Some effort should be made to test aggregates at relatively high water content, approaching saturation. This can be accomplished by introducing boiled distilled water to the bottom of the specimen in the triaxial cell and applying a partial vacuum to the top of the specimen. When for a given time interval the water passed through the sample is the same as the volume of water entering the sample, and no air is observed in the water passing through the sample (generally about 4 hours), the sample can be considered saturated. The question of whether to test in the drained or undrained condition is as yet unresolved.

Monismith et al. (56), after considerable testing in the laboratory (undrained) and on prototype test sections, report the following:

> *One of the most significant findings is the fact that water content, while causing a reduction in the modulus as it is increased, did not cause a marked reduction in modulus when the material was saturated. That is, liquefaction under repetitive loading was not obtained and only relatively small pore water pressures were observed in the prototype pavement as well as in the laboratory specimens.*

Appendix B of Reference 56 shows that the method of conditioning the laboratory sample could influence the measured moduli at load applications in excess of 25,000.

Hicks and Monismith (67) have shown that the elastic properties of granular materials are affected by factors such as aggregate density, percent passing No. 200 sieve, aggregate type (crushed or uncrushed), and degree of saturation. At a given stress level the modulus increases with increasing density, increasing particle angularity or surface roughness, decreasing fines content, and decreasing degree of saturation. These investigations indicated that the Poisson ratio could also vary with the above properties to such a degree as to have a significant effect on most primary reponse factors. However, variations in the Poisson ratio from 0.35 to 0.50 had only a minor effect on tensile strain in the surface layer.

When materials are not tested it will be necessary to estimate material properties. Most base and subbase materials reported in the literature will produce modular relationships within the limits described by $M_r = 8000\ \theta^{0.6}$ and $2000\ \theta^{0.6}$. It is recommended that base materials be assigned relationships between $4000\ \theta^{0.6}$ and $7000\ \theta^{0.6}$ depending on the amount and plasticity of the fines, density, angularity of the aggregate, and degree of saturation. Subbase materials can be assigned values between $2000\ \theta^{0.6}$ and $5000\ \theta^{0.6}$ with the same considerations as those given for the base materials.

For purposes of the fatigue-cracking subsystem it is recommended that a Poisson ratio of 0.45 be used in lieu of testing.

Soil-Cement. Soil-cement as defined herein generally refers to those combinations of soil and cement that comply with accepted gradation and plasticity or sand-equivalent requirements and that demonstrate satisfactory resistance to weight loss when subjected to wet-dry or freeze-thaw testing in accordance with ASTM procedures D559 or D560 (68). Cement-treated bases designed on the basis of compressive strength can also be included; however, it should be noted that the damage model was developed for materials complying with wet-dry or freeze-thaw requirements.

The modulus of elasticity for cement-treated materials can vary from 1×10^6 to 3×10^6 psi (6.9×10^6 to 20.7×10^6 kPa), depending on the quality of material. Material characterization may be accomplished according to ASTM method C469 or its equivalent. In the absence of tests, a value of 2.0×10^6 is not unreasonable with a Poisson ratio of 0.15.

For design of overlays, after the cement-treated layer has exhibited fatigue cracking, a value of 50,000 psi, comparable to the cracked asphalt concrete, may be assigned to this layer for prediction cracking in the asphalt concrete overlay.

Cohesive Soils (Subgrade Materials). Cohesive soils used as subgrade material should be tested in accordance with procedures described in Appendix H. Particular care should be taken to attempt to duplicate field conditions as much as possible. The results of laboratory tests should be plotted on log-log paper and a regression equation developed in the expression given by Equation A-8. Tests are performed at

deviator stresses ranging from 6 to 12 psi in accordance with recommended procedures.

If existing roadways are available for testing, seasonal variations in subgrade properties may be estimated according to variations in measured deflections. Reasonable estimates of subgrade modulus can be made by comparing calculated deflections with measured deflections. With this information an iterative structural analysis procedure is followed which allows the subgrade modulus to be adjusted seasonally to match measured surface deflections.

In the event that no laboratory testing is possible, estimates may be made; however, it must be recognized that such estimates could be off by a factor of 2.

The subgrade modulus may be approximated from the CBR test value according to the following relationship:

$$E(\text{psi}) = 1500 \times \text{CBR} \quad (\text{Ref. } \underline{69}) \qquad \text{(A-33)}$$

Van Til et al. (40) have shown a general relationship between various routine laboratory test values and the resilient modulus.

Santucci (48) reports information from the San Diego Test Road (California) indicating that the relationship between the R-value and the resilient modulus can be described by the following equation:

$$R = 4.43 + 2.19 M_R \qquad \text{(A-34)}$$

in which

R = resistance value; and

M_R = resilient modulus, in 10^3 psi.

The Poisson ratio for cohesive soils may be assigned the value of 0.45 for purposes of the fatigue subsystem.

APPENDIXES B, C, D, E, F, AND G

Appendixes B through G have been compiled in a separate publication prepared by Materials Research & Development in accordance with editorial policies of NCHRP. It is designed for those users who intend to study the details of the programs concurrent with their implementation. A brief resume of each appendix is given below.

Appendix B. USER'S MANUAL FOR THE COMPUTER PROGRAM PDMAP

The basic objective of the program PDMAP (Probabilistic Distress Models for Asphalt Pavement) is to enable the highway department personnel to predict distress conditions of given pavement sections. The two specific distress modes considered in the program are fatigue cracking and permanent deformation due to traffic (rut depth). Because of the uncertainties in the measurement of material properties, in the estimation of traffic, and in the damage model itself, exact predictions of fatigue and rut depth are not possible. The program PDMAP, therefore, employs probabilistic analysis that computes the expected amount of damage as well as damages with specified reliability factors at any time during the analysis period.

Appendix C. USER'S MANUAL FOR THE COMPUTER PROGRAM COLD

This computer program was developed for the Computations of Low-Temperature Damage (COLD) in a given pavement system. It can accommodate any pavement structure of one, two, or three layers and combines two different computer programs, one for predicting temperatures in the layered pavement system and the other for predicting thermal stresses in the surface layer due to the temperature changes. Both of these programs were developed by J.T. Christison for his doctoral thesis under the direction of Prof. K.O. Anderson at the University of Alberta. Documentation for this program is contained in Reference 19.

Appendix D. DOCUMENTATION OF THE COMPUTER PROGRAM PDMAP, INCLUDING DICTIONARY OF TERMS

The program PDMAP is written in standard ANSI FORTRAN IV. It is compatible with both CDC and IBM computers. A typical design problem with an analysis period of 25 years was run on a CDC-7600 with MNF compiler and on an IBM-370 with H-level compiler. The CPU times on CDC and on IBM were 70 seconds and 328 seconds, respectively, for the probabilistic analysis of a typical problem. For deterministic solution the CPU times are cut down by about 80 percent. A core storage of 106K in octal is required for the execution of the program on the CDC-7600. When running on the IBM system, an H-level compiler with option 2 seems to give the best code optimization of the program.

Appendix E. DICTIONARY OF TERMS USED IN COMPUTER PROGRAM COLD

A table of terms used in the computer program COLD, including all the variables and their appropriate units, is presented in this appendix.

Appendix F. PROBABILISTIC ANALYSIS IN PROGRAM PDMAP

The probabilistic analysis in PDMAP consists of the following parts:

1. probabilistic solution for primary structural response,
2. probabilistic evaluation of the parameters required in the damage prediction models,
3. probabilistic prediction of cumulative fatigue distress, and
4. probabilistic prediction of cumulative permanent deformation.

The basic probabilistic solutions were originally furnished by Prof. W. Hufford of the University of Utah as part of an FHWA-sponsored research project. The procedures were adapted to PDMAP by the project staff.

Appendix G. PROBABILISTIC ANALYSIS IN THE PROGRAM COLD

The program COLD calculates expected values and values with different reliability levels of tensile strength of asphalt concrete and thermally induced stress in asphalt pavement subjected to low temperatures.

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